REPORT

Dannevirke Raw Water Reservoir

Stage 2 Geotechnical Interpretation Report

Prepared for

Tararua District Council

Prepared by

Tonkin & Taylor Ltd

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Executive summary

Background

Following a leak from Dannevirke Raw Water Reservoir in 2021, Tonkin & Taylor Ltd (T+T) confirmed a dam safety deficiency related to abnormally high seepage rates at the subsoil drain outlet. The "Confirmed Dam Safety Deficiency" comprises a lack of filter compatibility between the existing subsoil bedding and surrounding materials, which is indicated based on information on as-built drawings and is consistent with observed damage to the liner system and subgrade.

A sequence of steps that could lead to an uncontrolled loss of reservoir contents, or "potential failure mode", was identified in relation to this deficiency. The steps in the potential failure mode are presented in Figure 0.1. Step 3 in this potential failure mode is known to be continuing to deteriorate. There are identified vulnerabilities to other steps in the potential failure mode, but initiation and progression of these other steps remains unavoidably uncertain.

T+T was subsequently engaged by Tararua District Council (TDC) to design remedial works to address the potential failure mode. We have previously presented a 25% design stage of the detailed design in "Dannevirke Raw Water Reservoir Remediation – Design Report" (Ref 1020688.4000, T+T September 2023).

This report

TDC is currently considering whether to proceed with remedial works to resolve the confirmed dam safety deficiency and potential failure mode described above, or whether to instead pursue alternative water supply options and decommission the Dannevirke Raw Water Reservoir.

To inform these decisions, T+T has been asked to provide professional advice on four selected issues relating to dam performance:

a. Stability of the western reservoir rim.
b. Stability of the eastern dam embankment.
c. Potential for internal erosion of the eastern dam embankment, including dam foundation, and backfill around the subsoil outlet pipe.
d. Urgency of remediating the existing liner, subgrade, and underlying subsoil network.

We understand that TDC is seeking advice on the first three issues because of their implications for cost of remediation. TDC has advised that cost is a key factor in their decision to remediate the dam or instead explore alternatives. T+T has previously[1] prepared cost estimates for remediation, which covered options with different extents of works. TDC's feedback was that the options with a downstream berm or complete rebuild of the eastern dam embankment were likely cost prohibitive. Issues "b" and "c" have the potential to require such a downstream berm or complete rebuild, and issue "a" has the potential to require similar scale works on the western side of the reservoir.

Issue "d" is included in this report because it is relevant to TDC's decisions on timing of remedial works. We note also that issue "c" and "d" together cover most of the steps in the potential failure mode in Figure 0.1, which is the core focus of the remedial design. Issue "d", specifically, relates to those steps in the potential failure mode that are known to have occurred and are visibly continuing to deteriorate.

In line with our agreed brief, T+T's advice on these four selected issues comprises:

[1] "Concept design and preliminary cost estimates for remediation options" (Ref 1020688.4100, T+T July 2023).

- Assessment of the dam safety risk related to each issue, and
- The likely scale and cost of physical works to mitigate the dam safety risk of issues “b” and “c” specifically. Cost estimates for issues “a” and “d” are excluded from the agreed scope.

This advice is based on review of monitoring data, geotechnical interpretation, geotechnical analysis, concept design, and preliminary cost estimates as agreed with TDC for each issue. This report does not comprise a full interpretative report or completion of the detailed design sufficient to enable remedial works to proceed. Our previous 25% design report should be referred to for a more holistic view of the design to address the potential failure mode, whereas this current report focusses on specific points agreed with TDC as important to their decisions.

Key findings

The key findings of this report are:

- Issue “d” and the associated potential failure mode (Figure 0.1) represent the greatest risk for dam safety. As such, repairing the liner, subgrade, and subsoil network should remain the central and urgent focus of the remedial works. It is considered possible that the existing situation could deteriorate rapidly and require emergency intervention at any time. This would likely require dewatering and take the reservoir out of service for water supply to Dannevirke. The longer the repair of the liner, subgrade, and subsoil network is deferred, the greater the risk that an emergency could arise that could affect water supply to Dannevirke. Therefore, we recommend preparing to remove the risk as soon as practicable either by repairing the liner, subgrade, and subsoil network or decommissioning the dam.
- The risk associated with issue “c” is largely alleviated under most loading conditions by repairing the liner, subgrade, and subsoil network, which will prevent high water pressures reaching possible (but unavoidably unconfirmed) defects through the eastern dam embankment.
- The non-compliances associated with stability performance for issues “a” and “b” represent risk that is higher than recommended industry practice but well below thresholds that represent an emergency (unlike issue “d” and the associated potential failure mode described above).
- We recommend that installation of an upstream filter blanket (to mitigate internal erosion risk) is assessed further during detailed design of the liner, subgrade, and subsoil network repair. This is because the upstream filter blanket would be installed under the repaired liner.
- Based on the level of dam safety risk, development of other remedial options to address the non-compliances for issues “a”, “b”, and the remaining risks for issue “c” (that are not addressed by the liner, subgrade, and subsoil network repair) could reasonably be deferred and undertaken as part of routine asset management and renewal processes.

Further detail is presented in Table 0.1 and in the following paragraphs.

Stability of the western reservoir rim and eastern dam embankment (key issues “a” and “b”)

Performance has been assessed against recommended industry practice, which has generally been based on the guidance for a Medium Potential Impact Classification (PIC) dam in NZSOLD’s New Zealand Dam Safety Guidelines (NZDSG).

We have assessed performance against the following base cases:

1. Long-term static stability – a minimum Factor of Safety (FoS) of 1.5 has been adopted as recommended in Table 6.3 of Module 3 of the NZDSG.

2 Operating Basis Earthquake (OBE) – the recommended performance is that the dam and appurtenant structures should remain operational, and any damage should be no more than minor and readily repairable. A minimum FoS of "generally 1.0" is recommended per Table 6.4 of Module 3 of the NZDSG. However, in some cases a non-compliance where the FoS < 1, only results in very small deformations (< 20 mm) and arguably could be considered "no more than minor" depending on the impact on functionality and ease of repair.

3 Safety Evaluation Earthquake (SEE) - the SEE is a more severe earthquake than the OBE. Some damage may occur in the SEE but the recommended performance is that the SEE should not lead to an uncontrolled release (meaning a dam failure). Compliance has been interpreted as the estimated deformations being less than the freeboard at normal reservoir levels (1.26 m). While performance may be assessed as compliant because the dam has not failed, it is likely that the liner will have failed. *Following the SEE, the reservoir will likely need to be dewatered for inspection and repair, and as such, will be out of service for water supply to Dannevirke.*

4 Post-earthquake – a minimum FoS of 1.2 to 1.3 is recommended per Table 6.4 of Module 3 of the NZDSG. The lower end of this range, 1.2, has been adopted based on our assessment of the level of conservatism of input parameters.

The four base cases above are considered the current design scenarios. We have also assessed performance against sensitivity cases, which should be treated as possible but unconfirmed. The sensitivity assessment provides information on the risk of change in compliance due to known areas of uncertainty. Sensitivity cases considered include lower bound soil strengths, increased seismic hazard in the OBE and SEE based on the revised National Seismic Hazard Model (NSHM 2022), and the possible presence of a softened surface below the western reservoir rim.

The internal slopes of the reservoir (on both the western and eastern sides) have been assessed as *generally compliant* with recommended industry practice. There is a marginal non-compliance in the OBE for one of the sensitivity cases (<20 mm permanent displacement estimated), which as a sensitivity case is considered possible but not confirmed.

The external slope of the western reservoir rim, which includes the access track to the reservoir and treatment plant, has a *marginal* non-compliance in the OBE (<20 mm permanent displacement estimated). This is a location where it could be argued that the deformations are "no more than minor damage". However, the FoS is less than 1 so compliance is arguable. Options to remove the marginal non-compliance could include installation of shear piles, drainage, and / or a stabilisation berm. These options are not unequivocally needed and pricing for them is excluded from the scope of the current report.

The external slope of the eastern dam embankment has a *moderate* non-compliance for the long-term stability, post-earthquake, and OBE cases (25 mm permanent displacement estimated for the OBE). There is also a non-compliance in the SEE for a sensitivity case. A preliminary design of a drainage and stability berm has been developed, which would remove the non-compliance in the long-term stability and post-earthquake cases. An approximate "middle" cost estimate for the preliminary berm design is $ 2.8 M to $ 3.2 M, which adds approximately 35% to the overall remedial works costs (see Appendix H for detail). A larger berm, deep shear key or shear piles, would be required to remove the non-compliance in the OBE base case and the possible non-compliance in the SEE for the sensitivity case.

All the non-compliances described above are well below thresholds for immediate danger or that would make the dam "dangerous", "earthquake-prone", or "flood-prone" under the Building Act 2004.

Based on dam safety and engineering considerations, we recommend that the non-compliances are addressed as part of routine asset management and renewal processes separate from the urgent remedial works to the liner, subgrade, and subsoil network because:

- The non-compliances do not relate directly to the higher risk potential failure mode shown in Figure 0.1 that requires urgent remedial work if the dam is to remain in service.
- The non-compliances for stability performance do not represent a level of risk necessitating emergency intervention.
- Addressing these stability non-compliances will be required under activities under the Building (Dam Safety) Regulations 2022. However, this will likely comprise developing and implementing a defensible plan to investigate, confirm, and resolve the non-compliances in a timeframe that reflects the level of risk of the non-compliances i.e., allowing for long-term planning to set aside budget for the works. These investigations may potentially determine that physical works are not necessary.
- If physical works are confirmed as necessary to resolve the non-compliances, this would most likely be for the external slopes. These works could likely be constructed without dewatering the reservoir i.e., would not necessarily benefit from being undertaken while the reservoir was already dewatered for the urgent remedial works to the liner, subgrade and subsoil network.

Internal erosion of the eastern dam embankment, foundation, and backfill around subsoil outlet pipe (key issue “c”)

Key issue “c” relates directly to the Confirmed Dam Safety Deficiency and the potential failure mode shown in Figure 0.1.

Four types of internal erosion are typically considered for dam performance; contact erosion, backward erosion, suffusion, and concentrated leak erosion.

Contact erosion is relevant at interfaces between the subsoil drainage bedding and surrounding finer grained materials, including the Low Permeability Fill (LPF) liner and natural ground, as per Step 3 in the potential failure mode in Figure 0.1. There is evidence contact erosion is progressing based on the depressions and ongoing movement observed in the reservoir floor in inspections by remotely operated vehicles (ROV). The lack of filter compatibility between the existing subsoil bedding and surrounding materials, which enables contact erosion, is a Confirmed Dam Safety Deficiency.

Following the remedial works to the liner, subgrade and subsoil network, this Confirmed Dam Safety Deficiency will be eliminated by specifying filter compatible materials in line with modern, recommended practice. Following remediation, the risk of contact erosion will therefore be reduced to very low.

Backward erosion and suffusion through the eastern dam embankment is potentially relevant to Steps 4, 5, and 6 in the potential failure mode shown in Figure 0.1. However, these internal erosion types have been assessed as low risk based on analysis of available information on materials.

Concentrated leak erosion is relevant to Steps 4, 5, and 6 in the potential failure mode shown in Figure 0.1. There have been no direct observations that concentrated leak erosion is occurring. However, initiation of concentrated leak erosion is considered possible in the existing situation based on:

- Analysis of available information on materials.
- Construction details.
- Evidence the subsoil drainage network is compromised.
- High risk of further HDPE liner holes and tears.

Following remediation, the risk of concentrated leak erosion will reduce to low under normal loading conditions because the repaired liner and subsoil systems will prevent high hydraulic pressures from reaching possible (unavoidably unconfirmed) in situ cracks and internal erosion pipes in the eastern dam embankment. Nevertheless, if in situ cracks and internal erosion pipes are confirmed as present in the eastern dam embankment, this could arguably be seen as a vulnerability and does increase the criticality of the repaired liner and subsoil systems functioning as designed. The presence of in situ cracks and internal erosion pipes can only be confirmed definitively by direct inspection, which is not possible without dewatering and removing the existing liner system, even then the presence of such defects within the body of the embankment / foundation may remain hidden.

Internal erosion of the eastern dam embankment through cracks induced by a large earthquake

Following remediation, in very large earthquakes such as the SEE, failure of the liner system is expected and transverse cracks through the western reservoir rim and eastern dam embankment could potentially develop. In this situation, concentrated leak erosion through the transverse cracks is currently considered possible, subject to confirmation by analysis during detailed design of the remedial works.

This introduces a risk of non-compliance with the performance requirement that the SEE should not lead to an uncontrolled release. Although this vulnerability involves internal erosion through the eastern dam embankment, this is a different potential failure mode than shown in Figure 0.1 and is considered substantially lower risk. The exposure to the risk would only persist over the three to four days it would take to fully dewater the reservoir through the subsoil network following major liner damage in an SEE (dewatering time estimated based on hydraulic capacity of the subsoil network and assuming the liner system does not limit flows into the network). The possible non-compliance related to concentrated leak erosion through transverse cracks in the SEE does not represent an immediate danger or make the dam “dangerous” or “earthquake-prone” under the Building Act 2004.

Options to address internal erosion risk through the eastern dam embankment and foundation

Consideration has been given to possible options to mitigate the concentrated leak erosion risk associated with in situ cracks / internal erosion pipes through the eastern dam embankment (key issue “c”) or transverse cracks caused by an SEE (new lower risk issue described above). The most promising options identified are:

- An upstream filter blanket – an approximate “middle” cost estimate is $ 0.7 M to $ 0.8 M, which adds approximately 8% to the overall remedial works cost (refer Appendix H for detail). The upstream filter blanket is located under the liner system so would need to be constructed at the same time as the liner, subgrade, and subsoil network repairs. The upstream filter blanket is intended to mitigate the risk of concentrated leak erosion through both the transverse cracks generated by an SEE and through the possible (unconfirmed) in situ cracks / internal erosion pipes through the eastern dam embankment.
- A filtered berm – the drainage and stability berm described above to improve stability of the eastern dam embankment could also served as a filtered berm to mitigate the risk of internal erosion. As already noted above, an approximate “middle” cost estimate is $ 2.8 M to $ 3.2 M, which adds approximately 35% to the overall remedial works costs (see Appendix H for detail).
- A downstream filter diaphragm and berm.

The last two options above are intended to mitigate the risk of concentrated leak erosion through the possible (unconfirmed) in situ cracks / internal erosion pipes through the eastern dam embankment (key issue “c”). The pathway that seepage and eroded particles may track through /

from these defects is uncertain, especially downstream of the 2050 mm manhole. As such, there is a risk that the filtered berm and downstream filter diaphragm / berm options may not be effective if seepage and eroded particles bypass the devices described above.

Our recommendations based on dam safety and engineering considerations are:

- That the upstream filter blanket is assessed further during the detailed design of the liner, subgrade, and subsoil network repairs, and potentially constructed at the same time. The opportunity to install the upstream filter blanket will be lost if not constructed when the liner is repaired.
- Those other remedial options, such as the filtered berm and downstream filter diaphragm and berm, are deferred and considered as part of routine asset management and renewal processes separately from the liner, subgrade and subsoil network repairs because:
  - The non-compliances / vulnerabilities these options address do not represent a level of risk necessitating emergency intervention once the liner and subsoil system is repaired, which will prevent high hydraulic pressures from reaching the possible unconfirmed defects.
  - The options can be constructed without dewatering i.e., would not necessarily benefit from being undertaken while the reservoir was already dewatered for the urgent remedial works required to the liner, subgrade and subsoil drains.
  - Inspections and mapping of possible defects in the internal faces of the reservoir will be completed when the reservoir is dewatered to repair the liner, subgrade, and subsoil drains, which will provide information relevant to the design of the remedial options.

Addressing the non-compliances and vulnerabilities related to internal erosion will be required under activities under the Building (Dam Safety) Regulations 2022. However, like for the stability non-compliances discussed above, this will likely comprise developing and implementing a defensible plan to investigate, confirm, and resolve the issues in a timeframe that reflects the level of risk i.e., allowing for long-term planning to set aside budget for the works. These further investigations may find that physical works to eliminate the non-compliance / vulnerability are not needed, or are impracticable, or disproportionate with the risk reduction i.e., mitigation measures like surveillance and emergency preparedness may prove preferable to physical works.

Urgency of remediating the liner, subgrade, and underlying subsoil network (key issue “d”)

Contact erosion associated with Step 3 of the potential failure mode in Figure 0.1 is known to be occurring. Therefore, the current safety of the dam depends on further holes and tears in the HDPE liner not developing (Steps 1 and 2) and concentrated leak erosion through the eastern dam embankment and foundation not occurring (Steps 4 to 6).

The risk of new leaks is considered high (Steps 1 and 2), either due to failure of the temporary liner patches installed in June 2023 or failure of the original HDPE liner due to the ongoing deterioration of the supporting subgrade. The risk of concentrated leak erosion (Steps 4 to 6) is also considered possible in the existing situation as described above.

On this basis, it is considered possible that the existing situation could deteriorate rapidly and require emergency intervention at any time. It is noted that a dam safety emergency would likely require dewatering and take the reservoir out of service for water supply to Dannevirke.

Our recommendations based on dam safety and engineering considerations are:

- Continue with current measures to mitigate the risk of the potential failure mode in Figure 0.1, including:

  - Ongoing enhanced surveillance; and
  - Maintaining preparedness to implement TDC's interim emergency action plan.
- Prepare to remove the risk *as soon as practicable either* by repairing the liner, subgrade and subsoil drains or decommissioning the dam.

Next steps

We understand that our advice, outlined in this report, will be considered by TDC as one input amongst a wider suite of considerations in key decisions for the project and community.

We anticipate that these key decisions will include:

- What issues should be addressed by the urgent remedial works?

  We have recommended:
  - That these works should include remediating the liner system, subgrade, and subsoil network as a minimum.
  - That works to address non-compliances with recommended industry practice for the stability and internal erosion performance of the western reservoir rim and eastern dam embankment should be undertaken separately as part of routine asset management and renewal cycles.
  - However, installing the upstream filter sand blanket at the same time as repairing the liner, subgrade, and subsoil network should be considered further during detailed design. This exception is recommended because the upstream filter sand blanket would be installed under the liner so the opportunity to install the blanket will be lost if not installed when the liner is repaired.
- When to carry out the urgent remedial works or decommission the dam.

  Based on current information and analysis, the risk related to the potential failure mode (shown in Figure 0.1) is not an immediate danger but could deteriorate rapidly and require emergency intervention at any time, which would likely take the reservoir out of service for water supply. The longer the damage and confirmed deficiencies related to the potential failure mode remain, the greater the risk that an emergency could arise that could affect water supply to Dannevirke. Our recommendation is therefore that TDC should prepare to remove the risk as soon as practicable either by repairing the liner, subgrade and subsoil drains or decommissioning the dam.
- Whether to proceed with the urgent remedial works to the dam, or alternatively decommission the dam and pursue alternative water supply options.

  In addition to considering the technical and economic viability of alternative water supply options, TDC should be aware that decommissioning will require a Building Consent as a "Large dam" and could potentially involve significant physical works such as redirecting inlet and outlet pipes, and modifying the reservoir to provide certainty that local rainfall cannot still build up to form a pond.

  Developing, consenting, and constructing entirely new, alternative water supply options is typically a multi-year process. As noted above, there is a significant risk that the current situation could deteriorate rapidly at any time and take the reservoir out of service, and this risk worsens the longer the repair of the liner, subgrade and subsoil drains is deferred. We recommend that the timeline for developing alternative water supply options and implications for dam safety and water supply risk should be considered by TDC as part of decision-making.

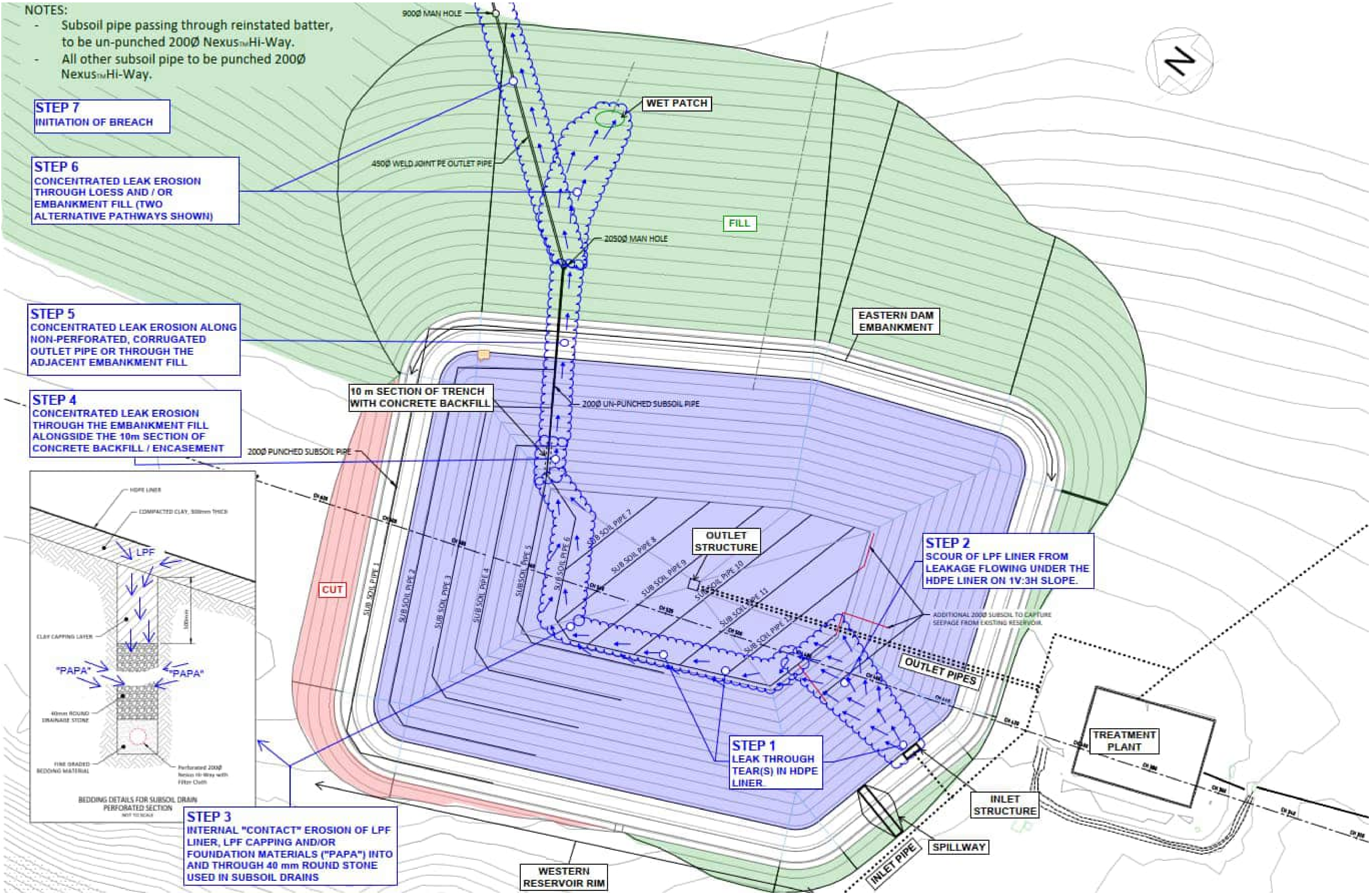

Figure 0.1: Steps in postulated failure mode for subject dam safety issue

Table 0.1: Summary of assessment

Key issue		Dam safety risk		Mitigation options	Recommendations based on dam safety and engineering considerations[1]
		Assessed risk	Comment		
Issue "a" Stability of the western reservoir rim.	External slope	Marginal with respect to recommended industry practice. The non-compliances are well below thresholds for immediate danger or that would make the dam "dangerous", "earthquake-prone", or "flood prone" under the Building Act 2004.	• Long-term static stability is marginally compliant. There is a risk this could change to non-compliant with only slight changes in the analysis. • Performance in the OBE is marginally non-compliant for the base case. The implication of non-compliance is that "more than minor" damage may occur in a smaller earthquake than recommended for a Medium PIC dam. Any permanent deformation in the OBE is expected to be small (<20 mm) and shallow. These displacements in the OBE would most likely affect the external face and access track into the dam but are not expected to cross the crest of the reservoir rim to damage the reservoir liner. • Performance in the OBE worsens for sensitivity cases, involving displacements of up to 60 mm, with deformation potentially extending across the crest of the reservoir rim and possibly damaging the liner.	Installation of shear piles, drainage, and/or a stabilisation berm could be considered but is not unequivocally needed.	Address non-compliances as part of routine asset management and renewal processes separate from the urgent repair of the liner, subgrade, and subsoil network because: • The non-compliances do not relate directly to the potential failure mode shown in Figure 0.1 that requires urgent remedial work if the dam is to remain in service. • Non-compliances are minor / not confirmed, and the level of risk does not require emergency intervention. • Addressing these possible non-compliances will be required under activities under the Building (Dam Safety) Regulations 2022. However, this will likely comprise developing and implementing a defensible plan to investigate and resolve the non-compliances in a timeframe that reflects the minor nature of the non-compliances. • If intervention is confirmed as necessary, this would most likely be for the external slope, and would likely not require dewatering. This removes the driver to do the intervention while the reservoir is already dewatered for the urgent remedial works required to the liner, subgrade and subsoil drains.
	Internal slope	Generally acceptable with respect to recommended industry practice.	• Performance is compliant for all base design cases. • However, one of the sensitivity cases has a marginal non-compliance in the OBE with small displacements predicted (<20 mm). Non-compliance with OBE criteria is less likely for the internal slope compared with the external slope, but the consequences may possibly include damage to the liner system and require dewatering to repair.	Physical interventions are not currently indicated as needed, but if needed would likely be challenging. Non-structural measures may be preferred i.e., surveillance, emergency preparedness, or other measures to mitigate the risk to water supply.	
Issue "b" Stability of the eastern dam embankment.	External slope	Non-compliant with respect to recommended industry practice. The non-compliances are below thresholds for immediate danger or that would make the dam "dangerous", "earthquake-prone", or "flood prone" under the Building Act 2004.	• Long-term static stability, post-earthquake, and OBE cases are non-compliant. • Performance in the SEE is non-compliant for one of the sensitivity cases. A preliminary design and cost estimate has been developed for a drainage and stabilisation berm. The berm would eliminate the non-compliance for long-term stability and post-earthquake cases but would need to be larger, and/or a deep shear key or shear piles added, to eliminate the non-compliance in the OBE and the possible non-compliance in the SEE for the sensitivity case. The implication of the non-compliance in the OBE is that "more than minor" damage may occur in a smaller earthquake than recommended for a Medium PIC dam. Permanent displacements of 25 mm are predicted in the OBE. Modelling indicates that these displacements may extend across the dam crest to possibly damage the liner. The ground movement may be relatively deep seated i.e., may extend relatively deeply into the dam embankment, which could be more difficult to repair and raise concerns for embankment integrity. The performance in the OBE is worsened for a sensitivity case with displacements increasing to 70 mm.	Construction of a drainage and stabilisation berm. Preliminary cost estimate \$2.8M to \$3.2M, which adds 35% to overall remedial works cost (see Appendix H for detail).	These non-compliances could be addressed as part of routine asset management and renewal processes separate from the urgent remedial works. The rationale is largely the same as above for the western reservoir rim. In addition, the possible non-compliance for the SEE for a sensitivity case may more readily be confirmed when updates to the NZDSG are published later this year, which are expected to cover interpretation of NHSM 2022 relevant to this case.
	Internal slope	Generally acceptable with respect to recommended industry practice.	• Performance is compliant for all base design cases. • However, one of the sensitivity cases has a marginal non-compliance in the OBE with small displacements predicted (<20 mm). Non-compliance with OBE criteria is less likely for the internal slope compared with the external slope, but the consequences may possibly include damage to the liner system and require dewatering to repair.	Physical interventions are not currently indicated as needed, but if needed would likely be challenging. Non-structural measures may be preferred i.e., surveillance, emergency preparedness, or other measures to mitigate the risk to water supply.	

Key issue		Dam safety risk		Mitigation options	Recommendations based on dam safety and engineering considerations[1]
		Assessed risk	Comment		
Issue “c” Potential for internal erosion of the eastern dam embankment, including dam foundation, and backfill around the subsoil outlet pipe.	Contact erosion	Existing situation: Known to be occurring. Confirmed Dam Safety Deficiency.	Depressions observed in the reservoir indicate contact erosion has already initiated. Ongoing movement of the reservoir floor observed in ROV inspections indicates contact erosion is still progressing.	Remove the deficient subsoil drainage bedding and replace with filter compatible material.	The urgent remedial works should include as a minimum: • Replacement of subsoil drainage bedding with filter compatible material. • Reinstatement of the subgrade to support the new liner system. • Replacement of the existing HDPE and LPF liner with a new liner system. We suggest that further remedial options to address any remaining internal erosion vulnerabilities and non-compliances (those not already addressed by the liner, subgrade, and subsoil network repair) are developed as part of routine asset management and renewal processes separate from the urgent remedial works. The rationale is largely the same as above for the western reservoir rim. However, we note that further investigations may find that surveillance and emergency preparedness are a more pragmatic way to manage the risk of the deficiency than physical interventions. However, one exception is that we recommend the upstream filter blanket is assessed further during detailed design of the liner, subgrade, and subsoil network repair, and potentially constructed at the same time. This is because the upstream filter blanket is positioned under the liner, so the opportunity to construct the blanket would be lost if not constructed when the liner is repaired.
		Following remediation: Very low risk by design. Deficiency eliminated.	Filter compatible materials will be specified based on modern recommended practice to reduce the risk to acceptable levels.		
	Backward erosion	Existing situation: Very low risk Following remediation: Very low risk	Based on analysis of available information on materials.	Not required.	
	Suffusion	Existing situation: Low risk Following remediation: Very low risk	Based on analysis of available information on materials.	Not required.	
	Concentrated leak erosion	Existing situation: Possible risk.	There have been no direct observations that concentrated leak erosion is occurring. However, the risk is considered moderately likely based on: • Available information on materials. • Construction details. • Evidence the subsoil drainage network is substantially damaged (depressions and ongoing movement observed by ROV inspections). • High risk of further HDPE liner holes and tears.	Remediate the liner and subsoil network to prevent elevated hydraulic pressures that could initiate / worsen concentrated leak erosion. Further options that could be considered include an upstream filter blanket ($0.7M to $0.8M), a filtered berm, and a downstream filter diaphragm / berm.	
		Following remediation: Low risk during normal operation.	Low risk during normal operation assuming the repaired liner and subsoil network operating as designed i.e., assuming these repaired systems will prevent hydraulic gradients that could drive concentrated leak erosion from reaching possible (unconfirmed) defects through the eastern dam embankment.		
New issue Potential for internal erosion through cracks induced by a large earthquake	Concentrated leak erosion	Existing situation and following remediation: Possible risk in very large earthquakes, such as the SEE. Below the threshold for immediate danger or that would make the dam “dangerous” or “earthquake-prone” under the Building Act 2004.	Possible risk in a very large earthquake, such as the SEE, that causes liner damage and transverse embankment cracks. In this situation, concentrated leak erosion could potentially lead to an uncontrolled release i.e., introduce a non-compliance with SEE criteria. However, exposure to this risk would only occur in very large, extreme events and would persist for the three to four days required to dewater the reservoir via the subsoil network. Note that the subsoil network is proposed to be designed to remain functional in the SEE.	An upstream filter blanket ($0.7M to $0.8M).	As noted above, further assessment of the upstream filter blanket is recommended during the detailed design of the liner, subgrade, and subsoil network repair.
Issued “d” Urgency of remediating the existing liner, subgrade, and underlying subsoil network.		High risk. The risk in the current situation is not considered to be an “immediate danger” but it is considered possible that the situation could deteriorate rapidly at any point such that the dam becomes “dangerous” as defined under the Building Act 2004 and requires emergency intervention. The risk worsens the longer repair of the liner, subgrade, and subsoil network is deferred.	ROV inspections indicate that deterioration of the LPF liner and subgrade is continuing to progress even with the reduction in subsoil flows since the June 2023 temporary repairs. The current safety of the dam depends on new leaks not developing and concentrated leak erosion through the eastern dam embankment and foundation not occurring. The risk of new leaks is considered high either due to failure of the temporary liner patches installed in June 2023 or failure of the original HDPE liner due to the ongoing deterioration of the supporting subgrade. The risk of concentrated leak erosion is also considered credible in the existing situation as described above. The longer the damage and confirmed deficiencies related to the potential failure mode shown in Figure 0.1 remain, the greater the risk that an emergency could arise that could affect water supply to Dannevirke.	Enhanced surveillance. Interim emergency action plan. Prepare to remove risk as soon as practicable.	Continue with current mitigation measures including ongoing enhanced surveillance and maintaining preparedness to implement TDC’s interim emergency action plan for the dam. Prepare to remove the risk as soon as practicable either by repairing the liner, subgrade, and subsoil network or decommissioning the dam.

1. T+T’s recommendations are limited to advice based on dam safety and engineering considerations in our area of expertise. We recognise that TDC’s decisions to proceed or not with our recommendations will be based on a broader suite of considerations.

1 Introduction

1.1 Scope

This report presents work undertaken by Tonkin + Taylor (T+T) for Tararua District Council (TDC) as our client in accordance with an approved Work Package Plan, dated 7 June 2024. The scope was developed to fulfil the requirements expressed by TDC at a meeting on 5 April 2024 and was discussed and revised further at a meeting with TDC and its peer reviewer on 31 May 2024.

At a high level, the scope of this work package comprises development and provision of T+T's professional advice on four key issues related to dam performance:

a Stability of the western reservoir rim – refer Section 3.

b Stability of the eastern dam embankment – refer Section 4.

c Potential for internal erosion of the eastern dam embankment, including dam foundation, and backfill around the subsoil outlet pipe – refer Section 5.1.

d Urgency of remediating the existing liner, subgrade, and underlying subsoil network – refer Section 6.

Section 2 of this report presents updated geotechnical interpretation of Stage 2 investigation data, which is an input to the assessment of the issues above.

Our advice comments on the dam safety risk of the four issues above and describes the likely scale and cost of physical works to mitigate the dam safety risk of issues b and c above. This advice is based on review of monitoring data, geotechnical interpretation, geotechnical analysis, concept design, and preliminary cost estimates as appropriate to each issue.

In line with our agreed brief, this report does not comprise a full interpretative report and detailed design to enable remedial works to proceed. Instead, this report presents targeted interpretation, analysis, and design specific to the four key issues above, to input to TDC's decisions on whether to proceed to the next stage, which would comprise the more detailed analysis.

1.2 Purpose

We understand that our advice, outlined above, will be considered by TDC as one input amongst a wider suite of considerations in key decisions for the project and community. Table 1-1 following summarises our understanding of the materiality of our advice to TDC's key decisions for the project and community.

Table 1-1: Materiality of T+T's professional advice to support TDC's decision-making

TDC's project decisions	Relevance of T+T's advice
1 Whether to proceed with remediating the dam, or instead decommission the dam and develop alternative water supply options.	Likelihood and consequences of 'deficiencies' relating to each issue, i.e. risk. Indicative remedial options and costs associated with each issue including monitoring approaches where appropriate. This will assist TDC to identify which issues are a priority and which can be deferred / monitored / managed within a risk profile acceptable to TDC.
If proceeding, decide:	
2 Which issues to address in the current works and which to address separately as part of a routine asset renewal programme.	
3 When to proceed with the remedial works.	Urgency of remedial works for each item.

1.3 Peer review and report version

This report was previously issued as a Version 1 on 21 July 2024 for comment by TDC and the Peer Reviewer, Damwatch Ltd. TDC and Peer Reviewer comments on Version 1 were provided by email (TDC (P Morris) to T+T on 5 August 2024) and at an online teleconference attended by TDC, the Peer Reviewer, and T+T on 5 August 2024. The review comments and T+T's response is summarised in Appendix I. The updates in this current version 2 of the report comprise minor changes in response to review comments and interpretation of Hole Erosion Test data.

2 Geotechnical interpretation of Stage 2 investigation data

2.1 Geology and geomorphology

2.1.1 Published geology

The published geological map of the area is presented as Figure 2-1. The map indicates that the dam is underlain by Late Pleistocene (Q4a) alluvial deposits, which forms an elevated river terrace about 40 m above, and 500 m to the east of the Tamaki River. Q4a deposits are described as weathered, well graded gravel and loess with minor sand and silt.

The western margin of the Q4a deposits is defined by a linear north-south orientated degradational terrace riser, which steps down in elevation to a younger Q2a terrace (and then steps down again to the Holocene (Q1a) river flood plain closest to the Tamaki River). The eastern margin of Q4a deposit is mapped as ‘approximate’ where it contacts Pliocene sandstone and siltstone of the Mangaheia Group (colloquially known as “papa”).

A fold axis of a north-south orientated anticline is mapped about 200 m to the east of the dam. The approximate position of an inactive fault is mapped about 500 m to the east of the dam. This fault is the northern continuation of the active Pahiatua Fault, which is mapped 1.5 km south of the dam.

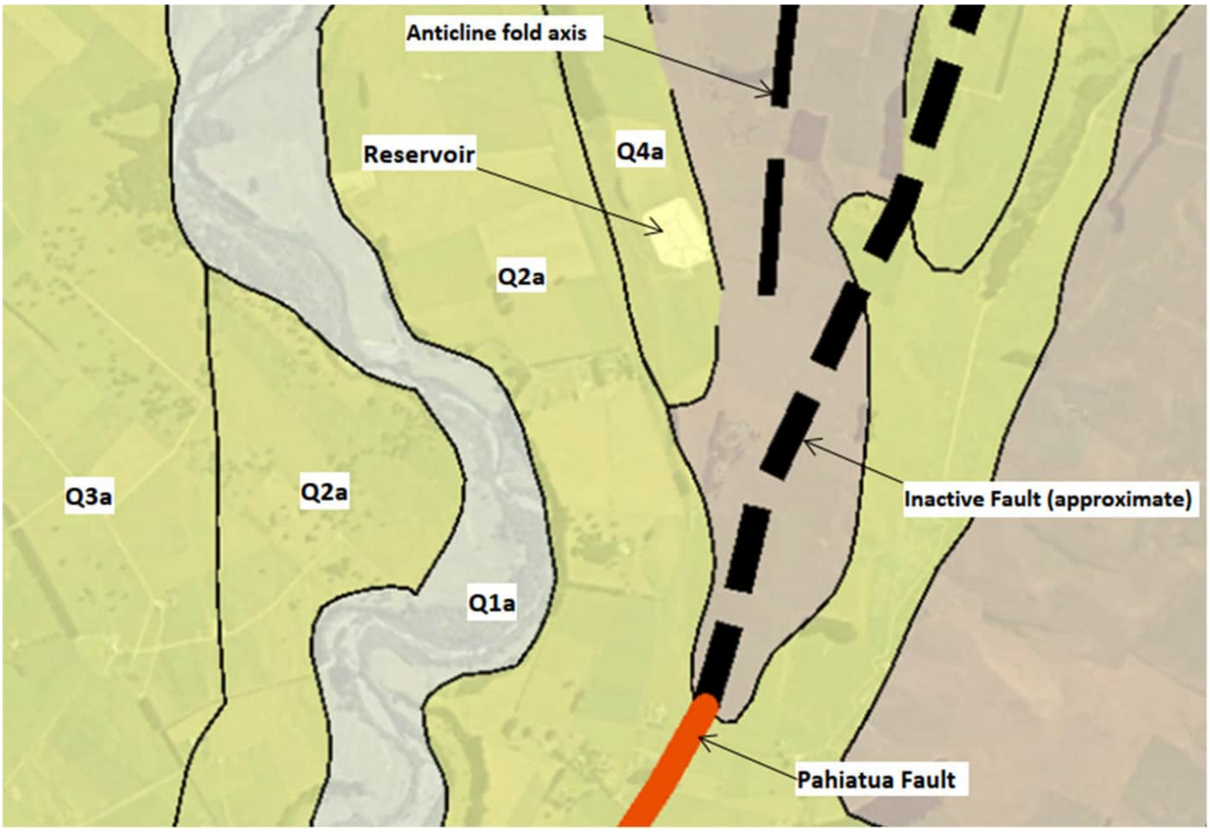

Figure 2-1: Published geological map showing the site in the context of the regional geology (“Geology of the Hawke’s Bay area” (Lee et al, 2011, Institute of Geological & Nuclear Sciences)).

2.1.2 Geological mapping

Geological mapping of the site was undertaken between 14 to 21 August 2023 by an engineering geologist from T+T as part of the Stage 2 geotechnical investigations. The geological map and mapping observations are presented on Figures A1 and A2 in Appendix A. The full descriptions of observations are included in a register in Appendix B of the “Stage 2 Geotechnical Factual Report” (T+T December 2023).

2.2 Geotechnical conditions

2.2.1 Geotechnical investigations from the original design and construction

Previous investigations undertaken by others in support of the original design of the reservoir comprised:

- 5 No. machine boreholes; and
- 5 No. machine excavated test pits.

The investigation locations and logs are presented in "Dannevirke Water Supply: No 1 Reservoir Upgrade Geotechnical Report" (L Wesley 2011, Auckland UniServices Limited).

2.2.2 Project specific geotechnical investigations

The first stage (Stage 1) of geotechnical investigations for the remedial works project was carried out on 3 April 2023 by an engineering geologist from T+T and a senior technician from Geotechnics Limited. The investigation consisted of 9 No. hand excavated test pits (TP01 to TP09) to obtain bulk samples for geotechnical laboratory testing. The findings from Stage 1 are presented in the "Geotechnical Factual Report" (T+T May 2023) and in the "Stage 1 Geotechnical Interpretative Report and Internal Erosion Assessment" (T+T September 2023).

A second stage (Stage 2) of geotechnical investigations was carried out between 16 and 23 August 2023 to inform detailed design. The Stage 2 geotechnical investigations comprised:

- 5 No. Cone Penetration Tests (CPT01 to CPT05).
- 2 No. machine (sonic) boreholes (BH01 and BH02) with Standard Penetration Testing (SPT) carried out at regular (typically 1.5 m) intervals. Falling head permeability tests were undertaken during drilling of BH01 within the encountered embankment fill and foundation soils.
- 8 No. machine excavated test pits (TP101 to TP108); and
- 1 No. hand auger (HA01) with hand-held (downhole) shear vane testing of fine grained (clay/silt) soils.

The CPTs were pushed to refusal (4.5 m to 24 m depth). CPT03-a and CPT04-a were undertaken following shallow refusal within the embankment fill during the first attempt of CPT03 and CPT04. The sonic boreholes were advanced to target depths of 27.8 m and 20.0 m, respectively. The hand auger and test pits were also advanced to target depths. An investigation location plan is presented on Figure A2 in Appendix A.

The findings from Stage 2 are presented in the "Stage 2 Geotechnical Factual Report" (T+T December 2023), the "Dannevirke Raw Water Reservoir Remediation Design Report" (V01 25% design stage, T+T September 2023), and in this report. The results from the SPT, CPT and shear vane tests are summarised in Table 2-8.

2.2.3 Laboratory testing

The Stage 1 and Stage 2 laboratory testing comprised Atterberg limits, Particle Size Distribution (wet sieve and hydrometer), standard compaction, natural water content, pinhole dispersion, hole erosion (HET), permeability, solid density and triaxial (consolidated undrained) testing of the encountered materials. As noted above, full laboratory test results are presented in the respective Stage 1 and Stage 2 Geotechnical Factual Reports (T+T May 2023, T+T December 2023).

The results of the completed laboratory tests are discussed below.

2.2.3.1 Particle size distribution

Particle size distribution tests including wet sieve and hydrometer were carried out on fourteen samples (six during Stage 1 and eight during Stage 2). The grading curves are presented graphically as Figure 2-2.

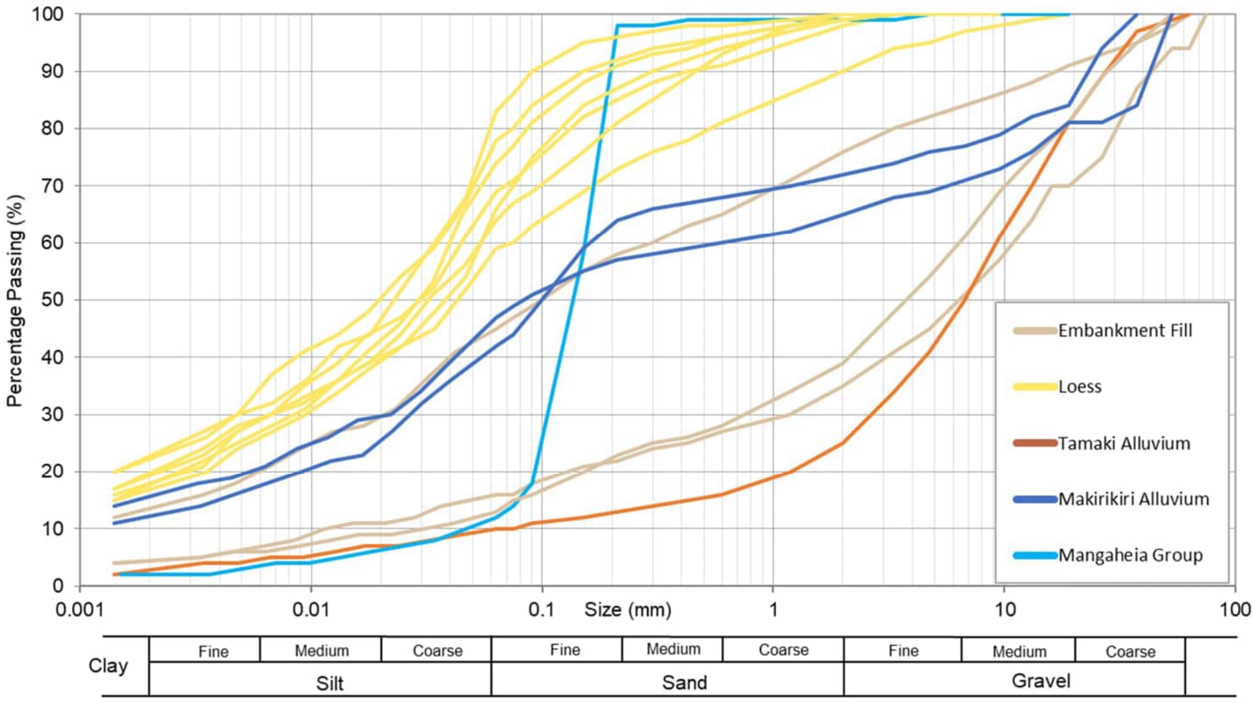

Figure 2-2: Particle size distribution curves for all samples.

2.2.3.2 Standard compaction and solid density

Standard compaction tests were carried out on ten samples including a combination of single point and five-point tests. The natural water content, dry density and bulk density was also determined based on triaxial Consolidated-Undrained (CU) and pinhole dispersion tests that were undertaken on undisturbed core samples, and the results are summarised in Table 2-1 (refer to the factual report for details).

A solid density test by way of vacuum (NZS 4402:1986 Test 2.7.2) was undertaken on a single sample of the Loess borrow material to inform the compaction testing. The average solid density recorded was 2.71 t/m^3.

Table 2-1: Standard compaction test results

Geology	Natural water content (%)	Bulk density (t/m^3)	Dry density (t/m^3)	Maximum dry density (t/m^3)	Optimum moisture content (%)
Embankment Fill	14 - 23	2.0 - 2.1	1.6 - 1.9	1.9	14
Loess (borrow)	22 - 25	1.7 - 2.0	1.2 - 1.6	1.6 - 1.8	18 - 23
Loess (foundation)	31 - 34	1.9	1.4	-	-
Makirikiri Alluvium	20 - 27	2.0	1.6	-	-
Tamaki Alluvium	13	2.1	1.8	-	-

2.2.3.3 Atterberg Limits

The Atterberg Limit test was undertaken on thirteen samples to determine the liquid limit, plastic limit and the plasticity index (noting that one sample of Tamaki Alluvium was considered non-plastic, and the test was not completed, and one sample of Loess borrow contained organics and has been discounted from the data set as considered not representative). The results are summarised in Table 2-2, and presented in Figure 2-3.

Table 2-2: Atterberg Limits test results

Geology	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural water content (%)
Embankment Fill	31-34	22	9-12	14 - 23
Loess (borrow)	33-40	21-24	12-16	22 - 25
Loess (foundation)	38-42	23-26	15-16	31 - 34
Makirikiri Alluvium	30-34	18-20	12-14	20 - 27
Tamaki Alluvium	46	24	22	13

**Note that prior to commencing each test, the material was passed through a 425-micron sieve.*

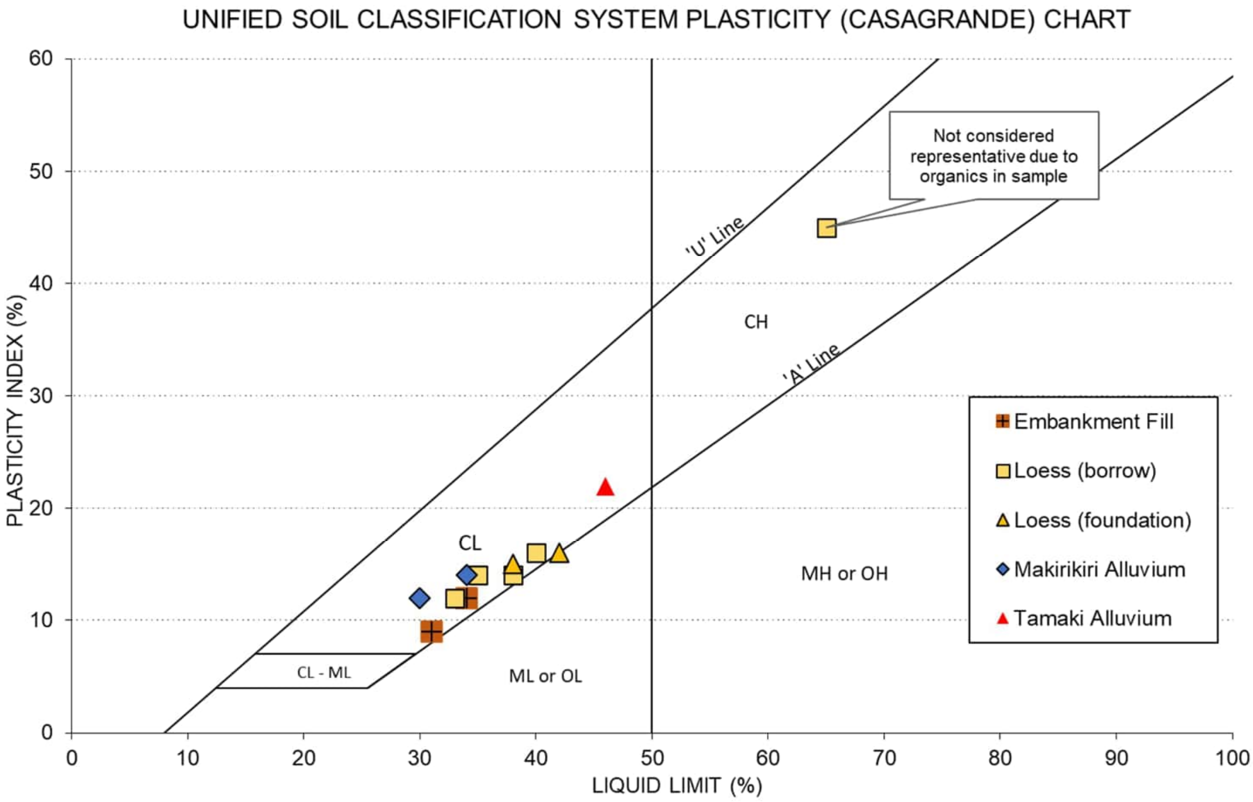

Figure 2-3: Casagrande chart presenting Atterberg Limit test results.

2.2.3.4 Triaxial (CU) tests

CU triaxial compression tests were undertaken for two undisturbed (push-tube) samples collected from BH01. The effective strength (Mohr-Coulomb) parameters have been derived based on a linear regression for each test and are presented in Table 2-3.

Table 2-3: Effective strength parameters derived from Triaxial (CU) tests

Geology	Sample depth (m)	Friction Angle (phi')	Cohesion (c')
Loess (foundation)	14.01-14.6	38	6
Makirikiri Alluvium	18.82-18.97	33	3

2.2.3.5 Constant head permeability

Two constant-head permeability tests were undertaken on remoulded samples of the Loess borrow material. The samples were remoulded to the optimum water content +3%, and 98% of the maximum dry density based on standard compaction testing and tested in a triaxial cell. The results are presented in Table 2-4.

Table 2-4: Laboratory permeability test results

Geology	Sample	Coefficient of permeability (m/s)
Loess (borrow)	TP105 (1.0m)	1.81×10^{-10}
Loess (borrow)	TP106 (0.5m)	4.65×10^{-10}

2.2.3.6 Pinhole dispersion

Pinhole dispersion tests were carried out on five remoulded and three undisturbed samples. The dispersion category for each test is presented in Table 2-5.

Table 2-5: Pinhole dispersion test results

Geology	Sample	Sample preparation	Dispersion Category
Embankment Fill	TP03 (0.35m)	Remoulded at NWC and 95% of MDD	ND1
Loess (borrow)	TP05 (0.35m)	Remoulded at NWC and 95% of MDD	ND1
Loess (borrow)	TP07 (0.35m)	Remoulded at NWC and 95% of MDD	ND1
Loess (borrow)	TP105 (0.8m)	Remoulded at OWC+3% and 98% of MDD	ND1
Loess (borrow)	TP106 (0.5m)	Remoulded to OWC+3% and 98% of MDD	ND1
Loess (foundation)	BH01 (13.5m)	Undisturbed core sample	ND2
Makirikiri Alluvium	BH01 (18.69-18.73m)	Undisturbed core sample	ND3
Makirikiri Alluvium	BH01 (19.7m)	Undisturbed core sample	D2

Dispersion category: D2, D1: Dispersive; ND4, ND3: Moderately to slightly dispersive; ND2, ND1: Non-dispersive.

2.2.3.7 Hole Erosion Test

Standard and critical head Hole Erosion Tests (HETs) were carried out on two bulk samples remoulded to the respective optimum water content and 98% of the maximum dry density based on standard compaction testing. The tests were undertaken on Embankment Fill (BH01 at 8.6-10.1m) and Loess borrow material (TP105 at 1.0m) following test procedures after Wan & Fell 2002, 2004a and 2004b. Interpretation of the HET testing is presented in Appendix E.

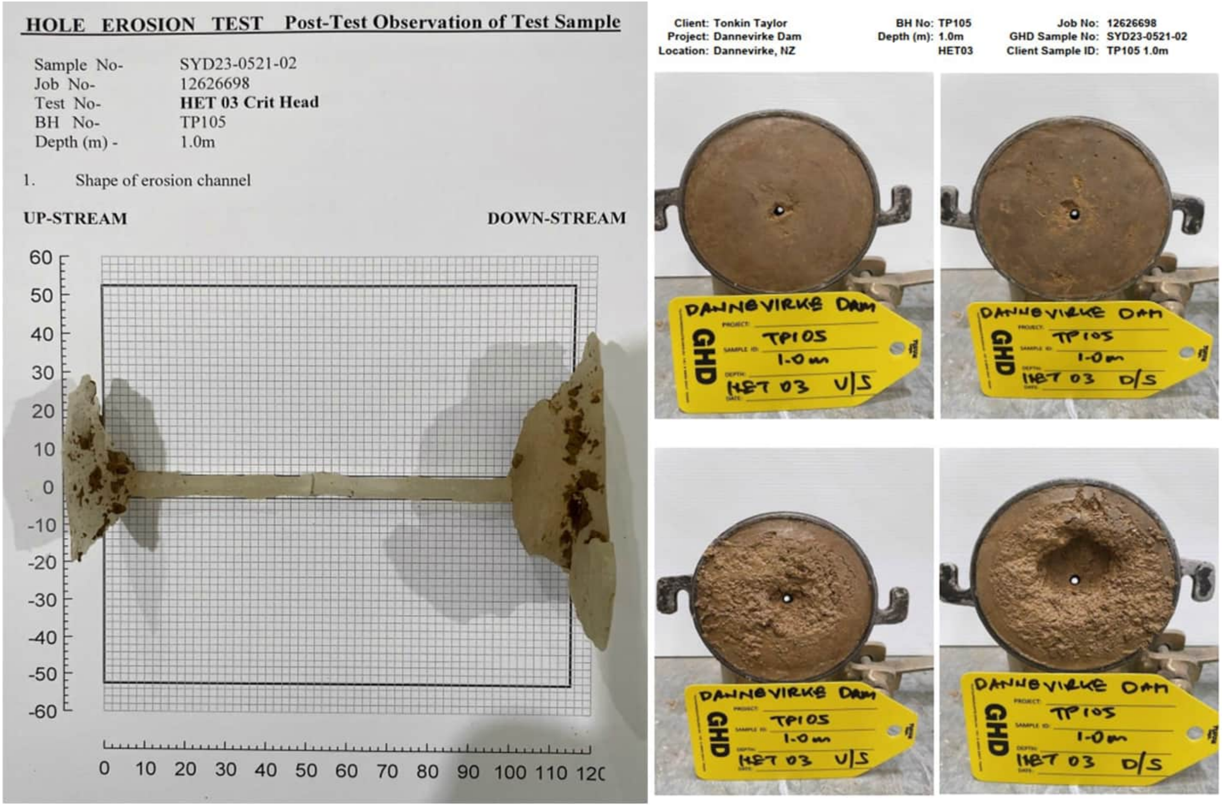

Figure 2-4: Example of post-test observations of a critical head HET on a remoulded sample from TP105 at 1.0m.

2.2.4 Groundwater monitoring

Following completion of the drilling, a multi-level (3 No.) vibrating wire piezometer (VWP) was installed within BH01 for long-term monitoring of pore water pressures within and below the eastern embankment. Similarly, a multi-level (2 No.) VWP was installed within BH02 for long-term monitoring of pore water pressures within and below the western reservoir rim. Additionally, a level logger was installed within a standpipe in HA01. The VWP and standpipe installation details are included in the "Stage 2 Geotechnical Factual Report" (T+T December 2023). The VWPs and level logger are connected to a Cirro telemetry system to enable remote monitoring of near real-time pore water pressure data, accessible via a web portal.

The data indicates groundwater levels are:

- Between 254.2 and 256.0 m RL under the crest of the eastern embankment (BH01 at 20.5m bgl).
- Between 249.2 and 250.9 m RL under the toe of the eastern embankment (HA01 at 2.1m bgl). The upper end of this range is close to the ground surface.
- Between 256.7 and 258.9 m RL under the western reservoir rim (BH02 at 19.0m bgl).

This is consistent with the elevation of groundwater seeps mapped on site. The available groundwater monitoring data is summarised in Table 2-6. The data is also plotted against rainfall, reservoir level and subsoil flow, and is included in Appendix B.

Table 2-6: Summary of groundwater monitoring data (to 12 July 2024)

ID	Ground Elevation (m RL)	Monitoring period	Tip (m bgl)	Tip (m RL)	Geology	Groundwater Level (m RL)		
						Max	Min	Average
BH01	272.44	25/08/2023 to 12/07/2024	11.5	260.94	Embankment Fill	262.07	dry	dry
			14.5	257.94	Loess	258.43	dry	dry
			20.5	251.94	Makirikiri Alluvium	255.92	254.27	255.02
BH02	272.02	23/08/2023 to 12/07/2024	12.0	260.02	Tamaki Alluvium	261.34	dry	dry
			19.0	253.02	Tamaki Alluvium	258.87	256.74	257.57
HA01	251.05	22/10/2023 to 12/07/2024	2.1	248.95	Makirikiri Alluvium	250.86	249.25	249.91

All elevations in terms of NZVD 2016.

2.2.5 Ground conditions

2.2.5.1 Geotechnical model

The inferred geotechnical model including the respective generalised Unified Soil Classifications (USC) are presented in Table 2-7. The inferred geological map and cross sections are presented as Figures A1 to A5 in Appendix A. Figure A3 is also presented as Figure 2-5.

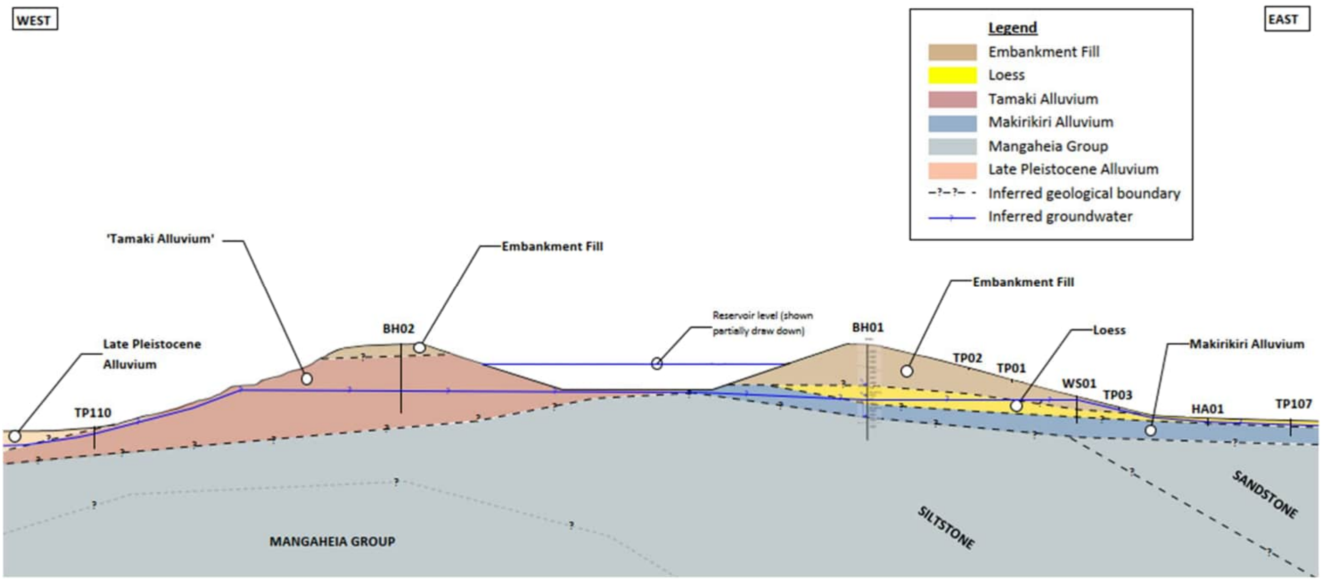

Figure 2-5: Geological long section (Figure A3) - not to scale.

Table 2-7: Ground model summary

Geological unit	Typical geotechnical description	Primary classification[1] (secondary materials potentially present)
Topsoil	Organic SILT minor sand	OL
Embankment Fill	Fine to coarse GRAVEL, some sand, minor silt, trace to minor clay, trace cobbles; medium dense to dense (inferred source Tamaki Alluvium)	GC/GM (SC/SM)

Geological unit	Typical geotechnical description	Primary classification[1] (secondary materials potentially present)
Loess	Clayey SILT, some sand; soft to firm; medium plasticity (cover deposit capping the Alluvium, Low Permeability Fill (LPF) liner source material[2])	CL (ML)
Makirikiri Alluvium	Silty, gravelly CLAY, minor sand; stiff to very stiff; medium to high plasticity (underlying the eastern embankment only)	GC/SC/CL
Late Pleistocene Alluvium	Sandy, fine to coarse GRAVEL, some sand, minor silt, trace cobbles; tightly packed	GW
Tamaki Alluvium	Fine to coarse GRAVEL, some sand, minor silt, trace cobbles; very dense (underlying the western embankment only)	GW
Mangaheia Group	Slightly to highly weathered sandy SILTSTONE / silty SANDSTONE, extremely weak to very weak (underlying the Alluvium)	Highly weathered or better Siltstone and Sandstone

1. USC code descriptions: G: Gravel, S: Sand, M: Silt, C: Clay, O: Organic, P: Poorly graded (many particles of approximately the same size), W: Well-graded (many different particle sizes), H: High liquid limit, L: Low liquid limit.
2. The existing reservoir is lined with high density polyethylene (HDPE) overlying a 300mm thick LPF liner. The LPF material is referred to in the original construction drawings as “compacted clay”. However, the material is understood to be sourced from the local Loess, which is typically a clayey SILT, rather than CLAY.

2.2.5.2 Groundwater model

Our interpretation of the groundwater conditions beneath the reservoir is presented on Figures A3 and A4 in Appendix A, and is based mapping of observable seepages and long-term monitoring of piezometers as presented in Table 2-6.

The regional groundwater level was not encountered during the investigations however is inferred to be close to the level of the Tamaki River and flood plain at roughly 235 mRL (some 25 m below the reservoir invert).

A higher groundwater level between 254 and 262 mRL is present within the elevated river terrace to the east of the Tamaki River upon which the reservoir has been constructed. This level is close to the reservoir invert and has been observed at the ground surface as seepage and measured in the piezometers installed into BH01 and BH02. Evidence of seepage (e.g. hydrophilic vegetation and soil slumping) has been observed on western and eastern flanks of the terrace, notably a flowing seepage was observed at mapping waypoint WP03 located 300 m to the north of the reservoir. The piezometric data indicates the groundwater level is sensitive to rainfall so is likely to be recharged though downward percolation of surface water though the terrace forming Tamaki Alluvium, and likely drains towards the south-east from a region of elevated ground immediately to the north of the reservoir. Photographs taken during the original construction also suggest that groundwater consistent with this higher level was exposed at the base of the excavation.

2.2.6 Geotechnical design parameters

The geotechnical design parameters presented in Table 2-8 have been determined taking a moderately conservative assessment based on the available Stage 1 and Stage 2 laboratory testing, Stage 2 in situ testing and published correlations between in situ (CPT and SPT) testing and soil parameters, and our local experience. Further detail on interpretation is provided in footnotes below the table.

Table 2-8: Geotechnical design parameters

Geological unit	SPT 'N' range (median)	Typical CPT Qc range (MPa)	Unit weight (kN/m^3)	Effective friction angle (°)	Effective cohesion (kPa)	Undrained shear strength (kPa)	Indicative coefficient of permeability, Kh (m/s)[1]
Embankment Fill	17 to 36 (31)	5 to 20	21	36	0	-	1×10^{-6}
Loess (in situ)[2]	2 to 10 (5)	1 to 5	18	26	3	60	1×10^{-7}
LPF ("compacted liner" from local Loess)	-	-	18	26	3	60	3×10^{-9}
Makirikiri Alluvium	10 to 21 (13)	5 to 15	19	30	6	200	1×10^{-7}
Late Pleistocene Alluvium[3]	-	-	21	41	0	-	1×10^{-6}
Tamaki Alluvium	27 to 50+ (50+)	-	21	41	0	-	1×10^{-6}
Mangaheia Group	50+	-	20	35	50	-	1×10^{-10}

1 Kv/ Kh (Ky/Kx) of 0.1 has been assumed for all soil units.

2 Cyclically softened residual shear strength of the Loess has been taken as 80% of the peak strength based on Robertson and Cabal (2015).

3 The following methodology was used to develop the effective friction and cohesion parameters.

- Borehole descriptions, CPT & SPT measurements, and PSDs indicate the Embankment Fill is typically as described in Table 2-7. An effective friction angle and cohesion typical for a medium dense gravel was adopted for design.
- Borehole descriptions, CPT & SPT measurements, PSDs, and Atterberg limits indicate the in-situ Loess is typically as described in Table 2-7. An effective friction and cohesion have been adopted based on the above interpreted characteristics and strength of the material. These parameters are conservative compared with the single CU triaxial test undertaken on the Loess material.
- Borehole descriptions, CPT & SPT measurements, PSDs, and Atterberg limits indicate the Makirikiri Alluvium is typically as described in Table 2-7. The adopted effective friction and cohesion are based on CU triaxial testing (refer Table 2-3) and interpreted strength from the above in-situ testing.
- For the Late Pleistocene Alluvium and Tamaki Alluvium the adopted effective friction and cohesion are based on material descriptions, available in-situ testing that indicate the material is dense to very dense, and our local experience with similar materials.

4 The following methodology was used to develop the horizontal coefficient of permeability.

- The horizontal coefficient of permeability for the Embankment Fill were based on in-situ falling head tests within the fill. The results of these tests and derivation of permeability coefficients per Hvorslev (1951) and Bouwer & Rice (1976) are presented in the Stage 2 Factual Report.
- The horizontal coefficient of permeability for the LPF was based on triaxial permeability testing undertaken on remoulded Loess specimens from the borrow area.
- The horizontal coefficient of permeability for the in-situ Loess, Makirikiri Alluvium, Late Pleistocene Alluvium and Tamaki Alluvium were approximated based on material grading and our local experience with similar materials.

5 The undrained shear strengths have been derived from correlation with CPT measurements. Mean undrained shear strength for each geological unit are provided above.

Effective (drained) parameters have generally been used for the long-term, elevated groundwater and seismic design cases. For the seismic design cases, a pseudo-static option available in the Slope/W software to characterise shear strengths using effective strength parameters prior to earthquake shaking has been adopted. However, as an exception, *both* drained and undrained parameters have been considered for the cyclically softened residual shear strength of the Loess material.

2.2.7 Potential borrow materials

A possible borrow area for material to reinstate the Low Permeability Fill (LPF) liner is located in a field to the north of the reservoir as shown on Drawing 1020688.4000-5005 in Appendix B of the "Dannevirke Raw Water Reservoir Remediation Design Report" (V01 25% design stage, T+T September 2023). The material targeted is intended to be the same as the original source of LPF liner, which was understood to be the Loess unit.

The borrow is shown at the northern end of the field because the Stage 2 test pits indicated the material in this location was more uniform and was typically described as a clayey SILT with some fine sand, moist, and firm to stiff with low to medium plasticity (note Plasticity Index 12-16% from testing).

Approximately 6,700 m^3 (compacted) would be required if 100% replacement of the existing LPF liner is required. The Stage 2 test pits indicate the Loess comprises a layer approximately 1.2 to 1.3 m thick underlying 300 mm of topsoil. As such, a relatively wide, shallow borrow pit is likely to be required.

From our experience on other projects, Loess deposits can have poor resistance to internal erosion. However, the observations of the material and Stage 1 and Stage 2 laboratory testing indicates that the Loess at this specific site has reasonable resistance to internal erosion based on the HET results, pinhole test results, Plasticity Index, and clay content. The material appears appropriate for a LPF liner i.e., non-dispersive, minimal organic content, sufficient fine material to have low permeability, and is acceptable for an HDPE/EIA/LLDPE subgrade. Properties appear reasonably consistent based on both laboratory testing and field descriptions.

An LPF liner by necessity has a very strict specification that is more challenging to meet than a typical earthworks specification. Construction of the LPF liner can be delayed by weather conditions and / or by difficulty complying with the specification. Placement / compaction trials prior to dewatering are recommended.

2.3 Earthquakes

2.3.1 Seismic site subsoil class

The site has been assessed as Class C as per the criteria set out in NZS 1170.5:2004.

The assessment is based on an estimated low amplitude site period using shear wave velocities (Vs) derived from correlation with site-specific CPTs, and an average depth to the Mangaheia Group Sandstone/ Siltstone inferred from site-specific geotechnical investigations. A low amplitude site period of 0.15 to 0.2 seconds was estimated at the base of the eastern embankment, which is significantly less than a period threshold of 0.6 seconds that differentiates site Class C and Class D.

2.3.2 National seismic hazard model update and Vs30

In October 2022, GNS Science released the revised National Seismic Hazard Model (NSHM). This represents the latest scientific knowledge of earthquake hazard in New Zeaaland and is an important factor for understanding and managing earthquake risk in the built environment.

The 2022 NSHM is expected to inform an update to NZSOLD's New Zealand Dam Safety Guidelines (NZDSG) later in 2024. However, in NSHM's current form, it is not a design standard or guideline that can be directly incorporated into design applications.

In accordance with guidance from MBIE[2], we have assessed seismic performance based on current industry guidance as the base case, noting this does not yet account for the revised NSHM. However, we have

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[2] MBIE's website recommends: "The 2022 NSHM results do not automatically change how we design buildings. Building professionals and practitioners should continue to use existing law, technical standards and guidance to demonstrate that their work complies with the Building Code" (https://www.building.govt.nz/getting-started/seismic-work-programme/national-seismic-hazard-model/).

undertaken an initial appraisal of the implications of the 2022 NSHM update on the stability of the Dannevirke Raw Water Reservoir as a sensitivity case.

The 2022 NSHM uses Vs30 to consider the effect of ground and site conditions on seismic waves as it travels to the surface, where Vs30 is defined as the time-averaged shear wave velocity over the upper 30 m. An approximate Vs30 of 375 m/s has been estimated for the site using Vs derived from correlation with CPTs and by assuming an approximate Vs of 500 m/s for the extremely weak to very weak Mangaheia Group Sandstone / Siltstone.

2.3.3 Seismic hazard

The current version of the NZDSG recommends the following return periods for a Medium PIC Dam when the seismic hazard values are derived using probabilistic methods:

- Operating Basis Earthquake (OBE) – Levels of earthquake shaking due to a 1 in 150 AEP earthquake.
- Safety Evaluation Earthquake (SEE) – Levels of earthquake shaking need not exceed that due to a 1 in 2,500 AEP earthquake.

In addition to the above, a 1 in 50 AEP earthquake comprises a “moderate earthquake” and a 1 in 250 AEP earthquake comprises an “earthquake threshold event” for the purposes of assessing whether a Medium PIC dam is “dangerous” or “earthquake prone”, respectively, under the Building Act 2004. The requirements under the OBE and SEE are expected to govern the design of remedial works, so have been considered in the first instance.

The seismic hazard values for the OBE and SEE and for the design of the remedial works have been developed as follows:

- For the liquefaction and cyclic softening assessment, unweighted peak ground accelerations (PGA) and effective magnitudes values have been obtained from Module 1 (2021). Module 1 was published under Section 175 of the Building Act 2004 and supersedes the seismic hazard values for geotechnical design previously provided in the Bridge Manual (2013).
- Module 1 (2021) does not provide spectral acceleration values (SA). SA values are required for the purpose of estimating seismic slope displacements, where the dam embankments are considered to behave like a flexible structure (Bray and Macedo, 2019). As such, design spectral accelerations have been derived from NZS1170.5:2004 for a Class C site.

As noted above, sensitivity analyses have also been undertaken to understand the implications of the revised NHSM on performance. Table 2-9 presents the unweighted PGA and effective magnitudes from Module 1 (2021) and NSHM 2022. In addition, Figure 2-6 presents the response spectra for 150- and 2500-year return periods based on NZS1170.5 and NSHM 2022. As indicated in the table and figure, the seismic hazard from the revised NSHM is significantly higher than the values in the current design standards and guidelines.

Table 2-9: Unweighted PGA and effective magnitude values

Return period	Module 1 (2021)		NSHM 2022	
	PGA (g)	Magnitude (M_w)	PGA (g)	Magnitude (M_w)
150 (OBE)	0.32	7.0	0.46	7.4
2500 (SEE)	1.01	7.5	1.59	7.9

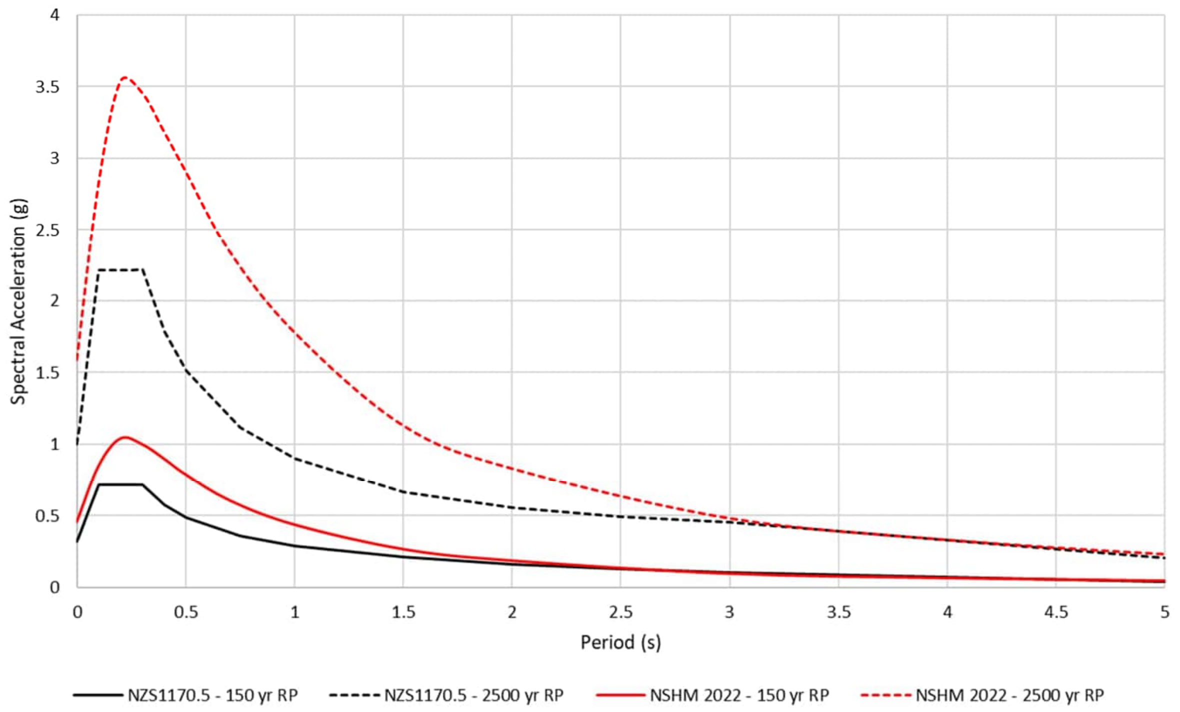

Figure 2-6: 5% damped response spectra for Dannevirke Raw Water Reservoir based on NZS1170.5 and NSHM 2022.

We have not assessed performance in an aftershock. While the NZDSG recommends consideration of aftershocks for all High PIC dams, it is not specifically recommended for a Medium PIC dam such as the Dannevirke Raw Water Reservoir.

2.3.4 Liquefaction and cyclic softening

2.3.4.1 General

Liquefaction and cyclic softening vulnerability have been assessed using the following steps:

1. Estimating the liquefaction / cyclic softening susceptibility of the soils present.
2. Estimating whether liquefaction / cyclic softening will be triggered in a susceptible soil layer for a given depth to groundwater and design levels of earthquake shaking.
3. Estimating the reduced soil strength because of the triggering of liquefaction / cyclic softening.

The liquefaction susceptibility and triggering assessment for ‘sand-like’ soils has been undertaken using the procedure outlined by Boulanger and Idriss (2014) and available CPT data. The susceptibility assessment has been based on the soil behaviour index (I_c) estimated from the CPT and using the criteria outlined by Bray and Sancio (2006).

The cyclic softening triggering assessment for ‘clay-like’ soils has been undertaken using the procedure outlined by Idriss and Boulanger (2008).

The shear strength of soil on triggering of liquefaction has been estimated by taking a weighted average of the liquefied strength ratio estimated using the methods proposed by Idriss and Boulanger (2008), Kramer and Wang (2015), and Weber et al (2015).

The shear strength of soil on triggering of cyclic softening has been estimated as per the evaluation method suggested by Robertson and Cabal (2015).

The detailed outputs from the liquefaction and cyclic softening assessment have been presented previously in Appendix D8 of the “Dannevirke Raw Water Reservoir Remediation Design Report” (V01 25% design stage, T+T September 2023). The key assumptions and results from the liquefaction and cyclic softening assessment are summarised below for ease of reference.

2.3.4.2 Liquefaction assessment results

The liquefaction triggering assessment has been undertaken using CPT data collected along the crest of the eastern embankment. The I_c calculated from the CPTs indicate that the Embankment Fill and Makirikiri Alluvium soils are susceptible to liquefaction if they are below the groundwater table. Loess deposits were generally noted to be too plastic to be susceptible to 'sand-like' liquefaction. However, the potential for cyclic softening of this material has been assessed and is discussed further below.

The liquefaction triggering assessment under design levels of earthquake shaking indicate that thin layers within the gravelly Embankment Fill and Makirikiri Alluvium are potentially liquefiable under OBE and SEE levels of earthquake shaking if saturated.

However, the impact of any liquefaction triggering is expected to be minor for the stability of the dam embankment for the following reasons.

- It is expected that the Embankment Fill is generally above the permanent groundwater table. This is based on the elevation at which groundwater seepage has been observed during site investigation and available groundwater monitoring results (even with the current abnormal subsoil seepage flows). Soil material above the groundwater table is not susceptible to liquefaction.
- Sensitivity analyses, with Embankment Fill assumed to be saturated, showed only thin, isolated bands are likely to liquefy under OBE and SEE levels of earthquake shaking. Side-by-side comparison of the CPT results show poor continuity of these thin layers that are potentially liquefiable. As such, the likelihood of a continuous liquefiable layer forming that could impact the stability of the embankment is considered low.
- Similarly, only very thin bands within the Makirikiri Alluvium were noted to trigger under OBE and SEE levels of earthquake shaking. Comparison of CPTs side-by-side shows no evidence of continuity in any potentially liquefiable layer. The late-Pleistocene Makirikiri Alluvium recovered in the borehole was also generally dense to very dense. This material would be expected to have a very low potential for liquefaction.
- Simplified liquefaction triggering assessment procedures consider each layer in isolation. However, system response effects (Module 3, 2021), where thin liquefiable layers are interbedded within non-liquefiable layers, is likely to significantly limit the generation of excess porewater pressures. For this reason, the triggering assessment using simplified procedures is expected to be conservative for the Embankment Fill and Makirikiri Alluvium.

2.3.4.3 Cyclic softening assessment results

As noted above, Loess deposits at the site were typically found to behave in a 'clay-like' manner and for this reason were not susceptible to liquefaction. However, this type of material can be susceptible to cyclic softening.

A cyclic softening triggering assessment was undertaken using CPT data and indicates that:

- Isolated bands of material (typically 0.2-0.5 m thick) may cyclically soften under OBE levels of earthquake shaking. As such, the likelihood of a continuous softened layer forming under the OBE that could impact the stability of the embankment is considered low.
- Most of the Loess deposits that behave in a 'clay-like' manner are likely to cyclically soften under SEE levels of earthquake shaking.

A softened strength has been adopted for the Loess deposits during SEE levels of earthquake shaking and a post-earthquake design case (but *not* during OBE levels of earthquake shaking). The softened strength for the SEE and post-earthquake cases has been adopted based on guidance provided in Robertson and Cabal (2015);

specifically, the softened strength has been taken as 80% of the peak undrained shear strength for stability analyses.

3 Stability of the western reservoir rim (issue “a”)

3.1 Seepage and stability modelling

3.1.1 Methodology

Slope stability and seepage analyses have been undertaken for the western reservoir rim based on the geological cross section in Appendix A.

The stability of the western reservoir rim has been modelled and assessed using Slope/W (Limit Equilibrium method) based on the geotechnical design parameters presented in Table 2-8. Applicable design cases have been assessed against the design criteria provided in NZSDSG for a Medium PIC Dam including:

1. Long-term static stability – a minimum Factor of Safety (FoS) of 1.5 has been adopted as recommended in Table 6.3 of Module 3 of the NZDSG.
2. Operating Basis Earthquake (OBE) as described in Section 2.3.3 – the recommended performance is that the dam and appurtenant structures should remain operational, and any damage should be no more than minor and readily repairable. A minimum FoS of “generally 1.0” is recommended per Table 6.4 of Module 3 of the NZDSG. However, we note that in some cases a non-compliance where the FoS < 1, only results in very small deformations (< 20 mm) and arguably could be considered “no more than minor” depending on the impact on functionality and ease of repair.
3. Safety Evaluation Earthquake (SEE) as described in Section 2.3.3 - the SEE is a more severe earthquake than the OBE. Some damage may occur in the SEE but the recommended performance is that the SEE should not lead to an uncontrolled release (meaning a dam failure). Compliance has been interpreted as the estimated deformations being less than the freeboard at normal reservoir levels (1.26 m). While performance may be assessed as compliant in the SEE based on the deformations being less than the freeboard and there being no uncontrolled release, *it is expected that the liner will have failed*. Following the SEE, it is expected that the reservoir will need to be dewatered for inspection and repair, and that the *reservoir will be out of service for water supply to Dannevirke*.
4. Post-earthquake – a minimum FoS of 1.2 to 1.3 is recommended per Table 6.4 of Module 3 of the NZDSG. The lower end of this range, 1.2, has been adopted based on the level of conservatism of input parameters.

The long-term porewater pressure has been modelled to reflect groundwater monitoring data (refer to Table 2-6) and the elevation of seeps observed on site, particularly at waypoint WP03. A uniform groundwater pressure at 258.9 mRL has been adopted for the stability assessment based on the maximum groundwater level measured within BH02 at the VWP2 location. The maximum groundwater level has been adopted for analysis as the groundwater appears to be sensitive to rainfall and the level corresponds well with the elevation at which groundwater seepage has been noted.

Porewater pressures for the post-earthquake design case for the downstream slope are based on the long term porewater pressure conditions outlined above, raised by 2.5 m to allow for leakage from the reservoir following damage to the liner system in the SEE.

The operational range of the reservoir ranges from Full Supply Level to close to empty. The reservoir is assumed to be drawn down where this is more critical for stability performance (i.e., checking the upstream stability of the slope).

Sensitivity cases assessed include:

- Long-term stability with liner failure / full leakage with pore water pressures equivalent to the post-earthquake case (discussed above).

- Long-term stability with lower bound soil strengths.
- OBE and SEE stability and deformation with increased seismic hazard based on NSHM 2022 values.
- Long-term, OBE, SEE, and post-earthquake performance with a softened (lower strength) surface present at the interface between the Tamaki Alluvium and underlying Mangaheia Formation. A friction angle of 22° and depth of 7 m below the downstream toe (maximum investigated depth) has been assumed in the analysis for the softened surface.

Deformation of the embankment slopes under seismic shaking has been assessed using the simplified method provided by Bray and Macedo (2019) for a subduction zone earthquake, based on the 50th percentile values. The relationships provided for a subduction zone have been used because available disaggregation information near the site indicate the Hikurangi subduction zone dominates the seismic hazard for OBE and SEE design cases.

Preliminary seepage analyses have been undertaken using Seep/W (Finite Element analysis) and used a saturated / unsaturated material model for all soil units. The assumed boundary conditions are shown in the outputs presented in Appendix C.

A steady-state seepage analysis was undertaken to represent the dam under normal operating conditions with the low-permeability HDPE/EIA/LLDPE and LPF liner in place and performing as intended.

A transient seepage analysis has also been undertaken with the reservoir drawn down from Full Supply Level (FSL) over a period of three days with the HDPE/EIA/LLDPE and LPF liner effectively “removed” to represent gross damage following the SEE earthquake. The dewatering time has been estimated based on hydraulic capacity of the subsoil network and assuming the liner system is damaged such that it does not limit flows from the reservoir into the subsoil network. Initial porewater pressures for the transient seepage analysis were taken from the steady-state seepage (parent) analysis where the HDPE/EIA/LLDPE and LPF liner are performing as intended. A sensitivity case was assessed to represent upper bound hydraulic conductivity for the Embankment Fill of 1×10^{-5} m/s.

3.1.2 Results

The stability analysis results are summarised in Table 3-1 with the Slope/W outputs presented in Appendix C.

Table 3-1: Stability results for western reservoir rim (orange indicates non-compliance)

Design case	Min. required FoS	Min. calculated FoS		Estimated seismic slope displacement (mm)	
		Upstream	Downstream	Upstream	Downstream
1. Long-term stability	1.5	2.37	1.53	-	-
Seismic*					
2. Seismic (OBE)	1.0	1.26	<1.0 Slip surfaces with FoS < 1.0 do NOT cross dam crest	-	<20
3. Seismic (SEE)	Disp. < freeboard	<1.0	<1.0	45	155
4. Post-earthquake – With damage to liner	1.2	2.37**	1.38	-	-
Sensitivity					
1a. Long-term stability – Liner failure/ full leakage (Downstream only)	Assume 1.3	Not assessed	1.38	Not assessed	-

Design case	Min. required FoS	Min. calculated FoS		Estimated seismic slope displacement (mm)	
		Upstream	Downstream	Upstream	Downstream
1b. Long-term stability – Lower bound soil strengths (Downstream only)	Assume 1.3	Not assessed	1.38	Not assessed	-
2a. Seismic (OBE) – 2022 NSHM	1.0	1.04	<1.0 Slip surfaces with FoS < 1.0 CROSS dam crest	-	30
3a. Seismic (SEE) – 2022 NSHM	Disp. < freeboard	<1.0	<1.0	220	650
1c. Long-term stability – Softened surface	1.5	2.24	1.51	-	-
2b. Seismic (OBE) – Softened surface	1.0	<1.0 Slip surfaces with FoS < 1.0 intercept but do NOT completely cross dam crest	<1.0 Slip surfaces with FoS < 1.0 CROSS dam crest	<20	60
3b. Seismic (SEE) – Softened surface	Disp < freeboard	<1.0	<1.0	165	575
4a. Post-earthquake – Softened surface	1.2	2.24**	1.40	-	-

*Soils underlying the western reservoir rim are not susceptible to cyclic softening and therefore softened strengths have not been applied under the seismic SEE and post-earthquake design cases.
**The FoS for the post-earthquake design case is essentially the same as the FoS for long-term stability with the liner intact and performing as intended as the groundwater pressures are similar for the critical upstream failure mechanism.

3.1.2.1 Upstream slope stability and deformation

The above results indicate that the stability performance of the upstream / internal slope of the western reservoir rim is generally compliant with design criteria. There is a marginal non-compliance in the OBE for one of the sensitivity cases, noting that sensitivity cases are more uncertain than the design cases and non-compliance should be considered possible but unconfirmed.

3.1.2.2 Downstream slope stability and deformation

The downstream / external slope of the western reservoir rim encompasses the access track to the reservoir and treatment plant. The stability performance of this slope is compliant under normal operating (long-term) conditions, and for the post-earthquake and SEE design cases. The compliance for the long-term stability case is marginal and there is a risk this could change to non-compliant with only slight changes in the analysis.

For the OBE design case, stability performance of the downstream slope is marginally non-compliant (FoS of 0.95) based on the currently assumed ground model (no softened surface) and Module 1 (2021) seismic hazard values. Given that the FoS for the slope is just less than 1.0, there is potential for some permanent yielding of the slope under OBE levels of shaking. Any permanent deformation is expected to be small (<20 mm) and shallow. This damage would most likely affect the external face and access track but is not expected to cross the crest of the reservoir rim to damage the liner.

For the sensitivity cases considering the 2022 NSHM and the presence of a softened surface along the soil-rock interface, the non-compliance in the OBE worsens and permanent yield and minor slope displacements are anticipated. Displacements of 30 mm and 60 mm respectively are estimated in the OBE. For these sensitivity cases, slip surfaces with a FoS < 1 in the OBE are modelled as extending across the dam crest to the upstream face of the dam, potentially resulting in liner damage.

3.1.2.3 Seepage

The Seep/W outputs are presented in Appendix C. The seepage analyses indicate no seepage through the embankment where the HDPE/EIA/LLDPE and LPF liner is intact under FSL. Transient seepage analyses with the liner effectively removed (representing damage following the SEE) indicate that only the first few meters of the dam embankment have time to become saturated in the three days that the dam is drawn down. Note that the transient seepage analysis is not representative of hydraulic conditions should significant transverse cracks occur in the SEE, as discussed further in Section 5.

The results of the sensitivity case using the upper bound hydraulic conductivity for the embankment fill indicate high hydraulic gradients (>1.3) extend into the upstream face by approximately 6 m further.

3.2 Dam safety risk

The analyses above have identified one marginal non-compliance for the base case and several potential non-compliances for sensitivity cases with respect to stability performance recommendations set out in the NZDSG:

- Marginal non-compliance with recommended performance in the OBE for the downstream / external face of the western reservoir rim in the base design case (< 20 mm permanent displacement estimated). The non-compliance worsens for sensitivity cases (30 mm and 60 mm displacement estimated for the cases considering NHSM 2022 and the presence of a softened surface respectively). Moreover, displacements are confined to the downstream face for the base case but may extend across the reservoir rim to affect the upstream face and liner for the sensitivity cases.
- Marginal non-compliance with recommended performance in the OBE for the upstream / internal face of the western reservoir rim in one of the sensitivity cases (< 20 mm permanent displacement estimated for the case with a softened surface present).

The performance in the sensitivity cases should be considered possible but not confirmed. Guidance on application of NHSM 2022 is scheduled to be provided in an update to the NZDSG later this year and may change the assessment. Similarly, the presence of a softened (lower strength) surface at the interface between the Tamaki Alluvium and underlying Mangaheia Formation has not been observed directly but has only been inferred as a risk based on experience in similar materials.

To provide guidance on the level of urgency, we note that these non-compliances are well below thresholds for immediate danger or that would make the dam “dangerous” or “earthquake-prone” under the Building Act 2004. A “dangerous” dam is defined as likely to fail in the ordinary course of events or in a moderate earthquake, which comprises a 1 in 50 AEP earthquake for a Medium PIC dam. An “earthquake-prone” dam is defined as likely to fail in an earthquake threshold event, which comprises a 1 in 250 AEP earthquake for a Medium PIC dam. The analyses above have indicated that the dam is unlikely to fail due to stability / deformation in the SEE, which is a much larger earthquake (1 in 2,500 AEP) i.e., there is a substantial margin of safety before the dam would be considered “earthquake-prone” or “dangerous” due to instability in an earthquake.

The recommended performance for the OBE is that the dam and appurtenant structures should remain operational, and any damage should be no more than minor and readily repairable. This has been taken as a minimum FoS of 1.0 per Table 6.4 of the NZDSG.

The consequence of non-compliance is that “more than minor” damage may occur in a smaller earthquake than recommended for a Medium PIC dam. This “more than minor” damage for the downstream slope is predicted to comprise permanent displacements <20 mm in the OBE for the base case (potentially up to 60 mm for one of the sensitivity cases). As noted above, in the base case, these displacements are expected to affect the downstream slope including the access track to the reservoir and treatment plant, but not the dam crest or reservoir liner. However, for the sensitivity cases, the displacements may extend to the upstream face of the dam and possibly result in liner damage. For the base case where the displacements are limited to the external face, it could be argued that the deformations are “no more than minor damage”. However, the FoS is less than 1 so compliance is arguable.

If the non-compliance of the upstream slope in the OBE is confirmed for the sensitivity case with a softened surface present, this could result in displacements < 20mm of the upstream face in the OBE and possibly damage to the liner.

3.3 Remedial options

3.3.1 Downstream slope

Remedial options to improve the stability performance of the downstream / external slope of the western reservoir rim in the OBE could comprise installation of:

- Drainage,
- A stabilisation berm, and / or
- Shear piles.

The specific measures would depend on whether any of the sensitivity cases are confirmed and would require further investigation and design. Drainage and / or a stabilisation berm may be sufficient if there is no softened surface, while deeper options like shear piles would be more relevant if a softened surface is confirmed as present. The measures above, if confirmed as required, would likely be constructed on the downstream face without dewatering or taking the reservoir out of service.

If needed, shear piles would likely be keyed into the underlying Mangaheia Group sandstone/siltstone. Shear piles could comprise bored reinforced concrete sections installed at regular spacings along the downstream slope of the western reservoir rim. Deep soil mixing (DSM), or continuous flight augur (CFA) piles are also potential options. Design of the shear piles would depend on the depth and characteristics (strength) of the softened surface (if present), and as such further investigations would be required to confirm this.

3.3.2 Upstream slope

Physical interventions to improve the stability performance of the upstream / internal slope of the western reservoir rim in the OBE would likely be more challenging and require dewatering to implement. However, it is noted that this non-compliance is for a sensitivity case that is considered relatively unlikely. If confirmed as needed, investigations to address the non-compliance may identify that alternatives to physical intervention are preferred i.e., surveillance, emergency preparedness, or measures to mitigate the risk to water supply.

3.4 Residual areas of uncertainty / risk

We note the following areas of uncertainty / risk with respect to stability, deformation, and seepage performance of the western reservoir rim slopes:

- As already noted, there is considered a risk of a softened (lower strength) surface being present along the soil-rock interface underlying the western reservoir rim slopes. However, the presence, depth, and characteristics of a softened surface are very uncertain based on current information. There may be no softened surface present, and no remediation needed. For the purposes of the current report, this uncertainty has been addressed as a sensitivity case. The assessment for the sensitivity case indicates that non-compliance in the OBE would likely be worsened if a softened surface is present.
- A friction angle of 22° has been assumed for the softened surface in the sensitivity case but this is very uncertain as the interface has not been directly inspected i.e., by borehole. Currently the western reservoir rim slopes are non-compliant for just the OBE case for the sensitivity case with a softened surface. However, friction angles as low as 11° may be possible for softened surfaces (if present). A lower friction angle could lead to further non-compliances for other design cases (not just the OBE).

- Guidance on how to consider the 2022 NSHM seismic hazard in assessment and design of dams is expected to be released in an update to the NZDSG later this year. This may change the recommended industry practice for seismic assessment and performance. This uncertainty has been addressed as a sensitivity case for the purposes of the current report. The assessment for the sensitivity case indicates the non-compliance in the OBE could potentially be worsened by considering 2022 NSHM seismic hazard, noting that this will depend on the guidance yet to be published.

3.5 Recommendations for decision-making and further work

Based on dam safety and engineering considerations, we recommend that the non-compliances for stability performance of the western reservoir rim are addressed as part of routine asset management and renewal processes separate from the urgent remedial works to the liner, subgrade, and subsoil network, because:

- The non-compliances do not relate directly to the higher risk potential failure mode shown in Figure 0.1 that that requires urgent remedial work if the dam is to remain in service.
- The non-compliances do not represent a level of risk necessitating emergency intervention.
- More information will be available on the sensitivity case relating to NHSM 2022 when an update to NZDSG is published later in 2024.
- If intervention is confirmed as necessary, this would most likely be for the external slopes. These interventions could most likely be constructed without dewatering the reservoir i.e., would not necessarily benefit from being undertaken while the reservoir was already dewatered for the urgent remedial works.

We expect that addressing the stability non-compliances will be required under activities under the Building (Dam Safety) Regulations 2022. This further work is anticipated to comprise developing and implementing a defensible plan to investigate, confirm, and resolve the non-compliances in a timeframe that reflects the level of risk of the non-compliances.

As noted above, the non-compliances in the OBE do not represent a level of risk necessitating emergency intervention. It is anticipated that the investigations and work to resolve the non-compliances would follow a reasonable asset renewal cycle that would provide time to set aside budget for the works. These works would likely include:

- Resolving the uncertainty relating to NHSM 2022 i.e., updating the assessment of seismic performance (OBE and SEE) once the update to NZDSG is published.
- Resolving the uncertainty associated with a softened surface:
  - Confirming or disproving the presence of a softened surface, and if present, identifying the depth and characteristics of the surface. This is expected to require additional geotechnical investigation comprising a minimum of one machine borehole and piezometer installation drilled near the access track. The machine borehole would need to be cored and advanced to a depth below the top of the Mangaheia Formation to allow inspection of the soil-rock interface and recover any potential softened surface.
  - If a softened surface is present, refining the ground model and stability modelling for the western reservoir slopes based on the findings of the additional geotechnical investigation.
- Once the uncertainties above are resolved, finalising the stability modelling and confirming the scale and risk of the stability non-compliances for the western reservoir rim.
- This would then inform development and implementation of options to address the non-compliances, which might comprise a combination of surveillance, emergency preparedness, operational measures, and physical works. The investigations may potentially determine that physical works are not necessary.

4 Stability of the eastern dam embankment (issue “b”)

4.1 Seepage and stability modelling

4.1.1 Methodology

Slope stability and seepage analyses have been undertaken for the eastern embankment slopes based on the geological cross section presented in Appendix A.

The stability of the eastern embankment slopes has been modelled and assessed using Slope/W (Limit Equilibrium method) based on the geotechnical design parameters presented in Table 2-8. Applicable design cases have been assessed against the design criteria provided in NZSDSG for a Medium PIC Dam including:

1. Long-term static stability – a minimum FoS of 1.5 has been adopted as recommended in Table 6.3 of Module 3 of the NZDSG.
2. Operating Basis Earthquake (OBE) as described in Section 2.3.3 – the recommended performance is that the dam and appurtenant structures should remain operational, and any damage should be no more than minor and readily repairable. A minimum FoS of “generally 1.0” is recommended per Table 6.4 of Module 3 of the NZDSG. However, we note that in some cases a non-compliance where the FoS < 1, only results in very small deformations (< 20 mm) and arguably could be considered “no more than minor” depending on the impact on functionality and ease of repair.
3. Safety Evaluation Earthquake (SEE) as described in Section 2.3.3 - the SEE is a more severe earthquake than the OBE. Some damage may occur in the SEE but the recommended performance is that the SEE should not lead to an uncontrolled release (meaning a dam failure). Compliance has been interpreted as the estimated deformations being less than the freeboard at normal reservoir levels (1.26 m). While performance may be assessed as compliant in the SEE based on the deformations being less than the freeboard and there being no uncontrolled release, *it is expected that the liner will have failed*. Following the SEE, it is expected that the reservoir will need to be dewatered for inspection and repair, and that *the reservoir will be out of service for water supply to Dannevirke*.
4. Post-earthquake – a minimum FoS of 1.2 to 1.3 is recommended per Table 6.4 of Module 3 of the NZDSG. The lower end of this range, 1.2, has been adopted based on the level of conservatism of input parameters.

The stability of the eastern dam embankment is governed by slip surfaces in the Loess and Makirikiri alluvium depending on the specific design or sensitivity case under consideration. Based on the groundwater monitoring to date, groundwater levels in the Loess are likely to be below the tip level of BH01 VWP2 (257.9 mRL). The porewater pressures measured in BH01 VWP2 have generally been negative except during the 6 weeks immediately following installation. BH01 VWP3 indicates a maximum groundwater level in the Makirikiri Alluvium at 255.9 mRL. For the stability analyses, a moderately conservative uniform groundwater level at 257.5 mRL has been adopted, 0.4 m below the tip level for BH01 VWP2 and 1.6 m above the maximum groundwater level in BH01 VWP3.

Porewater pressures for the post-earthquake design case for the downstream slope are based on the long term porewater pressure conditions outlined above, raised by 2.5m to allow for leakage from the reservoir following damage to the liner system in the SEE.

The operational range of the reservoir ranges from Full Supply Level to close to empty. The reservoir is assumed to be drawn down where this is more critical for stability performance (i.e., checking the upstream stability of the slope).

Sensitivity cases assessed include:

- Long-term stability with liner failure / full leakage with pore water pressures equivalent to the post-earthquake case (discussed above).
- Long-term stability with lower bound soil strengths.

- OBE and SEE performance with increased seismic hazard based on NSHM 2022.

Deformation of the embankment slopes under seismic shaking has been assessed using the simplified method provided by Bray and Macedo (2019) for a subduction zone earthquake, based on the 50th percentile values. The relationships provided for a subduction zone have been used because available disaggregation information near the site indicate the Hikurangi subduction zone dominates the seismic hazard for OBE and SEE design cases.

Preliminary seepage analyses have been undertaken using Seep/W (Finite Element analysis) and used a saturated/ unsaturated material model for all soil units. The assumed boundary conditions are shown in the outputs presented in Appendix C.

A steady-state seepage analysis was undertaken to represent the dam under normal operating conditions with the low-permeability HDPE/EIA/LLDPE and LPF liner in place and performing as intended.

A transient seepage analysis has been undertaken with the reservoir drawn down from FSL over a period of three days with the HDPE/EIA/LLDPE and LPF liner effectively “removed” to represent gross damage following the SEE earthquake. The dewatering time has been estimated based on hydraulic capacity of the subsoil network and assuming the liner system is damaged such that it does not limit flows from the reservoir into the subsoil network. Initial porewater pressures for the transient seepage analysis were taken from the steady-state seepage (parent) analysis where the HDPE/EIA/LLDPE and LPF liner are performing as intended. A sensitivity case was assessed to represent upper bound hydraulic conductivity for the Embankment Fill of 1 x 10^{-5} m/s.

4.1.2 Results

The stability analysis results are summarised in Table 4-1 with the Slope/W outputs presented in Appendix C.

Table 4-1: Stability results for eastern embankment slopes (orange indicates non-compliance)

Design case	Min. required FoS	Min. calculated FoS		Estimated seismic slope displacement (mm)	
		Upstream	Downstream	Upstream	Downstream
1. Long-term stability	1.5	2.05	1.35 Slip surfaces with FoS < 1.5 do NOT cross dam crest	-	-
Seismic					
2. Seismic (OBE)	1.0	1.11	<1.0 Slip surfaces with FoS < 1.0 CROSS dam crest	-	25
3. Seismic (SEE)*	Displacement to be < freeboard (1260 mm)	<1.0	<1.0	210	925
4. Post-earthquake* – Damage to liner	1.2	1.97	<1.2 Slip surfaces with FoS < 1.2 do NOT cross dam crest	-	-
Sensitivity					
1a. Long-term stability – Liner failure/ increased leakage (Downstream only)	Assume 1.3	Not assessed	1.20 Slip surfaces with FoS < 1.3 do NOT cross dam crest	Not assessed	-
1b. Long-term stability – Lower bound soil strengths (Downstream only)	Assume 1.3	Not assessed	1.13 Slip surfaces with FoS < 1.3 do NOT cross dam crest	Not assessed	-

Design case	Min. required FoS	Min. calculated FoS		Estimated seismic slope displacement (mm)	
		Upstream	Downstream	Upstream	Downstream
2a. Seismic (OBE) – 2022 NSHM Seismic Hazard	1.0	<1.0 Slip surfaces with FoS < 1.0 CROSS dam crest	<1.0 Slip surfaces with FoS < 1.0 CROSS dam crest	<20	70
3a. Seismic (SEE)* – 2022 NSHM Seismic Hazard	Displacement to be < freeboard (1260 mm)	<1.0	<1.0	835	2,910

*Soils underlying the eastern embankment are potentially susceptible to cyclic softening. Softened strengths have been applied under the SEE and post-earthquake design cases as discussed in Section 2.3.4.

4.1.2.1 Upstream slope stability

The above results indicate that the stability performance of the upstream / internal slope of the eastern dam embankment is generally compliant with design criteria. There is a marginal non-compliance in the OBE for one of the sensitivity cases, noting that sensitivity cases are more uncertain than the design cases and non-compliance should be considered possible but unconfirmed.

4.1.2.2 Downstream slope stability

The downstream / external slope of the eastern dam embankment is moderately non-compliant with the design criteria under the normal operating (long-term), and post-earthquake design cases.

Performance in the OBE is also non-compliant with permanent displacements of 25 mm estimated. Slip surfaces with a FoS < 1 are modelled as extending across the dam crest to the upstream face of the dam, potentially resulting in liner damage. The slip surfaces are also relatively deep seated i.e., may extend relatively deeply into the dam embankment, which could be more difficult to repair and raise concerns for embankment integrity. For the sensitivity case considering the 2022 NSHM, the non-compliance in the OBE worsens and permanent displacements of 70 mm are estimated.

Performance in the SEE is also non-compliant for the sensitivity case considering 2022 NSHM spectra. The estimated permanent displacement exceeds the available freeboard (1.26 m) and could result in an uncontrolled release of reservoir contents.

4.1.2.3 Downstream slope stability with remediation

Modelling has been completed to identify the likely type and scale of remediation needed to improve stability performance of the downstream slope. The modelling and remedial options presented should be considered preliminary and concept level. Further work will be required to advance these to a detailed design level. Refer also to further discussion of remedial options in Section 4.3.1.

The remedial options considered comprise:

- Option A: Counterfort drains 4 m deep and extending 20 m upslope.
- Option B: Drainage and stabilisation berm with 10 m wide crest at RL 255 m, and 3H:1V slope.
- Option C: Drainage and stabilisation berm with 17 m wide crest at RL 260 m, and 3H:1V slope.
- Option D: Drainage and stabilisation berm larger than above, and / or a deep shear key or shear piles.

Table 4-2 presents the improvements in compliance with stability criteria indicated by the preliminary modelling. The performance for the existing situation (already presented in Table 4-1) has been included for ease of comparison. More detailed results for Option C are presented in Table 4-3 and the Slope/W outputs in Appendix C.

Table 4-2: Indicative stability improvements for eastern embankment downstream slope with remediation

Design case	Requirement	Existing	Option A (drainage only)	Option B (255 mRL berm)	Option C (260 mRL berm)	Option D** (larger berm and/or shear key / piles)
1. Long-term stability	FoS > 1.5	NC	Complies	Complies	Complies	Complies
Seismic						
2. Seismic (OBE)	FoS > 1.0	NC	NC	NC	NC	Complies
3. Seismic (SEE)*	Disp. < freeboard	Complies	Complies	Complies	Complies	Complies
4. Post-earthquake* – Damage to liner	FoS > 1.2	NC	NC	Complies	Complies	Complies
Sensitivity						
2a. Seismic (OBE) – 2022 NSHM	FoS > 1.0	NC	NC	NC	NC	Complies
3a. Seismic (SEE)* – 2022 NSHM	Disp. < freeboard	NC	NC	NC	Complies	Complies

"NC" = non-compliant.

*Soils underlying the eastern embankment are potentially susceptible to cyclic softening. Softened strengths have been applied under the SEE and post-earthquake design cases as discussed in Section 2.3.4.

**Performance assumed, not modelled.

Table 4-3: Stability results for eastern embankment slopes with remedial Option C

Design case	Min. required FoS	Min. calculated FoS	Estimated seismic slope displacement (mm)
		Downstream	Downstream
1. Long-term stability	1.5	2.23	-
Seismic			
2. Seismic (OBE)	1.0	<1.0** Slip surfaces with FoS < 1.0 CROSS dam crest	<20
3. Seismic (SEE)*	Displacement to be < freeboard (1260 mm)	<1.0	315
4. Post-earthquake* – Damage to liner	1.2	1.72	-
Sensitivity			
2a. Seismic (OBE) – 2022 NSHM Seismic Hazard	1.0	<1.0 Slip surfaces with FoS < 1.0 CROSS dam crest	35
3a. Seismic (SEE)* – 2022 NSHM Seismic Hazard	Displacement to be < freeboard (1260 mm)	<1.0	1,200

*Soils underlying the eastern embankment are potentially susceptible to cyclic softening. Softened strengths have been applied under the SEE and post-earthquake design cases as discussed in Section 2.3.4.

**Performance is non-compliant but improved against the existing situation; the FoS increases from 0.82 to 0.93, and estimated displacements decrease from 25 mm to < 20 mm. Slip surfaces with a FoS < 1 are still expected to extend across the dam crest to the upstream face of the dam in the OBE with the berm.

4.1.2.4 Seepage

The Seep/W outputs are presented in Appendix C. The seepage analyses indicate no seepage through the embankment where the HDPE/EIA/LLDPE and LPF liner is intact under FSL. Transient seepage analyses with the liner effectively removed (representing damage following the SEE) indicate that only the first few meters of the dam embankment have time to become saturated in the three days that the dam is drawn down. Note that the transient seepage analysis is not representative of hydraulic conditions should significant transverse cracks occur in the SEE, as discussed further in Section 5.

The results of the sensitivity case using the upper bound hydraulic conductivity for the embankment fill indicate water total heads approximately 2 m higher than the baseline. The results of this case also indicate high hydraulic gradients (>1.3) extend further into the upstream face by approximately 3 m.

4.2 Dam safety risk

The analyses described in the preceding sections have identified three confirmed non-compliances for base design cases and several potential non-compliances for sensitivity cases with respect to stability performance recommendations set out in the NZDSG:

- Non-compliance with recommended performance for the long-term static, post-earthquake, and OBE base cases for the downstream / external face of the eastern dam embankment. Performance in the SEE is non-compliant for one of the sensitivity cases (displacement exceeding freeboard for the case considering 2022 NHSM).
- Marginal non-compliance with recommended performance in the OBE for the upstream / internal face of the western reservoir rim in one of the sensitivity cases (< 20 mm permanent displacement estimated for the case considering 2022 NHSM).

The performance in the sensitivity cases should be considered possible but not confirmed. Guidance on application of NHSM 2022 is scheduled to be provided in an update to the NZDSG later this year and may change the assessment.

To provide guidance on the level of urgency, we note that these non-compliances are well below thresholds for immediate danger or that would make the dam “dangerous” or “earthquake-prone” under the Building Act 2004.

The recommended performance for the OBE is that the dam and appurtenant structures should remain operational, and any damage should be no more than minor and readily repairable. This has been taken as a minimum FoS of 1.0 as per Table 6.4 of the NZDSG.

The consequence of non-compliance is that “more than minor” damage may occur in a smaller earthquake than recommended for a Medium PIC dam. This “more than minor” damage for the downstream slope is predicted to comprise permanent displacements 25 mm in the OBE for the base case (potentially 70 mm for the sensitivity case considering 2022 NHSM). The displacements may extend to the upstream face of the dam and possibly result in liner damage.

If the non-compliance of the upstream slope in the OBE is confirmed for the sensitivity case with 2022 NHSM seismic hazard considered, this could result in displacements < 20mm of the upstream face in the OBE and possibly damage to the liner.

4.3 Remedial options

4.3.1 Downstream slope

As already noted in Section 4.1.2.3, a preliminary design has been developed for a drainage and stabilisation berm to improve the stability and deformation performance of the downstream shoulder. The design is shown in sketches in Appendix G. A preliminary “middle” cost estimate for the design is $ 2.8 to $ 3.2 M, which adds approximate 35% to the overall remedial works costs (refer Appendix H for further detail).

This berm design is slightly larger than Option B, which was presented in Section 4.1.2.3. Based on the modelling in Section 4.1.2.3, the design is expected to:

- Eliminate the non-compliance for long-term stability base case.
- Eliminate the non-compliance for the post-earthquake base case.
- Would not eliminate the non-compliance in the SEE for the sensitivity case with 2022 NHSM seismic hazard.
- Would not eliminate the non-compliance in the OBE base case.

Following construction of the berm (as shown in Appendix G, slightly larger than Option B bund), residual non-compliances are expected to include non-compliance in the OBE and possibly non-compliance in the SEE for the sensitivity case. Remedial options to address the residual non-compliances could comprise enlarging the stabilisation berm further and/or a deep shear key or shear piles i.e., Option D from Section 4.1.2.3. If confirmed as needed, these measures would likely be constructed on the downstream face without dewatering or taking the reservoir out of service.

Table 4-4 provides comment on the dam safety risk of three alternative approaches to manage the risks of the stability non-compliances for the eastern dam embankment.

Table 4-4: Comparison of alternative approaches to manage eastern dam embankment stability risks

Approach	Dam safety risk
Construct the drainage and stabilisation berm at the same time as the liner, subgrade, and subsoil works.	The liner, subgrade, and subsoil works provide the step-change reduction in dam safety risk related to the potential failure mode in Figure 0.1. This approach is considered higher risk for dam safety if design, approval, and funding of the berm delays the more dam safety critical works above.
Construct the drainage and stabilisation berm as a separate project some time after the liner, subgrade, and subsoil works have been completed.	As above, the liner, subgrade, and subsoil works still provide the step-change reduction in dam safety risk related to the potential failure mode in Figure 0.1. Deferring construction of the berm, means accepting a higher level of dam safety risk in relation to slope instability and deformation than recommended in industry guidelines. However, if construction is only deferred for a few years, the consequences are unlikely to be significant unless a large earthquake or other unusual loading condition happens to occur that triggers more significant consequences. Notwithstanding, the longer the period deferred, the higher the risk such an adverse event will occur.
Undertake no physical works to improve stability. Instead, rely on surveillance and emergency preparedness to mitigate risk.	Comments are as per the approach above. However, if remedial works are deferred indefinitely, there is a much higher likelihood that an adverse event with more significant consequences could occur.

Based on the dam safety considerations above, our recommended approach would be to proceed with the second approach i.e., deferring construction of the berm until after the more urgent dam safety critical work unless construction of the berm can be carried out without delaying the liner, subgrade, and subsoil works.

4.3.2 Upstream slope

Physical interventions to improve the stability performance of the upstream / internal slope of the eastern dam embankment in the OBE would likely be challenging and require dewatering. However, non-compliance for the upstream slope in the OBE is for a sensitivity case that is not confirmed. If confirmed, investigations to address the non-compliance may identify that alternatives to physical intervention are preferred i.e., surveillance, emergency preparedness, or measures to mitigate the risk to water supply.

4.4 Residual areas of uncertainty / risk

We note the following area of uncertainty with respect to the stability, deformation, and seepage performance of the eastern dam embankment:

- Guidance on how to consider the 2022 NSHM seismic hazard in assessment and design of dams is expected to be released in an update to the NZDSG later this year. This may change the recommended industry practice for seismic assessment and performance. This uncertainty has been addressed as a sensitivity case for the purposes of the current report. The assessment for the sensitivity case indicates the non-compliance in the OBE could potentially be worsened and a non-compliance in the SEE could be introduced by considering 2022 NSHM seismic hazard, noting that this will depend on the guidance yet to be published.

4.5 Recommendations for decision-making and further work

Based on dam safety and engineering considerations, we recommend that the non-compliances for stability performance of the eastern dam embankment are addressed as part of routine asset management and renewal processes separate from the urgent remedial works to the liner, subgrade, and subsoil network, because:

- The non-compliances do not relate directly to the higher risk potential failure mode shown in Figure 0.1 that requires urgent remedial work if the dam is to remain in service.
- The non-compliances do not represent a level of risk necessitating emergency intervention.
- More information will be available on the sensitivity case relating to NHSM 2022 when an update to NZDSG is published later in 2024.
- If intervention is confirmed as necessary, this would most likely be for the external slopes. These interventions could most likely be constructed without dewatering the reservoir i.e., would not necessarily benefit from being undertaken while the reservoir was already dewatered for the urgent remedial works.

Addressing the non-compliances remaining following construction of the berm would likely be driven by activities under the Building (Dam Safety) Regulations 2022. This further work is anticipated to comprise developing and implementing a defensible plan to investigate, confirm, and resolve the non-compliances in a timeframe that reflects the level of risk of the non-compliances.

As noted above, the non-compliances do not represent a level of risk necessitating emergency intervention. It is anticipated that the investigations and work to resolve the non-compliances would follow a reasonable asset renewal cycle that would provide time to set aside budget for the works. These works would likely include:

- Resolving the uncertainty relating to NHSM 2022 i.e., updating the assessment of seismic performance (OBE and SEE) once the update to NZDSG is published.
- Once the uncertainty above is resolved, finalising the stability modelling and confirming the scale and risk of the stability non-compliances for the eastern dam embankment.
- This would then inform development and implementation of options to address the non-compliances, which might comprise a combination of surveillance, emergency preparedness, operational measures, and physical works. These investigations may potentially determine that physical works are not necessary.

Further seepage and stability modelling is also recommended as part of completing detailed design for the urgent remedial works:

- Modelling two additional cross sections (one section analysed for this report only).
- Further work as may be required to close out the peer review and building consent processes.

5 Internal erosion

5.1 Internal erosion of the eastern dam embankment, including the foundation, and backfill around the subsoil outlet pipe (issue “c”)

5.1.1 Introduction

The risk of internal erosion due to issue “c” and the potential failure mode shown in Figure 0.1 in the existing situation, prior to remedial works, was assessed as part of the “Stage 1 Geotechnical Interpretative Report and Internal Erosion Assessment” (T+T, 5 September 2023, 1020688.4200 v1). This was based on findings of the Stage 1 ground investigation presented in the “Geotechnical Factual Report” (T+T, 15 May 2023, 1020688.4200 v1).

In this section, the previous internal erosion analysis has been updated to reflect:

1. Results from the Stage 2 geotechnical investigations (refer Section 2).
2. The 25% detailed design for the remedial works, as presented in the “Dannevirke Raw Water Reservoir Remediation Design Report” (V01 25% design stage, T+T September 2023).
3. Interpretation of the Hole Erosion Test (HET) results (refer Section 2.2.3.7 and Appendix E).

The purpose of the analysis is to assess the potential for internal erosion of the dam embankments, their foundations, and the subsoil pipes and outlet pipes, associated with the potential failure mode shown in Figure 0.1 in the situation following remediation (based on the 25% design).

5.1.2 Methodology

This assessment has been conducted following the guidance presented in Fell et al. (2015)[3]. The approach adopted generally conforms to a level of detail in line with an “enhanced engineering judgement approach” / a “risk enhanced engineering judgement approach” as defined in Fell et al. (2015) Section 8.12.6.3; generally defining the potential for initiation, but not looking to define annual probabilities of given mechanisms.

Input data to the assessment is summarised in Section 5.1.3. Assessment of internal erosion by the four mechanisms detailed by Fell et al. (2015) is presented in Sections 5.1.4 to 5.1.7, for concentrated leak erosion, backward erosion, suffusion, and contact erosion respectively.

5.1.3 Input data

A geotechnical interpretation of the site is presented in Section 2. This includes the presentation of laboratory test results of particular importance to the assessment of internal erosion; specifically particle size distribution, Atterberg limits, pinhole dispersion, and HET test results presented in Sections 2.2.3.1, 2.2.3.3, 2.2.3.6, and 2.2.3.7 respectively.

5.1.4 Concentrated leak erosion

5.1.4.1 Initiation assessment

Calculations in Appendix D present an assessment of concentrated leak erosion associated with the subsoil pipes and outlet pipe shown on 25% Detailed Design drawing 1020688.4000-5031 Rev 1.

5.1.4.1.1 Mechanisms

Two potential mechanisms of concentrated leak erosion have been considered; i) along the outlet pipe conduit, and ii) through cracks in the foundation materials or embankment fill due to the conduit trench

[3] R. Fell, P. MacGregor, D. Stapledon, G. Bell and M. Foster, Geotechnical Engineering of Dams, 2nd edition, London, UK: Taylor & Francis Group, 2015.

excavation and backfilling (see Figure 5-1 (b) and (c) respectively). These mechanisms are considered to represent the most likely mechanisms of concentrated leak erosion (excluding earthquake and post-earthquake conditions).

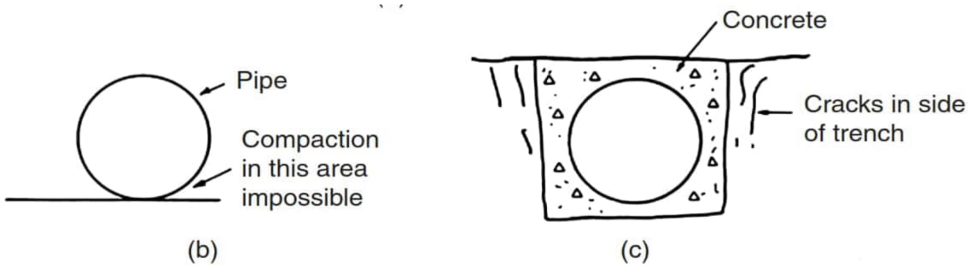

Figure 5-1: Causes of internal erosion around conduits due to (b) Inadequate compaction under the pipe, and (c) cracking in soil in the sides of the trench. *Source: Fell et al (2015) Figure 8.19.*

5.1.4.1.2 Hydraulic conditions

The hydraulic conditions considered in the calculations in Appendix D are presented in Table 5-1. A base case is considered, which represents the likely, long-term situation following remedial works. Several sensitivity cases have also been considered representing unlikely, adverse situations. A key uncertainty is the driving head from the reservoir which is varied between the base and sensitivity cases.

Table 5-1: Hydraulic conditions in base and sensitivity cases for internal erosion assessment

ID	Case	Comments
0	Base case: Normal operating conditions	It is expected that under normal operating conditions “pinhole” seepage through the HDPE/EIA/LLDPE and LPF liner will be insufficient to create a driving head for concentrated leak erosion.
1	Sensitivity case: Elevated leakage rates	Due to unexpected, elevated leakage rates, flow rates into Subsoil Pipe 6 are assumed to be sufficient to create a driving head between the elevation of the subsoil pipe and the existing Ø 900 mm manhole at the toe of the embankment.
2	Sensitivity case: Elevated leakage rates and blocked Subsoil Pipe 5 and 6	As above but assuming Subsoil Pipes 5 and 6 are blocked, resulting in a driving head between the elevation of Subsoil Pipe 4 and the existing Ø 900 mm manhole at the toe of the embankment.
3	Sensitivity case: HDPE/EIA/LLDPE liner damaged and drainage completely blocked	Condition considers a Safety Evaluation Earthquake (SEE) resulting in significant damage to the HDPE/EIA/LLDPE liner and drainage system. The driving head is assumed to be 50% of Maximum Water Level above Subsoil Pipe 5 and 6.
4	Sensitivity case: HDPE/EIA/LLDPE and LPF liner completely compromised, and drainage completely blocked	Condition considers a Safety Evaluation Earthquake (SEE) resulting in the HDPE/EIA/LLDPE liner, LPF liner and drainage system being completely compromised. Full driving head from the reservoir at Maximum Water Level is assumed.

5.1.4.1.3 In-situ soils’ resistance

Another key area of uncertainty relates to the in-situ soils’ resistance to internal erosion. The soil resistance assumed in the calculations in Appendix D is largely based on empirical correlations. Analysis of the HET testing presented in Appendix E has indicated that the resistance of the Embankment Fill and Loess for the samples tested is better than assumed in the empirical correlations. However, some uncertainty remains over the in-situ materials’ resistance due to the natural variability of materials, and variability of resistance with compaction and saturation rate, which could differ from the samples tested. As such, the results and recommendations based on Appendix D are still considered appropriate.

5.1.4.2 Results

The assessment concludes that:

- Under normal operating conditions in the base case (Condition 0) with only "pinhole" seepage through the HDPE/EIA/LLDPE and LPF liner there is expected to be insufficient driving head for concentrated leak erosion to occur. Even under higher leakage rates, if the subsoil drainage is effective at draining the leakage, insufficient driving head for concentrated leak erosion is expected.
- It is only for the sensitivity cases under conditions where leakage rates are assumed to exceed the subsoil drains' capacity to prevent significant pore pressure building up, or where the drains are compromised due to blockage or damage in an earthquake, that the hydraulic conditions for concentrated leak erosion to initiate may develop.
- If these conditions were to be realised, the calculations conducted indicate that initiation of concentrated leak erosion is credible. The likelihood of initiation is higher for the sensitivity cases where it is considered that following an SEE, the HDPE/EIA/LLDPE liner is damaged or completely compromised.
- It should be noted there is significant uncertainty in these calculations; notably in the driving head, leakage hole size and persistence, and the soil's erosive resistance; all to which the calculations are sensitive. Further, these calculations do not assess all possible mechanisms of concentrated leak erosion exhaustively (for example cracking or hydraulic fracture due to arching across the outlet pipe trench). However, the calculations for the two mechanisms are considered to have provided an indication of the susceptibility to concentrated leak erosion in general terms.

5.1.4.2.1 Implications for risk in the existing situation

The risk of concentrated erosion in the existing situation has previously been assessed as presented in the "Stage 1 Geotechnical Interpretative Report and Internal Erosion Assessment" (T+T, 5 September 2023, 1020688.4200 v1). To recap, concentrated leak erosion is relevant to Steps 4, 5, and 6 in the potential failure mode shown in Figure 0.1. There have been no direct observations that concentrated leak erosion is occurring. However, the risk is considered relatively high in the existing situation based on:

- Analysis of available information on materials.
- Construction details.
- Evidence the subsoil drainage network is compromised.
- High risk of further HDPE liner holes and tears.

The results for the sensitivity cases above are relevant for the existing situation because the liner and subsoil drainage system are known to be damaged such that we cannot be certain the hydraulic conditions are as per the base case. Elevated leakage and hydraulic pressures are possible, noting that these may not be reflected in measurements of subsoil drain flows or vibrating wire piezometers i.e., leakage flows could be bypassing the damaged subsoil drainage network, and the installed piezometers may not be positioned where hydraulic pressures are highest. These elevated leakage and hydraulic pressures could potentially be represented by some of the sensitivity cases in Table 5-1 for which initiation of concentrated leak erosion is considered credible.

5.1.4.2.2 Implications for risk following remediation

Following remediation, under normal loading conditions, the risk of concentrated leak erosion will reduce from high to low because the liner and subsoil systems will be repaired to provide certainty that the hydraulic conditions for the base case in Table 5-1 are achieved.

In a seismic event which does not compromise the liner, hydraulic conditions are expected to be similar to under normal loading conditions, and as such are not expected to be sufficient to initiate concentrated leak erosion.

In an event where the liner is compromised, the preliminary transient seepage analysis discussed in Section 3.1.2 and Section 4.1.2 has shown that with the liner removed (representing damage following a SEE) only the first few meters of the dam embankment have time to become saturated if the dam is drawn down over the expected three to four days. This small advance of the wetting front is not expected to be sufficient for concentrated leak erosion associated with the outlet pipe conduit to occur. The risk would be further reduced with the inclusion of the upstream filter blanket (refer Section 5.3.2).

5.1.5 Backward erosion

5.1.5.1 Initiation screening

Fell et al. (2015) Section 8.4.1 reports the following initiation screening criteria for backward erosion:

1. Only non-plastic soils are likely to be subject to backward erosion piping.
2. Backward erosion piping mostly occurs where the eroding soil is fine to medium grain size sand, with a uniformity coefficient, C_u<3.

Considering the laboratory test results presented in Section 2.2.3, a summary of the materials present against these criteria is presented in Table 5-2.

Table 5-2: Initiation screening of susceptibility to backward erosion

Material	Significant proportion of non-plastic strata? (refer Table 2-2)	$C_u = d_{60}/d_{10}$	Susceptible to backward erosion?
TNZ F/2 Filter Material	Yes	9	No
Embankment Fill	Yes	210-960	No
Loess	No	34-74	No
Tamaki Alluvium	Yes	146	No
Makirikiri Alluvium	No	160-600	No
Mangaheia Group	No; not particulate	-	No

Note: For calculation of C_u, where D_{10} was not proven by the particle size distribution test, D_{10}=0.001mm was assumed.

5.1.5.2 Results

Based on the information in Table 5-2, none of the materials present are considered susceptible to backward erosion.

It should be noted that the single Mangaheia Group particle size distribution result (sample AKL438.6) presented in Figure 2-2 and previously analysed for susceptibility to backward erosion in the "Stage 1 Geotechnical Interpretative Report and Internal Erosion Assessment" (T+T, 5 September 2023, 1020688.4200 v1) is not considered here. This is because BH01 in Ground Investigation Stage 2 has shown that the material in situ is not particulate (meaning it is a siltstone / sandstone as opposed to a silt / sand), and consequently it is not considered to be susceptible to backward erosion.

5.1.6 Suffusion

5.1.6.1 Initiation screening

Fell et al. (2015) Section 8.5.2 reports that materials with Plasticity Index > 7 should be considered not subject to suffusion at the gradients usually experienced in dams and their foundations.

A summary of the materials present against this criterion is presented in Table 5-2.

Table 5-3: Initiation screening of susceptibility to backward erosion

Material	Plasticity Index (%) (refer Table 2-2)	Susceptible to suffusion?
TNZ F/2 Filter Material	Non-plastic	No; engineered drainage material
Embankment Fill	Generally non-plastic	See further assessment below
Loess	12-16	No; PI > 7
Tamaki Alluvium	Generally non-plastic	See further assessment below
Makirikiri Alluvium	12-14	No; PI > 7
Mangaheia Group	No; not particulate	No

5.1.6.2 Initiation assessment

The Wan and Fell adaption of the Burenkova method presented in Fell et al. (2015) has been used to assess susceptibility to suffusion. The material gradings presented in Figure 2-2 have been used to plot the probability of internal instability in Figure 5-2. The results are summarised in Table 5-4.

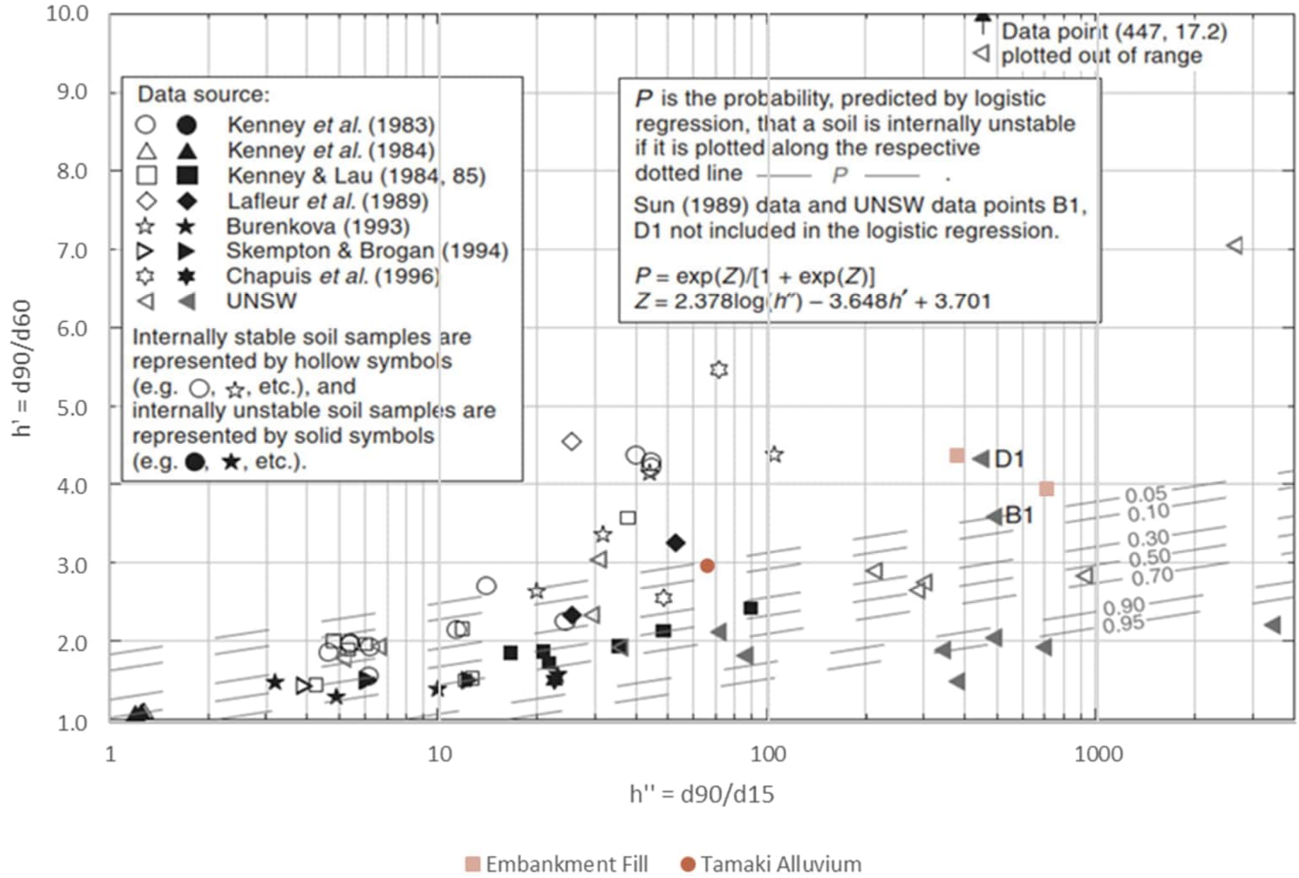

Figure 5-2: Probability of internal instability

Note: Figure is applicable for silt-sand-gravel and clay-silt-sand-gravel mixtures with a plasticity index less than 13% and less than 10% clay size fraction (% passing 0.002 mm).

Table 5-4: Initiation assessment of susceptibility to backward erosion

Material	Probability of susceptibility to suffusion
Embankment Fill	<5%
Tamaki Alluvium	5-10%

5.1.6.3 Critical seepage gradient

Figure 2-2 shows the Embankment Fill and Tamaki Alluvium generally have 10%-15% fines (passing 0.075 mm sieve). Fell et al (2015) Section 8.5.4, suggests critical hydraulic gradients greater than 0.3 may be expected for material of this nature.

The concentrated leak erosion equations presented in Section 5.1.4 show that even for conditions in sensitivity cases where the drainage system is compromised the hydraulic gradients through the embankment are expected to be significantly lower than 0.3. Consequently, suffusion is not expected to initiate.

5.1.6.4 Results

The Embankment Fill and Tamaki Alluvium have a low probability of susceptibility to suffusion.

There is uncertainty in the literature over the magnitude of critical hydraulic gradients for suffusion to occur. From the information available, the actual hydraulic gradients are expected to be lower than the critical hydraulic gradients. Consequently, even if the Embankment Fill and Tamaki Alluvium do contain potentially suffusive material, suffusion is not expected to initiate.

5.1.7 Contact erosion

5.1.7.1 Existing situation

Contact erosion is relevant at interfaces between the subsoil drainage bedding and surrounding finer grained materials, including the LPF liner and natural ground, as per Step 3 in the potential failure mode in Figure 0.1.

The risk of contact erosion in the existing situation has previously been assessed as presented in the "Stage 1 Geotechnical Interpretative Report and Internal Erosion Assessment" (T+T, 5 September 2023, 1020688.4200 v1). The analysis in this previous report confirmed that the existing "40 mm round drainage stone" in the subsoil drains is not filter compatible with the existing LPF liner nor the Mangaheia siltstone. The subsoil drainage material exceeds the "continuing erosion" boundary for retaining the LPF liner and the Mangaheia siltstone.

Contact erosion is also confirmed as progressing based on the depressions and ongoing movement observed in the reservoir floor in inspections by remotely operated vehicles (ROV).

The lack of filter compatibility between the existing subsoil bedding and surrounding materials, which enables contact erosion, is a Confirmed Dam Safety Deficiency.

5.1.7.2 Following remediation of the liner, subgrade, and subsoil network

Following remediation, contact erosion will be avoided and the deficiency will be eliminated by specifying filter compatible materials in line with modern, recommended practice. The remainder of this section presents an assessment of the interim design at the 25% design stage and identifies how this design is proposed to be developed further during the remainder of detailed design to comply with recommended practice.

5.1.7.2.1 Initiation assessment

The potential for contact erosion has been assessed for the 25% design following the filter compatibility criteria in Fell et al. (2015) Section 9.2.4 and 9.3.2.

The assessment is presented in Table 5-5, which:

- Presents for each base (fine) material, the particle size thresholds of adjacent coarse material which would result in some, excessive and continuing erosion. These thresholds are given as values of D15F; the particle size of the coarser soil for which 15% is finer. For each base soil, the thresholds have been calculated from the particle size distribution data presented in Figure 2-2.
- Presents for each coarse material, the coarsest D15F value from the particle size distribution data presented in Figure 2-2 (or for the TNZ F/2 filter material, the coarsest permissible grading specified in TNZ F/2: 2000).
- Describes the location of the design interfaces between different materials.
- Is colour coded to indicate whether a given interface is calculated to experience no, some, excessive or continuing erosion.

5.1.7.2.2 Results

All design interfaces have been shown to meet the no erosion criteria, with the exception of the Makirikiri Alluvium eroding into the TNZ F/2 filter material where a minor exceedance of the no erosion criterion is calculated. This interface is most critical due to the fine grading and dispersive nature of the Makirikiri Alluvium. The exceedance applies at the coarse end of the TNZ F/2 envelope (TNZ F/2 D15F range = 0.200 to 0.522 mm, and the D15F no erosion boundary is 0.5 mm). At this interface where coarse TNZ F/2 material is present, “some” erosion of the Alluvium is predicted to occur, after which the filter is expected to seal.

It should also be noted that whilst the Loess (and derived LPF liner and outlet pipe backfill) is not predicted to experience erosion into the TNZ F/2 filter material, the range of particle size distributions for this material presented in Figure 2-2 indicates it is possible some Loess may be fine enough to experience “some” erosion at this interface.

In both cases, a failure mode is not expected to develop. However, it is anticipated that during future stages of detailed design, the grading for “critical filters” (as defined by Fell et al. (2015) Section 9.1.3 will be specified (revised from the TNZ F/2 envelope) to comply with no-erosion filter criteria in locations relevant to the Makirikiri Alluvium and Loess.

Table 5-5: Filter compatibility assessment following remediation (based on the 25% design)

Fine material						Coarse material			
Material	**Base soil category**	**Dispersive**	**D15F (mm) above which respective levels of erosion occur**			**D15F (mm)**			
			Some erosion	**Excessive erosion**	**Continuing erosion**	**TNZ F/2 drainage filter**	**Embankment Fill**	**Tamaki Alluvium**	**Makirikiri Alluvium**
						0.522	0.075	0.425	0.005
Loess (and compacted clay liner and backfill derived thereof)	1	No	0.637	1.350	1.350	Subsoil drainage; Loess as both compacted clay liner and in situ where the drains are excavated in to Loess (primarily anticipated at the Eastern Embankment).	Eastern Embankment foundation. Outlet pipe; Loess as outlet pipe backfill in surrounding Eastern Embankment Fill	Loess as compacted clay liner where Tamaki Alluvium is at formation. BH02 indicates the Loess cover bed has been removed within the reservoir footprint.	Loess as compacted clay liner where Makirikiri Alluvium is at formation. In situ in Eastern Embankment foundations.
Embankment Fill	2A	No	0.700	2.620	22.997	Subsoil drainage where the drains are excavated in to Embankment Fill (primarily at the Eastern Embankment).	#N/A	Where Embankment Fill directly overlies Tamaki Alluvium; primarily where a relatively thin thickness of fill has been placed to form the north and west upper reservoir slopes.	See opposite coarse-fine relationship for these materials.
Tamaki Alluvium	4A	Assumed Yes	12.392	19.363	38.594	Subsoil drainage where the drains are excavated in to Tamaki Alluvium (primarily anticipated at the north and west reservoir slopes and reservoir floor).	See opposite coarse-fine relationship for these materials.	#N/A	See opposite coarse-fine relationship for these materials.
Makirikiri Alluvium	2A	Yes	0.500	1.550	18.652	Subsoil drainage where the drains are excavated in to Makirikiri Alluvium (primarily anticipated at the upstream toe of the Eastern Embankment).	Where Embankment Fill directly overlies Makirikiri Alluvium; primarily under the Eastern Embankment where in situ Loess is absent.	Possible interface beneath reservoir floor.	#N/A

Key

Cell text describes design interfaces between materials

Cell colours reflect the following erosion conditions

No erosion
Some erosion
Excessive erosion
Continuing erosion

5.1.8 Dam safety risk for issue "c" and the potential failure mode in Figure 0.1

The dam safety risk for issue "c" based on the four internal erosion mechanisms detailed by Fell et al. (2015) has been assessed above and is summarised in . The assessment has primarily focussed on the performance of the 25% design of the remedial works, but implications have also been drawn with respect to risk in the existing situation.

Table 5-6: Assessed dam safety risk for four internal erosion mechanisms

Mechanism	Existing situation	Following remediation
Contact erosion	Known to be occurring Confirmed deficiency relating to lack of filter compatibility between the existing subsoil drainage bedding and surrounding materials.	Very low risk Deficient subsoil drainage bedding to be removed and replaced with filter compatible material specified in accordance with modern, recommended practice.
Backward erosion	Very low risk Materials are not susceptible based on screening criteria.	Very low risk Materials are not susceptible based on screening criteria.
Suffusion	Low risk Embankment fill and Tamaki Alluvium have a <5% and 5-10% probability of being susceptible respectively. There is more uncertainty in the existing situation than following remediation, but hydraulic gradients are still expected to be below a critical value of 0.3.	Very low risk Embankment fill and Tamaki Alluvium have a <5% and 5-10% probability of being susceptible respectively. However, hydraulic gradients following remediation are expected to be well below a critical value of 0.3.
Concentrated leak erosion	Possible risk For concentrated leak erosion to occur, hydraulic pressures must exceed critical values. This requires a defect (i.e., an in situ crack or internal erosion pipe in the ground) of a certain size as well as for high hydraulic pressures to reach that defect from the reservoir. There is no direct evidence that a defect is present (cannot be directly inspected without dewatering), but a defect is considered possible in the ground around the subsoil pipes, outlet pipe, and the outlet pipe trench excavation based on construction details. The liner and subsoil drainage system are known to be damaged such that we cannot be certain that high hydraulic pressures will not reach a possible defect. Based on the above, we cannot be certain that the hydraulic conditions are as per the base case in Table 5-1, and may instead be closer to the sensitivity cases in Table 5-1, for which, initiation of concentrated leak erosion is credible.	Low risk during normal operation The repaired liner and subsoil systems will provide confidence that the hydraulic conditions for the base case in Table 5-1 are achieved i.e., that high hydraulic pressures will not reach any possible in situ cracks / pipes in the ground associated with subsoil pipes, outlet pipe, and the outlet pipe trench excavation.

Refer also to Section 6.2, which discusses the implications of these risks in the context of the potential failure mode in Figure 0.1. Section 6.2 concludes that the existing situation could deteriorate rapidly and require emergency intervention at any time. This conclusion is strongly related to the known occurrence of contact erosion and possible risk of concentrated leak erosion identified in the table above for the existing situation.

5.2 Internal erosion of the eastern dam embankment through cracks induced by a large earthquake

This section addresses a vulnerability relating to internal erosion through the eastern dam embankment, but which is considered a different potential failure mode than shown in Figure 0.1 and separate from issue “c” that has been discussed in the previous section.

In a very large earthquake, such as the SEE, many earthfill dams can settle and spread in the upstream-downstream direction, resulting in longitudinal cracking, and some transverse cracking, typically in the upper part of the embankments.

In the SEE for both the existing situation and following remediation, it is anticipated that the liner system would be grossly damaged but that the subsoil drainage network would be designed to remain functional (functionality of the subsoil drainage network following the SEE will be more likely following remediation). The reservoir is estimated to be completely drawn down over three to four days through the subsoil network based on the hydraulic capacity of the subsoil network. If concentrated leak erosion progressed to a breach and uncontrolled release in these few days, this would represent non-compliance with recommended performance in the SEE.

The preliminary transient seepage analysis discussed in Section 3.1.2 and Section 4.1.2 has shown that with the liner removed (representing damage following a SEE) only the first few meters of the dam embankment have time to become saturated if the dam is drawn down over three days. Consequently, a persistent transverse crack would need to be present for an associated failure mechanism to develop prior to drawdown being completed.

The risk of concentrated leak erosion through potential transverse cracks is considered possible based on current information. Should TDC decide to proceed with the remedial works, further work to estimate the size and depth of transverse cracks is recommended as part of detailed design, which will enable this risk to be quantified.

However, exposure to this risk would only occur in very large, extreme earthquakes where substantial damage is expected (and accepted under recommended practice). Even then, exposure to the risk would only persist for the three to four days required to dewater the reservoir via the subsoil network.

The possible non-compliance related to concentrated leak erosion through transverse cracks in the SEE does not represent an immediate danger or make the dam “dangerous” or “earthquake-prone” under the Building Act 2004. The risk is considered substantially lower than the risk associated with the potential failure mode shown in Figure 0.1 and issue “c”.

5.3 Concept design of potential remedial options for internal erosion

5.3.1 Remedial works needed to address critical risks

The reduction in risk of the potential failure mode shown in Figure 0.1 following remediation that is presented in relates predominantly to the following three components of the 25% design:

- Replacement of subsoil drainage bedding with filter compatible material.
- Reinstatement of the subgrade to support a new liner system.

- Replacement of the existing HDPE and LPF liner with a new liner system.

The components above change the contact erosion risk from "known occurrence" to "very low risk" (Step 1 and 2 of the potential failure mode in Figure 0.1) and the concentrated leak erosion risk from "possible" to "low risk" (Steps 4 to 6 of the potential failure mode in Figure 0.1). As a result, the risk of a dam failure (per the potential failure mode shown in Figure 0.1) is improved from the current situation, where it is possible that an emergency could arise at any time, to a situation where the risk of dam failure is very low except following an extreme adverse event, like a very large earthquake.

The three components listed above should be included as a minimum in the remedial works. They provide a step-change reduction in the level of risk of a dam safety incident, emergency, or failure occurring. This also, in turn, provides a step-change reduction in the risk of the reservoir being unavailable for water supply to Dannevirke.

The risk of other non-compliances that have been discussed in this report (i.e., for stability per Section 3 covering issue "a", Section 4 covering issue "b", and internal erosion following earthquake-induced cracking per Section 5.2) represent risks that are higher than recommended industry practice, but well below thresholds for immediate danger unlike the risk of the potential failure mode in Figure 0.1 that is addressed by the three listed components above.

A preliminary "middle" cost estimate for repair of the liner, subgrade, and subsoil network is $5.1M to $5.9M (refer Appendix H for further detail).

5.3.2 Remedial options to address remaining, lower risks

Residual risks for internal erosion that remain following repair of the liner, subgrade, and subsoil network comprise:

- Concentrated leak erosion through transverse cracks caused by settlement in an earthquake as described in Section 5.2. These cracks would likely be near the dam crest / reservoir rim, possibly on both the eastern and western sides. There is a possible risk of non-compliance with the recommended criterion that the SEE should not lead to a dam failure.
- There is no direct evidence, but it is possible that an existing in situ crack or internal erosion pipe is present in the ground around the subsoil pipes, outlet pipe, and outlet pipe trench excavation through the eastern dam embankment. Concentrated leak erosion through these possible defects (per Steps 4 to 6 of the potential failure mode in Figure 0.1) is considered low risk following repair of the liner and subsoil network, which will prevent high hydraulic pressures reaching the defects. However, the defects (if present) do represent a vulnerability in the dam and increases the criticality of the repaired liner and subsoil systems performing as designed.

As noted above, these residual risks are considered well below thresholds for immediate danger and much lower than the risk associated with issue "c" in the existing situation. Nevertheless, Table 5-7 presents several remedial options to address the residual risks above, which are relevant in terms of longer-term costs to maintain the dam in line with recommended practice.

Table 5-7: Remedial options to address remaining, lower internal erosion risks

Option	Description	Comment
Upstream filter blanket	The upstream filter blanket would comprise a filter sand layer on the internal slopes of the eastern and western sides of the reservoir. The layer would be between the subgrade (bulk fill / in situ ground) of the slopes and the LPF liner. Sketches are included in Appendix G for an upstream filter blanket on the eastern slope only. A preliminary "middle" estimate for the concept is $ 0.7 M to $ 0.8 M, which adds approximately 8% to overall remedial works cost (see Appendix H for detail). The cost would approximately double if the upstream filter blanket was added to both eastern and western sides. An upstream filter blanket would need to be installed at the same time as the remediation of the liner system given its location below the LPF liner.	The upstream filter blanket would reduce the risk of concentrated leak erosion through transverse cracks caused by an earthquake by the following: • The estimated drawdown of the full reservoir in three to four days following gross damage to the liner system in the SEE, assumes that leakage through the rupture can drain into the subsoil network freely. The blanket provides a high-capacity seepage pathway into the subsoil network from the reservoir that is less dependent on the permeability of the subgrade where the rupture occurs. The blanket is more beneficial where the subgrade could be lower permeability, such as in the eastern slope. Keeping the drawdown period short minimises the period of exposure to the risk of erosion through the transverse cracks. Moreover, if the liner leak occurs near a transverse crack generated by the SEE, the drainage blanket provides an alternative flow path so that leakage is not solely concentrated through the crack. • The filter blanket provides material for "crack filler", which can reduce seepage velocities and inhibit initiation and continuation of erosion through transverse cracks. However, this benefit may be limited if the transverse cracks are too large, and the "crack filler" is washed through with limited resistance. The upstream filter blanket would reduce the risk of concentrated leak erosion through possible defects along the subsoil pipe, outlet pipe, and outlet pipe trench excavation by the following: • The filter blanket provides material for "crack filler" for these defects like for the transverse cracks. However, the crack filler action may be more beneficial for the possible in situ defects, which are likely to be smaller in size than the transverse cracks caused by an SEE.
Seepage barrier on crest	This could comprise sheet piles or a diaphragm wall embedded in the dam crest at locations where transverse cracks are predicted.	The seepage barrier would reduce the risk of concentrated leak erosion through transverse cracks caused by an earthquake by spanning the cracks and blocking flow. If seepage barriers do not extend far enough, flows can be concentrated below the barrier or at its lateral extents, which can create high hydraulic gradients that may itself initiate erosion. The seepage barrier option has been considered and discarded because there are no clear impermeable strata to tie into to manage this risk.
Filtered berm	The drainage and stability berm described in Section 4.3.1 to improve stability performance has a secondary benefit in mitigating internal erosion risk. The preliminary "middle" cost estimate for the concept is $ 2.8 M to $ 3.2 M, which adds approximately 35% to overall remedial works cost (see Appendix H for further detail).	A filtered berm acts by capturing particles eroded along the defects and preventing continuation and progression of erosion. It is only effective if it is positioned where seepage is emerging. The filtered berm would not reduce the risk of concentrated erosion through transverse cracks caused by an earthquake because these transverse cracks are likely to be above the top of the berm. The filtered berm may reduce the risk of concentrated leak erosion through possible defects along the subsoil pipe, outlet pipe, and outlet pipe trench excavation if seepage from these defects is emerging through natural ground to the south of the outlet pipe trench, rather than following the trench. The benefit in reducing internal erosion risk is considered a secondary rather than primary purpose of the berm, because it is possible that seepage from these defects would emerge above the berm (for cracks associated with upper benches in the outlet pipe trench) or further to the north (following the outlet pipe trench and pipe alignment).
Downstream filter diaphragm and berm	This option would comprise a vertical sand trench, at the toe of the dam below the 900 mm dia outlet manhole, surrounding the outlet pipe and spanning the outlet pipe trench to key into natural ground to either side. A filtered berm like the option above would be positioned above the diaphragm to contain high hydraulic pressures upstream of the diaphragm. A pipe would need to extend to the stream to enable the diaphragm to drain by gravity. Sketches are included in Appendix G for the concept.	The intent of this option would be to reduce the risk of concentrated leak erosion through possible defects along the subsoil pipe, outlet pipe, and outlet pipe trench excavation by intercepting intragranular seepage, seepage along defects, and particles eroded along defects. The option would not reduce the risk of concentrated erosion through transverse cracks caused by an earthquake because these transverse cracks are likely to be above the top of the berm. The key risk of this option is that seepage and any eroded soil particles may be tracking along a path that bypasses the diaphragm, either emerging above the top of the berm or to either side away from the outlet pipe trench. Further consideration could be given to extending the filter diaphragm to match the extent of the filtered berm above. However, it would still be possible for seepage to emerge above the top of the berm.
Internal filter diaphragm	The internal filter diaphragm would comprise a vertical filter sand trench, positioned near the downstream edge of the dam crest, surrounding the subsoil outlet pipes and spanning the full width of the outlet pipe trench excavation to key into natural ground to either side.	The intent of this option would be to reduce the risk of concentrated leak erosion through possible defects along the subsoil pipe, outlet pipe, and outlet pipe trench excavation by intercepting intragranular seepage, seepage along defects, and particles eroded along defects. Constructability would be challenging. Because of the depth, an innovative technique would be needed such as construction in a bio-polymer slurry trench in which the slurry is biologically absorbed to leave a filter sand trench – further work would be needed to confirm cost and viability. Seepage collected by diaphragms is typically drained by a granular subsoil drain at low elevation, which would also be challenging to install. This option would likely be expensive and intrusive. There is a risk the integrity of the dam could be worsened during the works. The internal filter diaphragm has been considered and discarded because of the significant risks above, which are seen to outweigh dam safety benefits given the presence of defects is not certain, and even if the defects are present, the risk of concentrated leak erosion is low in normal conditions following remediation with the repaired liner and subsoil network operating as designed.
Full rebuild of the eastern dam embankment	Full rebuild of the eastern dam embankment was shown in sketches and previously priced as Option B3 in the "Concept design and preliminary cost estimates for remediation options" report (T+T, July 2023). The preliminary cost for the full rebuild component was $ 12.3 M to $ 14.3 M, which adds approximately 154% to overall remedial works cost (see Appendix H).	The full rebuild option would eliminate the risk of concentrated leak erosion through possible defects along the subsoil pipe, outlet pipe, and outlet pipe trench excavation by removing these defects (if present). The rebuild would follow modern, recommended practice to prevent defects reoccurring. The full rebuild option would be unlikely to eliminate the risk of concentrated leak erosion through transverse cracks caused by an earthquake since existing slopes would remain. This option has been considered and discarded due to the high cost and disruption to water supply, which is seen to outweigh dam safety benefits, given the presence of defects is not certain, and even if present, the risk of concentrated leak erosion is low in normal conditions following remediation with the repaired liner and subsoil network operating as designed.

Table 5-8 provides comment on the dam safety risk of three alternative approaches to manage the residual concentrated leak erosion risks that are not addressed by repair of the liner, subgrade and subsoil network.

Table 5-8: Comparison of approaches to manage remaining, lower internal erosion risks

Approach	Dam safety risk
Defer all remedial options listed in Table 5-7 until after the liner, subgrade, and subsoil works. In the interim, mitigate the unresolved risks by surveillance and emergency preparedness.	The liner, subgrade, and subsoil works provide the step-change reduction in dam safety risk related to the potential failure mode in Figure 0.1. The dam safety risk associated with the deferred options is arguably higher than recommended practice but unlikely to have significant consequences if deferred for only a few years. However, the longer the period deferred, the more likely a large earthquake or other unusual loading condition could trigger more significant consequences. When the reservoir is dewatered for the liner, subgrade, and subsoil works, the internal faces of the reservoir will be inspected to identify defects (if present). If the Table 5-7 options are deferred, this information will be available to inform their design, which may result in a better design and lower dam safety risk longer term. The upstream filter blanket can only practicably be installed at the same time as the liner, subgrade, and subsoil works. If deferred, the opportunity to install the blanket will be lost, which may worsen dam safety risk longer term.
Install the upstream filter blanket at the same time as the liner, subgrade, and subsoil works. Defer all other remedial options listed in Table 5-7. In the interim, mitigate the unresolved risks by surveillance and emergency preparedness.	Comments generally as above. However, this option is considered slightly more favourable for dam safety because it allows for the upstream filter blanket to be installed. The main risk for this approach is if the upstream filter blanket is identified as not needed later but has already been installed. This is considered a cost risk rather than dam safety risk.
Construct several remedial options listed in Table 5-7 at the same time as the liner, subgrade, and subsoil works.	As above, the liner, subgrade, and subsoil works will provide the step-change reduction in dam safety risk related to the potential failure mode in Figure 0.1. However, this approach is considered higher risk for dam safety if design, review, funding, and approval of the Table 5-7 options delays the more dam safety critical works above. The design of the Table 5-7 options will also not be informed by an earlier inspection of the dewatered reservoir for defects, which may result in a less targeted and effective design, in turn resulting in higher dam safety risk long term.

Based on the dam safety considerations above, our recommended approach would be as described in row 2 above i.e., to assess the benefit of the upstream filter blanket further during detailed design, and if confirmed as beneficial, install this blanket at the same time as repairing the liner, subgrade, and subsoil network.

5.4 Residual areas of uncertainty / risk

We note the following areas of uncertainty and risk with respect to the internal erosion assessment:

- The presence and size of existing cracks or internal erosion pipes associated with the subsoil pipes, outlet pipe, and outlet pipe trench excavation are unable to be verified by direct inspection. The risk of concentrated leak erosion is highly sensitive to the size of cracks and internal erosion pipes. This uncertainty will reduce if the upstream face of the eastern dam embankment is able to be directly inspected when the reservoir is dewatered for repair of the lining system, subgrade, and subsoil network. However, uncertainty will still remain over the presence and size of cracks along the full length of the buried pipes.

- The hydraulic conditions and reservoir leakage flows in the existing situation are not able to be directly monitored and thus remain uncertain. Leakage flows could be bypassing the damaged subsoil drainage network, and the installed piezometers may not be positioned where hydraulic pressures are highest. The uncertainty has been addressed for the purposes of the current report by considering a range of sensitivity cases. This risk will reduce following remediation, because the repaired liner and subsoil system will provide confidence in the hydraulic conditions and leakage flows reaching possible existing cracks or internal erosion pipes in the eastern dam embankment.
- There is uncertainty over the in-situ soils' resistance to internal erosion even following the interpretation of HET testing as described in Section 5.1.4.1.3. The residual uncertainty relates to the natural variability of materials, and variability of resistance with compaction and saturation rate.
- The size, depth, and continuity of any transverse cracks that could develop during large earthquakes, such as the SEE, is uncertain.

5.5 Recommendations for decision-making and further work for internal erosion risk

5.5.1 Further work

The following further work is recommended:

- When the reservoir is dewatered, and HDPE liner is removed for construction of the remedial works:
  - Detailed inspection and mapping of the exposed ground should be undertaken to capture as much information as possible on the presence and size of any existing cracks and internal erosion pipes, especially associated with the subsoil pipes, outlet pipe, and outlet pipe trench excavation.
- During the remainder of detailed design of the remedial works:
  - The size and depth of any transverse cracks that could develop during the OBE and SEE should be estimated. This may be informed by updates to the NZDSG, covering interpretation of NHSM 2022 seismic hazard, which are scheduled to be published in 2024. The assessment of the likelihood of concentrated erosion leakage through these transverse cracks in Section 5.2 should then be updated.
  - The proposed filter drainage particle grading should be refined to meet no erosion criteria (refer Section 5.1.7). The permissible perforation size in the drainage subsoil pipes should also be verified.

5.5.2 Recommendations to inform TDC's decision-making

Based on the dam safety and engineering considerations in this section, we recommend that the urgent remedial works should include as a minimum:

- Replacement of subsoil drainage bedding with filter compatible material.
- Reinstatement of the subgrade to support the new liner system.
- Replacement of the existing HDPE and LPF liner with a new liner system.

We also recommend that installation of an upstream filter blanket (to mitigate internal erosion risk) is assessed further during detailed design of the liner, subgrade, and subsoil network repair. This is because the upstream filter blanket would be installed under the repaired liner. The opportunity to install the upstream filter blanket will be lost if not constructed when the liner is repaired.

We recommend that other remedial options, such as the filtered berm and downstream filter diaphragm and berm, are deferred and considered as part of routine asset management and renewal processes separately from the liner, subgrade and subsoil network repairs because:

- The non-compliances / vulnerabilities these options address do not represent a level of risk necessitating emergency intervention.
- The options can be constructed without dewatering i.e., would not benefit from being undertaken while the reservoir was already dewatered for the urgent remedial works required to the liner, subgrade and subsoil drains.
- Inspections and mapping of possible defects in the internal faces of the reservoir will be completed when the reservoir is dewatered to repair the liner, subgrade, and subsoil drains, which will provide information relevant to the design of the remedial options.

We expect that addressing internal erosion vulnerabilities and non-compliances will be required under activities under the Building (Dam Safety) Regulations 2022. This further work is anticipated to comprise developing and implementing a defensible plan to investigate, confirm, and resolve the non-compliances in a timeframe that reflects the level of risk of the non-compliances. We note that further investigations may find that surveillance and emergency preparedness are a more pragmatic way to manage the risk of the deficiency than physical interventions.

6 Urgency of remediating the existing liner, subgrade, and underlying subsoil network (issue "d")

The urgency of remediating the existing liner, subgrade, and underlying subsoil network relates directly to the potential failure mode presented in Figure 0.1.

6.1 Review of ROV reports and surveillance data

We have reviewed the urgency of the remedial works based on consideration of:

- ROV reports – refer detailed review in Appendix F.
- Surveillance data – refer Appendix B for rainfall, reservoir level, subsoil outlet flow, and vibrating wire piezometer data.

The piezometer data indicates that the phreatic surface at BH01, located in the eastern dam embankment, has been gradually reducing over time. The phreatic surface at BH02, in the western reservoir rim, has been elevated for periods, specifically in October 2023 and most recently in July 2024. The subsoil outlet flows reduced dramatically following the temporary repair works on 13 to 18 June 2023, and appear to be relatively stable between 2 to 3.5 L/s.

However, the remedial works are still considered urgent due to the ongoing movement of the ground beneath the HDPE liner that has been observed at every ROV inspection since the June 2023 temporary repairs, including the latest inspection on 8 March 2024. This movement has been ongoing despite the reduced leakage indicated by the measured subsoil outlet flows and despite the reservoir level being operated at lower levels. We note that:

- The scale of movement that has been observed at each ROV inspection is considered significant, even the 1 m by 1 m by 350 mm deep depression that had formed at the southern edge of Depression 2 at the last inspection. The LPF liner is only 300 mm thick over most of the impoundment, and locally up to 800 mm thick immediately above each subsoil drain. Thus, a 300 mm depression can reflect complete loss of the low permeability liner depending on the location.
- The ongoing ground movements indicate that the deterioration of the LPF liner and subgrade, via the internal erosion mechanism through the subsoil drain network (contact erosion associated with Step 3 of the potential failure mode in Figure 0.1), is continuing to progress even with the reduction in subsoil flows since the June 2023 temporary repairs.
- It is possible that the ground movements are slowing based on smaller changes seen at the last two inspections, but there are insufficient data points to be confident in this trend. Moreover, internal erosion mechanisms can progress in steps, with alternating periods of activity and inactivity.

6.2 Interpretation in the context of the potential failure mode

Contact erosion associated with Step 3 of the potential failure mode in Figure 0.1 is observed to be occurring as described in Section 6.1. Therefore, the current safety of the dam depends on further holes and tears in the HDPE liner not developing (Steps 1 and 2) and concentrated leak erosion through the eastern dam embankment and foundation not occurring (Steps 4 to 6).

In the existing situation, the risk of new leaks is considered high (Steps 1 and 2), either due to failure of the temporary liner patches installed in June 2023 or failure of the original HDPE liner due to the ongoing deterioration of the supporting subgrade as indicated by the ROV inspections described above. The risk of concentrated leak erosion (Steps 4 to 6) is also considered credible in the existing situation as discussed in Section 5.

6.3 Recommendations

Based on the above, it is still considered possible that the existing situation could deteriorate rapidly and require emergency intervention at any time due to the potential failure mode in Figure 0.1. It is noted that a dam safety emergency would likely require dewatering and take the reservoir out of service for water supply to Dannevirke.

Our recommendations based on dam safety and engineering considerations are:

- Continue with current measures to mitigate the risk of the potential failure mode in Figure 0.1, including:
  - Ongoing enhanced surveillance; and
  - Maintaining preparedness to implement TDC's interim emergency action plan.
- Prepare to remove the risk as soon as practicable either by repairing the liner, subgrade, and subsoil network (as recommended in 5.5.2) or decommissioning the dam.

7 Conclusions

7.1 Introduction

This report presents T+T's professional advice on four selected issues relating to dam performance:

a Stability of the western reservoir rim.

b Stability of the eastern dam embankment.

c Potential for internal erosion of the eastern dam embankment, including dam foundation, and backfill around the subsoil outlet pipe.

d Urgency of remediating the existing liner, subgrade, and underlying subsoil network.

Advice is also provided on the risk of internal erosion through the eastern dam embankment through cracks induced by a large earthquake, which is a separate and additional issue that has been identified during this study.

Issues "c" and "d" relate directly to a Confirmed Dam Safety Deficiency comprising a lack of filter compatibility between the existing subsoil bedding and surrounding materials, and the related potential failure mode that has been presented in Figure 0.1.

The scope of work has been developed in conjunction with TDC and the peer reviewer and is intended as one of the inputs to TDC's decisions on what issues should be addressed by the remedial works, whether to proceed with remedial works or pursue alternative water supply options and decommission the dam, and timing of when risks related to the potential failure mode identified at the dam should be resolved.

7.2 Stability of the western reservoir rim and eastern dam embankment (issues "a" and "b")

Our conclusions for issues "a" and "b" comprise:

- Stability performance has been assessed for base cases, which represent current design scenarios, and for sensitivity cases, where there is known uncertainty.
- Stability performance of the internal slopes of the reservoir is compliant across most of the assessed cases, except in the OBE for one of the sensitivity cases.
- Stability performance of the external slope of the western reservoir rim, which encompasses the access track to the reservoir and treatment plant, has a marginal non-compliance in the OBE (<20 mm) in the base case, which worsens for the sensitivity cases. Options for physical works to remove this non-compliance could include installation of shear piles, drainage, and / or a stabilisation berm. These options are not unequivocally needed and development of cost estimates for these options is excluded from the scope of the current report.
- Stability performance of the external slope of the eastern dam embankment, is non-compliant in the long-term stability, OBE, and post-earthquake base cases, which worsens for sensitivity cases. Performance in the SEE is also non-compliant for one of the sensitivity cases. A preliminary design of a drainage and stability berm has been developed, which would remove the non-compliance in the long-term static stability and post-earthquake cases. An approximate "middle" cost estimate for the preliminary berm design is $ 2.8 M to 3.2 M (refer Appendix H for detail). A larger berm, and/or deep shear key or shear piles, would be required to remove the non-compliance in the OBE and the possible (unconfirmed) non-compliance in the SEE for the sensitivity case.
- All the non-compliances associated with stability performance are well below thresholds for immediate danger or that would make the dam "dangerous", "earthquake-prone", or

"flood-prone" under the Building Act 2004. The risk of the non-compliances is considered significantly lower than the risk associated with the Confirmed Dam Safety Deficiency and potential failure mode presented in Figure 0.1.

- The assessment of seismic performance (OBE and SEE) should be updated when the guidance on interpretation of NHSM 2022 for dams is published in a revision to NZDSG later this year.

7.3 Internal erosion of the eastern dam embankment, including dam foundation, and backfill around the subsoil outlet pipe (issue "c")

As already noted, issue "c" relates directly to the Confirmed Dam Safety Deficiency and the potential failure mode shown in Figure 0.1. Our conclusions for issue "c" comprise:

- Four types of internal erosion have been considered; contact erosion, backward erosion, suffusion, and concentrated leak erosion.
- Backward erosion and suffusion have been assessed as relatively low risk both in the existing situation and following remediation.
- Contact erosion relates to Step 3 in the potential failure mode in Figure 0.1. This type of internal erosion is known to be occurring in the existing situation. However, the risk will change to very low following remediation by replacing the deficient subsoil bedding material with filter compatible materials specified based on modern, recommended practice.
- Concentrated leak erosion is relevant to Steps 4, 5, and 6 in the potential failure mode in Figure 0.1. This type of internal erosion is considered possible in the existing situation because the damaged liner and subsoil systems could lead to high hydraulic pressures reaching in situ cracks and / or internal erosion pipes in the ground around the subsoil pipes, outlet pipe, and outlet pipe trench through the eastern dam embankment. Following remediation, the risk will change to low under normal conditions because the repaired liner and subsoil systems will prevent high hydraulic pressures reaching the in situ cracks and / or internal erosion pipes.
- Based on the points above, the risk related to internal erosion through and/or under the eastern dam embankment is largely alleviated under most loading conditions by repairing the liner, subgrade, and subsoil network. As will be discussed further in Section 7.5, it is currently considered possible that the existing situation could deteriorate rapidly to an emergency. The repair of the liner, subgrade, and subsoil network is expected to reduce the risk of such an emergency arising from possible to very low.
- A residual risk related to the potential failure mode shown in Figure 0.1 will remain following the liner, subgrade, and subsoil network repair. This comprises the unconfirmed, but possible presence of in situ cracks and / or internal erosion pipes in the ground around the subsoil pipes, outlet pipe, and outlet pipe trench. If these defects are present, the risk of concentrated leak erosion is still low provided the repaired liner and subsoil systems are functioning as designed. However, the defects could be considered a vulnerability, and do increase reliance on the repaired liner and subsoil systems.

  These remaining internal erosion risks following the liner, subgrade, and subsoil network repair are considered below thresholds for immediate danger or that would make the dam "dangerous" or "earthquake-prone" under the Building Act 2004.
- Possible options that could mitigate this residual risk comprise:
  - An upstream filter blanket – an approximate "middle" cost estimate for the preliminary design is $ 0.7 M to $ 0.8 M (refer Appendix H for detail). The upstream filter blanket is located under the liner system so would need to be constructed at the same time as the liner, subgrade, and subsoil network repairs.

- A filtered berm – the drainage and stability berm to improve stability of the eastern dam embankment could also serve as a filtered berm to mitigate the risk of internal erosion.
- A downstream filter diaphragm and berm.

The pathway that seepage and eroded particles may track through / from the possible defects in the ground around the subsoil pipes, outlet pipe, and outlet pipe trench is uncertain, especially downstream of the 2050 mm manhole. There is a risk that the last two options above may not be effective if seepage and eroded particles bypass the devices.

7.4 Internal erosion through the eastern dam embankment through cracks induced by a large earthquake

During the current study, we have identified that in very large earthquakes, concentrated leak erosion through transverse cracks is a possible risk, even following remedial works, and represents a possible (unconfirmed) non-compliance in the SEE. Although this vulnerability involves internal erosion through the eastern dam embankment, this is a different potential failure mode than shown in Figure 0.1 and is substantially lower risk than issue “c”. This risk is considered well below thresholds for immediate danger or that would make the dam “dangerous” or “earthquake-prone” under the Building Act 2004.

The upstream filter blanket described in Section 7.3 would mitigate the risk of concentrated leak erosion through the transverse cracks generated by an SEE in addition to mitigating the risk related to the potential failure mode shown in Figure 0.1 as described in the previous section.

7.5 Urgency of remediating the existing liner, subgrade, and underlying subsoil network (issue “d”)

The urgency of remediating the liner, subgrade and subsoil network relates directly to the potential failure mode shown in Figure 0.1. Our conclusions for issue “d” comprise:

- ROV inspections indicate that deterioration of the LPF liner and subgrade is continuing to progress (Step 3 of the potential failure mode) even with the reduction in subsoil flows since the June 2023 temporary repairs.
- Therefore, the current safety of the dam depends on holes and tears in the HDPE liner not developing or enlarging (Steps 1 and 2) and concentrated leak erosion through the eastern dam embankment and foundation not occurring (Steps 4 to 6).
- The risk of new leaks is considered high in the existing situation, either due to failure of the temporary liner patches installed in June 2023 or failure of the original HDPE liner due to the ongoing deterioration of the supporting subgrade.
- The risk of concentrated leak erosion through defects in the eastern dam embankment and foundation is considered possible in the existing situation as described in Section 7.3 above.
- Based on the above, the potential failure mode in Figure 0.1 is still considered the greatest and most urgent risk for dam safety. As already noted, it is possible that the existing situation could deteriorate rapidly and require emergency intervention at any time, which would likely require dewatering and take the reservoir out of service for water supply to Dannevirke. The longer the repair of the liner, subgrade, and subsoil network is deferred, the greater the risk that an emergency could arise.

7.6 Recommendations to support key decisions

Based on the dam safety and engineering considerations above, our recommendations are:

i Continue with current measures to mitigate the risk of the potential failure mode in Figure 0.1, including:
- Ongoing enhanced surveillance; and
- Maintaining preparedness to implement TDC's interim emergency action plan.

ii Prepare to remove the risk associated with the potential failure mode in Figure 0.1 as soon as practicable either by repairing or decommissioning the dam.

iii If TDC decide to proceed with repair rather than decommissioning, these works should include remediating the liner system, subgrade, and subsoil network as a minimum.

iv Based on the level of dam safety risk, works to address the non-compliances for the stability and internal erosion performance of the western reservoir rim and eastern dam embankment (meaning those non-compliances remaining following the liner, subgrade, and subsoil network repair) could reasonably be deferred and undertaken as part of routine asset management and renewal cycles.

v However, installing the upstream filter blanket, which addresses internal erosion risk in the eastern dam embankment, should be assessed further during detailed design of the liner, subgrade, and subsoil network repairs. This exception is recommended because the upstream filter blanket would be located under the liner so the opportunity to install the blanket will be lost if not constructed at the same time as repairing the liner.

vi These conclusions should be reviewed when guidance on NSHM 2022 is published in NZDSG later in 2024.

When deciding whether to proceed with remedial works or pursue alternative water supply options and decommission the dam, TDC should also be aware that decommissioning will require a Building Consent as a "Large dam" and could potentially involve significant physical works such as redirecting inlet and outlet pipes, and modifying the reservoir to provide certainty that local rainfall cannot build up to form a pond. As part of decision-making, we also recommend that TDC consider the timeline for developing alternative water supply options and implications for duration of exposure to the current dam safety and water supply risk related to the potential failure mode in Figure 0.1.

8 Applicability

This report has been prepared for the exclusive use of our client Tararua District Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

The construction rates utilised for this high level cost estimate are based on assumed design concepts, estimated quantities and a combination of recently submitted tender rates for similar projects within the regional area along with the latest available rates from QV Cost Builder database (formerly Rawlinsons). These rates are based on historic information and data and do not include allowance for any cost escalation since the date of the data other than where/as specifically stated.

Consequently, a significant margin of uncertainty exists on the cost estimate and the contingency we have allowed should be considered as part of the cost rather than a potential add on.

In particular, we have not made any attempt to allow for the potential impact of COVID-19 in this estimate. Also, supply chain disruptions are currently having quickly-changing effects on construction costs and schedules. We recommend you seek up-to-date specialist economic advice on what budgetary allowances you should make for escalation, including for any potential changes in construction costs and timing in relation to both COVID-19 and supply-chain issues.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:

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Dewi Knappstein
Business Leader - Dams

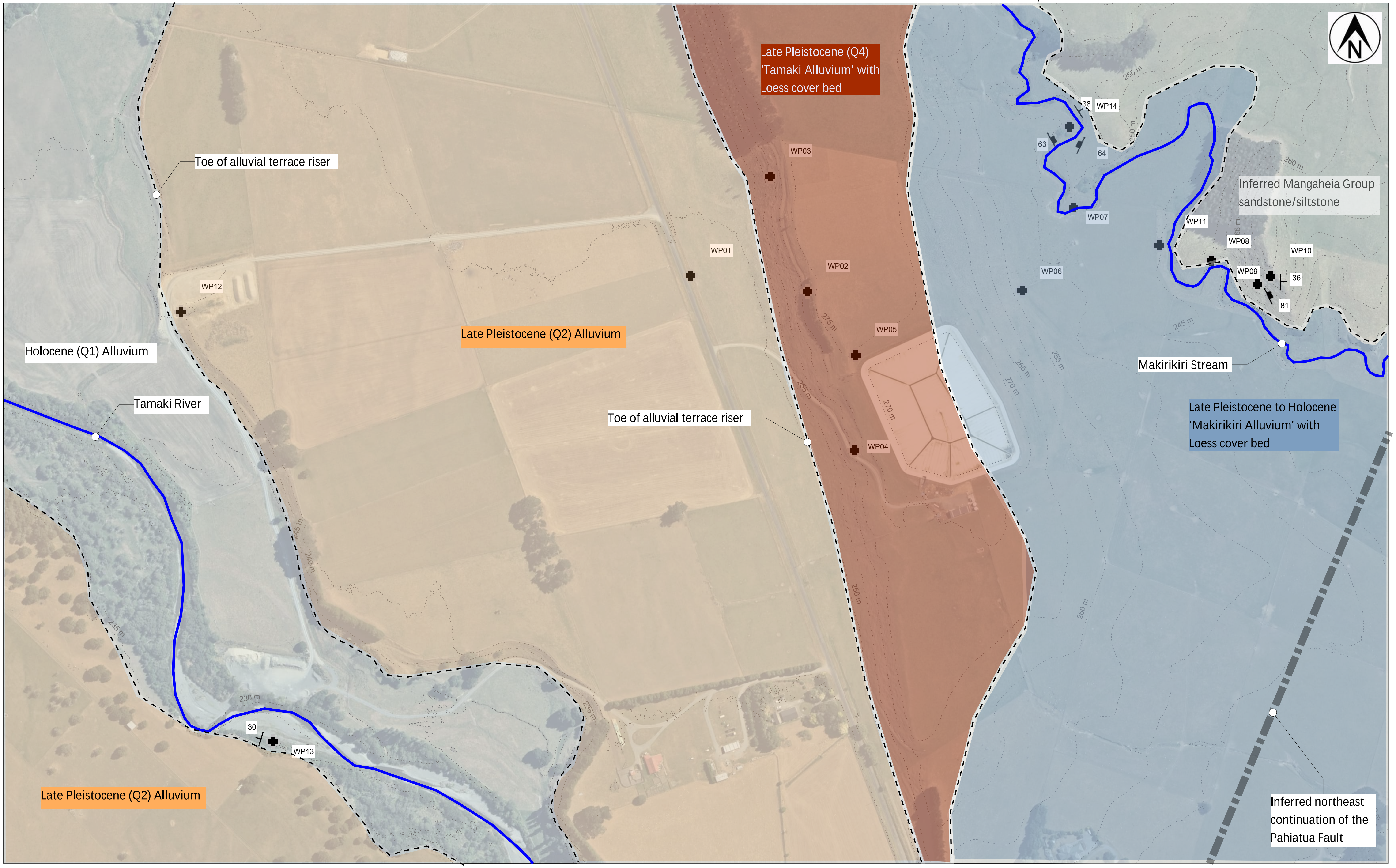
Authorised for Tonkin & Taylor Ltd by:

....

Hugh Cherrill
Project Director

DMK
\\ttgroup.local\corporate\wellington\tt projects\1020688\1020688.6000\workingmaterial\01 report\dannevirke raw water reservoir - stg 2 geo interp rep.v2.docx

Appendix A Ground model



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NOTES:
1. Aerial image Manawatu-Whanganui 3m rural aerial photos 2021-2022. Source: LINZ Data Service.

REV	DESCRIPTION	GIS	CHK	DATE
1	STAGE 2 GEOTECHNICAL INTERPRETATION REPORT	-	-	27/6

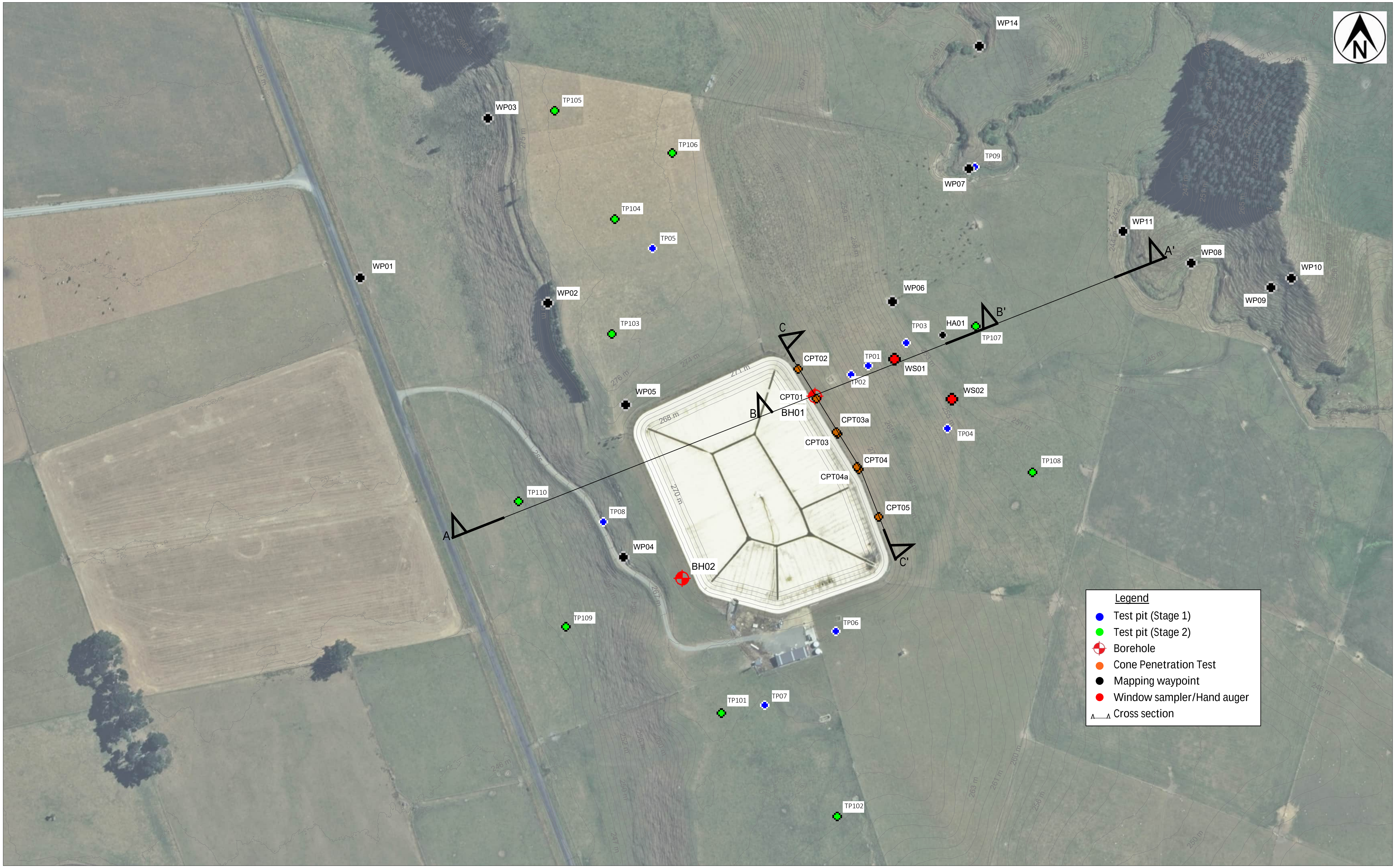
LOCATION PLAN

PROJECT No. 1020688.6000

DESIGNED	-	-
DRAWN	TH	27/6
CHECKED	DMK	12/7

APPROVED DATE

CLIENT TARARUA DISTRICT COUNCIL
PROJECT DANNEVIRKE RAW WATER RESERVOIR
TITLE STAGE 2 INTERPRETATION REPORT
GEOLOGICAL MAP AND MAPPING OBSERVATIONS
SCALE (A3) 1:4,000 FIG No. A1 REV 1



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NOTES:

1. Aerial image Manawatu-Whanganui 3m rural aerial photos 2021-2022. Source: LINZ Data Service.
2. 1m contours shown from Kumeti Digital Elevation Model, 2022. Source: Commissioned by Horizons Regional Council, and supplied by TDC for the purposes of the project.

REV	DESCRIPTION	GIS	CHK	DATE
1	STAGE 2 GEOTECHNICAL FACTUAL REPORT	-	-	28/11
2	STAGE 2 GEOTECHNICAL INTERPRETATION REPORT	-	-	27/6

LOCATION PLAN APPROVED DATE

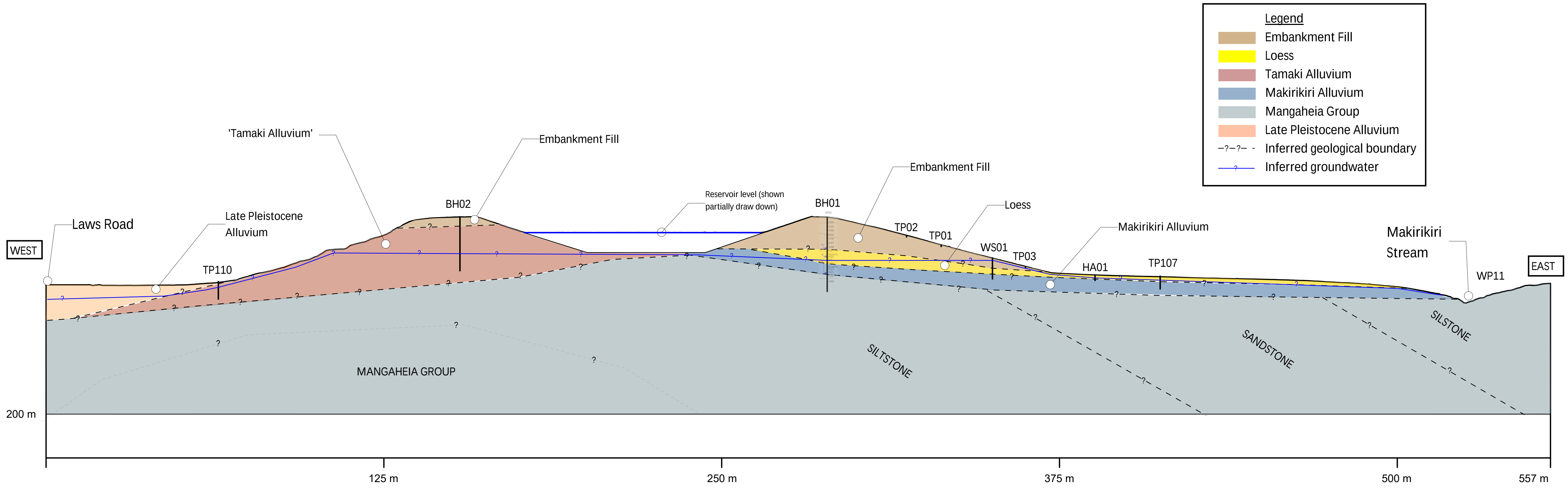
PROJECT No.	1020688.6000	
DESIGNED	-	-
DRAWN	TH	27/6
CHECKED	DMK	12/7

CLIENT TARARUA DISTRICT COUNCIL

PROJECT DANNEVIRKE RAW WATER RESERVOIR

TITLE STAGE 2 INTERPRETATION REPORT
INVESTIGATION LOCATION PLAN

SCALE (A3) 1:2,500 FIG No. A2 REV 2



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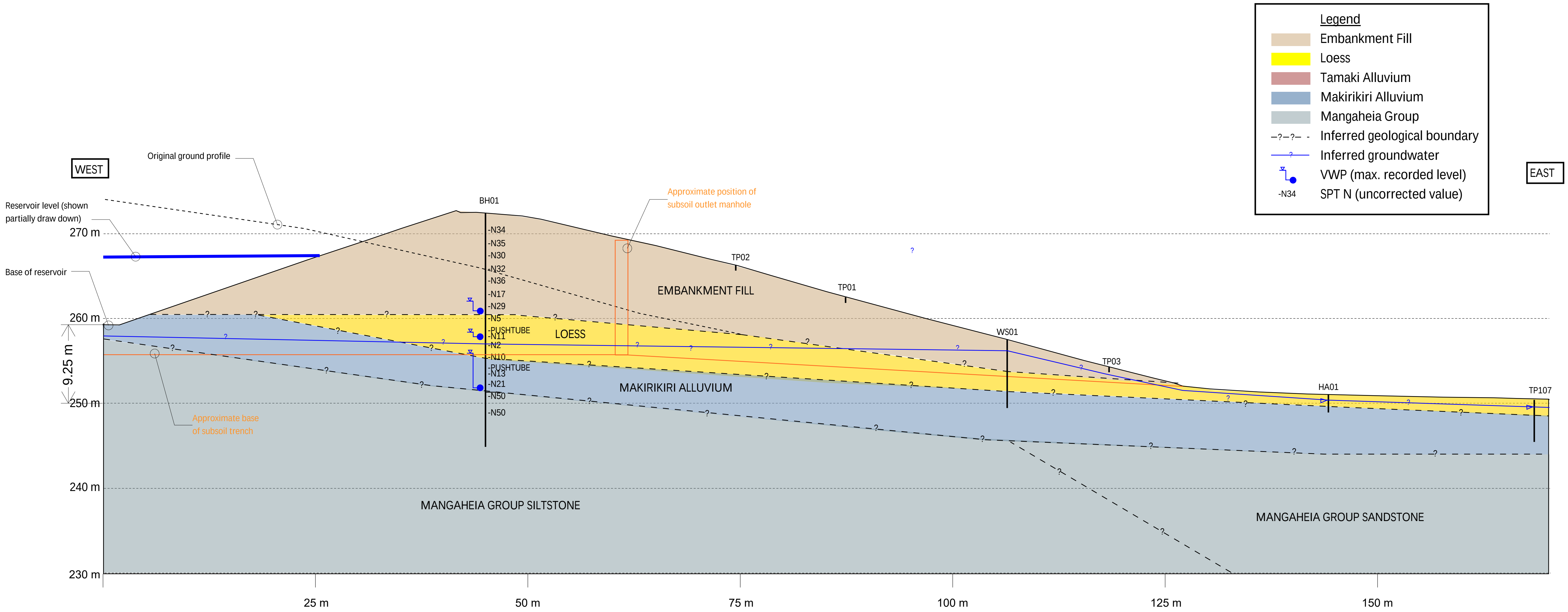
NOTES:
1. The ground surface is based on the Kumeti Digital Elevation Model, 2022. Source: Commissioned by Horizons Regional Council, and supplied by TDC for the purposes of the project.
2. The presented geological model is inferred and is based on limited subsurface information. It should be appreciated that actual conditions may vary from what is shown in this figure.

REV	DESCRIPTION	GIS	CHK	DATE
1	STAGE 2 GEOTECHNICAL INTERPRETATION REPORT	-	-	27/6

LOCATION PLAN

PROJECT No.	1020688.6000	
DESIGNED	-	-
DRAWN	TH	27/6
CHECKED	DMK	12/7
APPROVED		DATE

CLIENT **TARARUA DISTRICT COUNCIL**
PROJECT **DANNEVIRKE RAW WATER RESERVOIR**
TITLE STAGE 2 INTERPRETATION REPORT
GEOLOGICAL LONG SECTION A-A'
SCALE (A3) 1:1,500 FIG No. A3 REV 1



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NOTES:
1. The ground surface is based on the Kumeti Digital Elevation Model, 2022. Source: Commissioned by Horizons Regional Council, and supplied by TDC for the purposes of the project.
2. The presented geological model is inferred and is based on limited subsurface information. It should be appreciated that actual conditions may vary from what is shown in this figure.

REV	DESCRIPTION	GIS	CHK	DATE
1	STAGE 2 GEOTECHNICAL INTERPRETATION REPORT	-	-	27/6

LOCATION PLAN

PROJECT No. 1020688.6000

DESIGNED	-	-
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CHECKED	DMK	12/7

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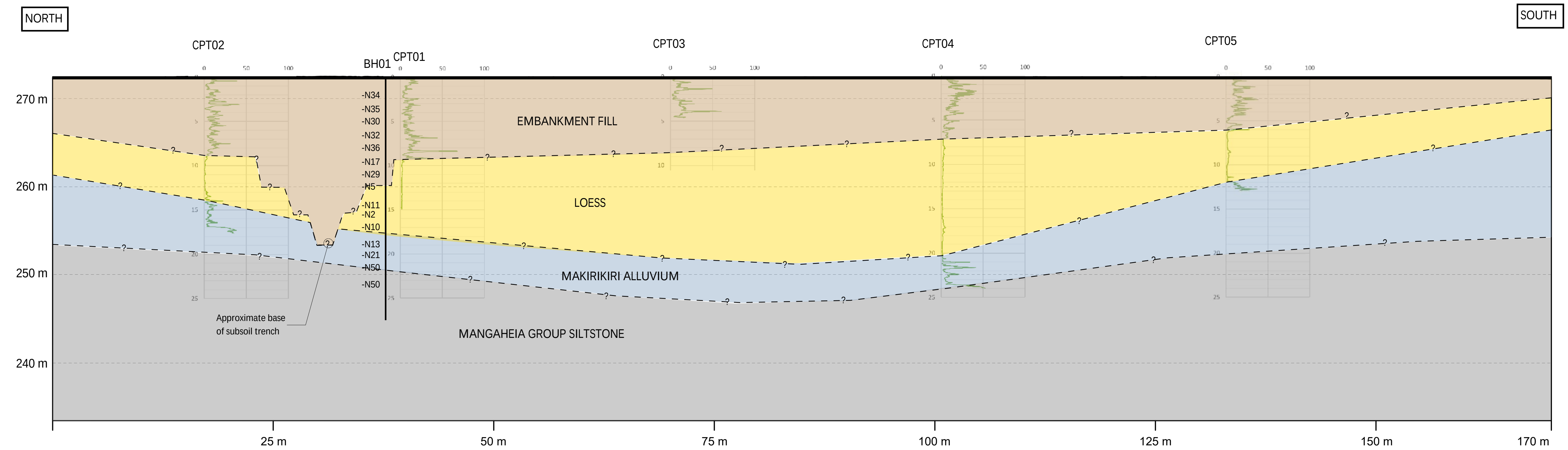
CLIENT TARARUA DISTRICT COUNCIL
PROJECT DANNEVIRKE RAW WATER RESERVOIR
TITLE STAGE 2 INTERPRETATION REPORT
GEOLOGICAL CROSS SECTION B-B'
SCALE (A3) 1:500 FIG No. A4 REV 1

Legend
- Embankment Fill
- Loess
- Tamaki Alluvium
- Makirikiri Alluvium
- Mangaheia Group
- –?–?– - Inferred geological boundary
- -N34 SPT N (uncorrected value)



NOTES:
1. The ground surface is based on the Kumeti Digital Elevation Model, 2022. Source: Commissioned by Horizons Regional Council, and supplied by TDC for the purposes of the project.
2. The presented geological model is inferred and is based on limited subsurface information. It should be appreciated that actual conditions may vary from what is shown in this figure.

REV	DESCRIPTION	GIS	CHK	DATE
1	STAGE 2 GEOTECHNICAL INTERPRETATION REPORT	-	-	27/6

LOCATION PLAN

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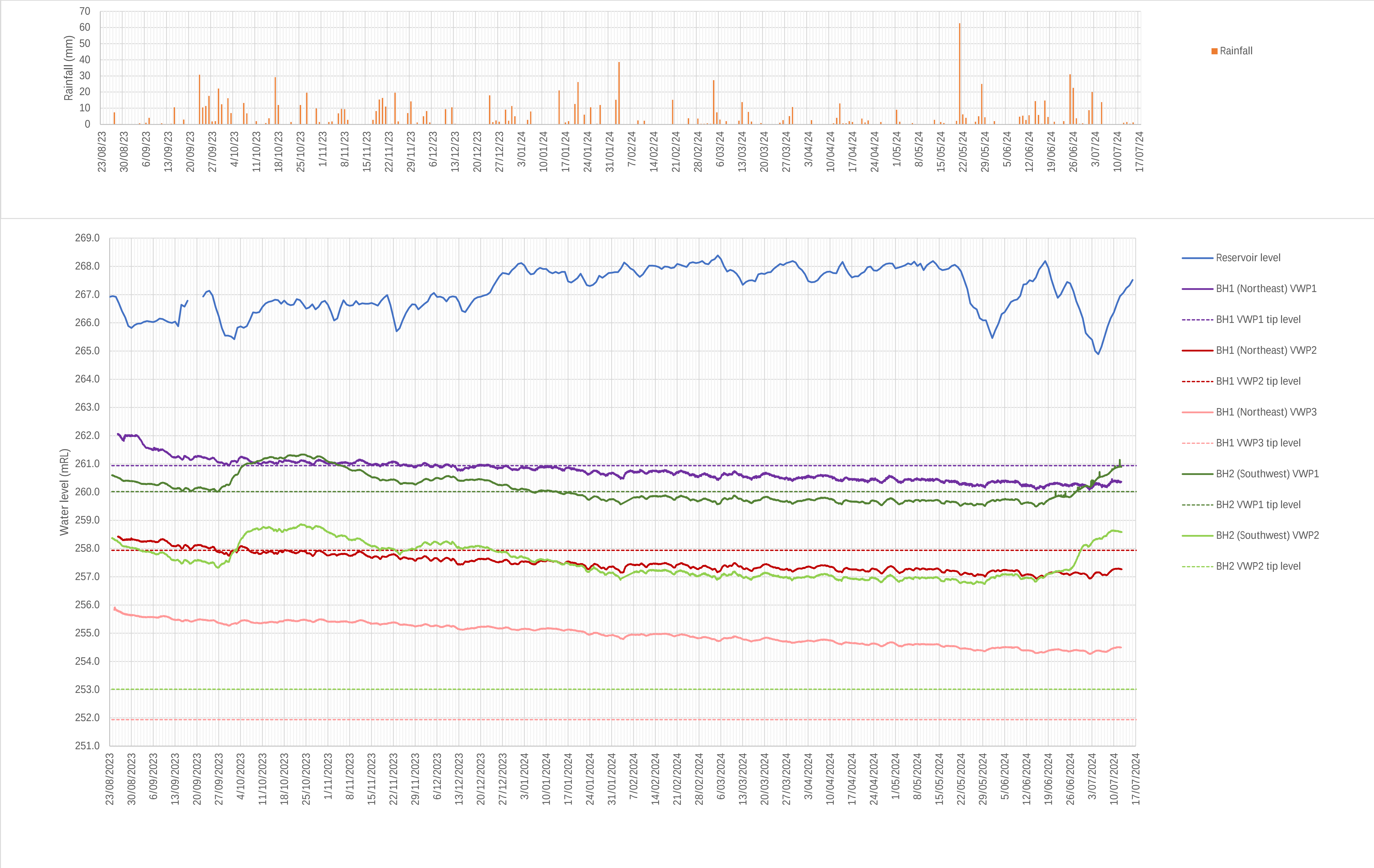
PROJECT No.	1020688.6000	
DESIGNED	-	-
DRAWN	TH	27/6
CHECKED	DMK	12/7
APPROVED		DATE

CLIENT TARARUA DISTRICT COUNCIL
PROJECT DANNEVIRKE RAW WATER RESERVOIR
TITLE STAGE 2 INTERPRETATION REPORT GEOLOGICAL LONG SECTION C-C'
SCALE (A3) 1:500 FIG No. A5 REV 1

Appendix B Surveillance data



Rainfall (mm)
70
60
50
40
30
20
10
0
Rainfall
23/08/23
30/08/23
6/09/23
13/09/23
20/09/23
27/09/23
4/10/23
11/10/23
18/10/23
25/10/23
1/11/23
8/11/23
15/11/23
22/11/23
29/11/23
6/12/23
13/12/23
20/12/23
27/12/23
3/01/24
10/01/24
17/01/24
24/01/24
31/01/24
7/02/24
14/02/24
21/02/24
28/02/24
6/03/24
13/03/24
20/03/24
27/03/24
3/04/24
10/04/24
17/04/24
24/04/24
1/05/24
8/05/24
15/05/24
22/05/24
29/05/24
5/06/24
12/06/24
19/06/24
26/06/24
3/07/24
10/07/24
17/07/24
Water level (mRL)
269.0
268.0
267.0
266.0
265.0
264.0
263.0
262.0
261.0
260.0
259.0
258.0
257.0
256.0
255.0
254.0
253.0
252.0
251.0
Reservoir level
BH1 (Northeast) VWP1
BH1 VWP1 tip level
BH1 (Northeast) VWP2
BH1 VWP2 tip level
BH1 (Northeast) VWP3
BH1 VWP3 tip level
BH2 (Southwest) VWP1
BH2 VWP1 tip level
BH2 (Southwest) VWP2
BH2 VWP2 tip level
23/08/2023
30/08/2023
6/09/2023
13/09/2023
20/09/2023
27/09/2023
4/10/2023
11/10/2023
18/10/2023
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6/03/2024
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20/03/2024
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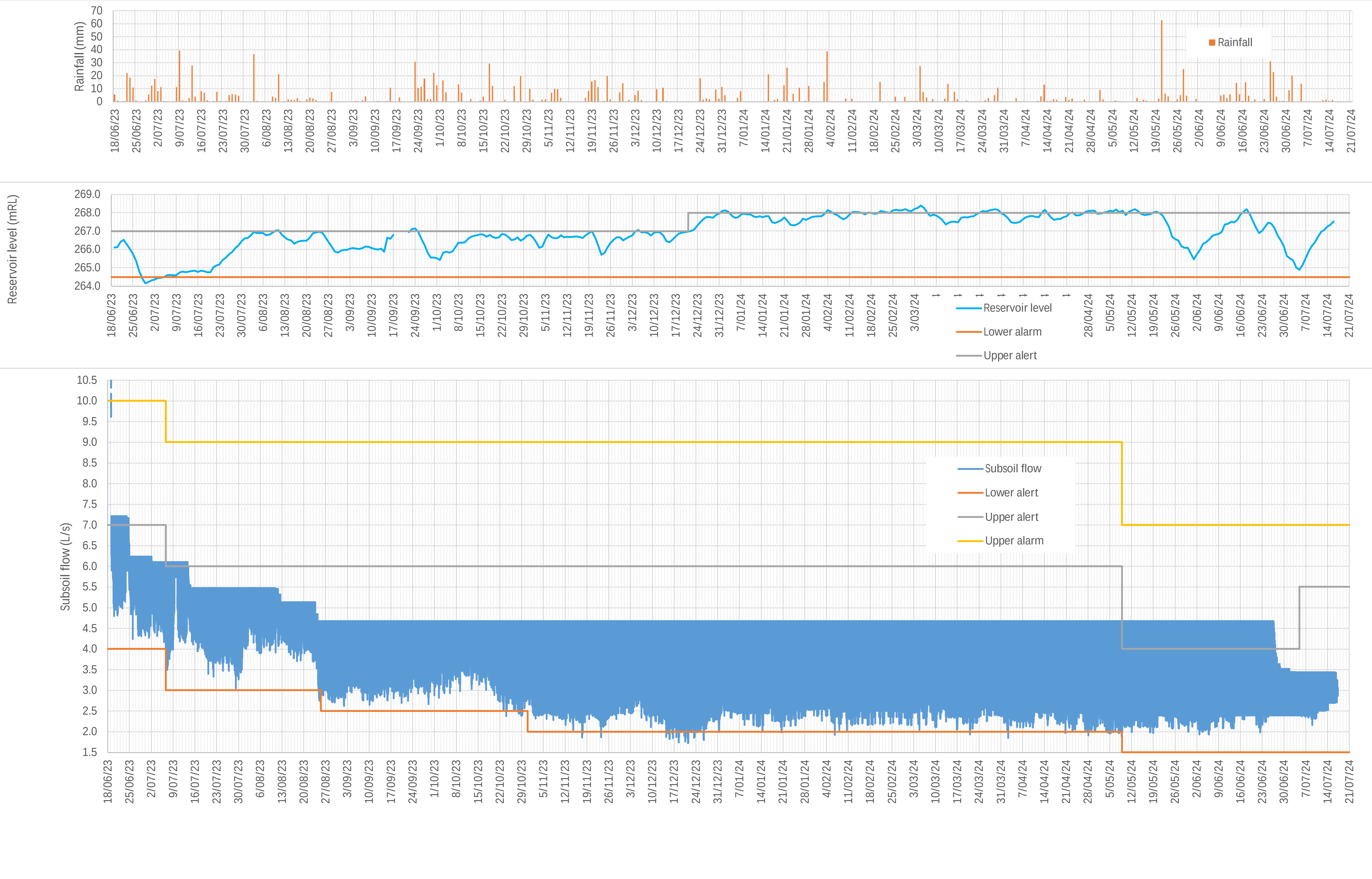
3/04/2024
10/04/2024
17/04/2024
24/04/2024
1/05/2024
8/05/2024
15/05/2024
22/05/2024
29/05/2024
5/06/2024
12/06/2024
19/06/2024
26/06/2024
3/07/2024
10/07/2024
17/07/2024

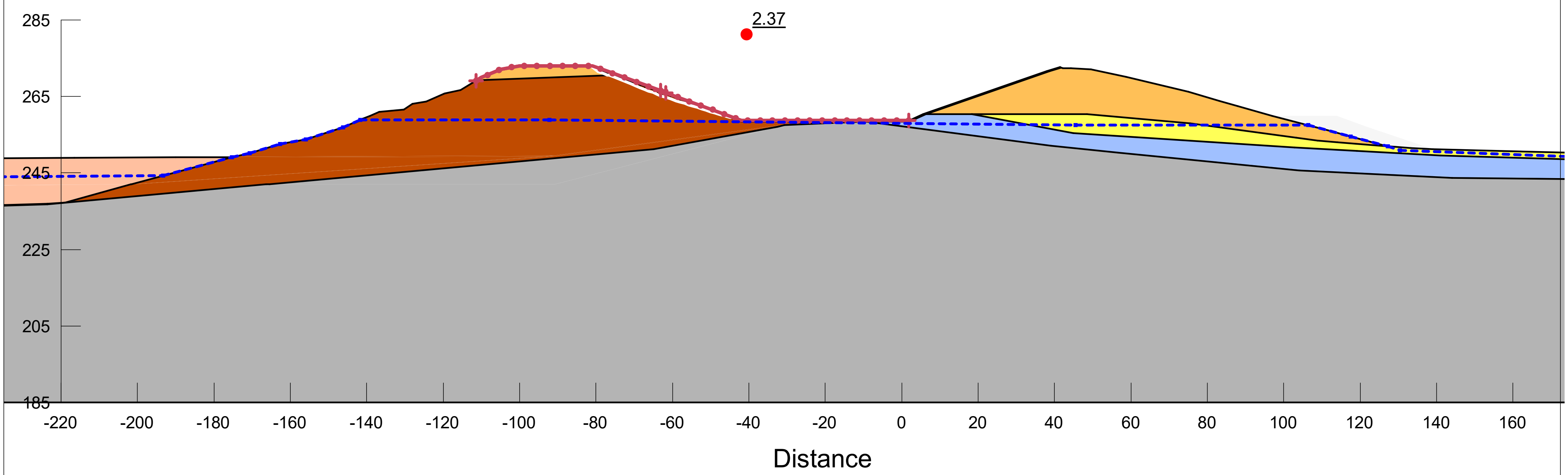
Hydrometric data since temporary repair works on 13-18 June 2023



Appendix C Seepage and stability analyses

Western Reservoir Rim Stability
Design Case 01 to 04
Analysed By: CHEV
Checked By: ABL

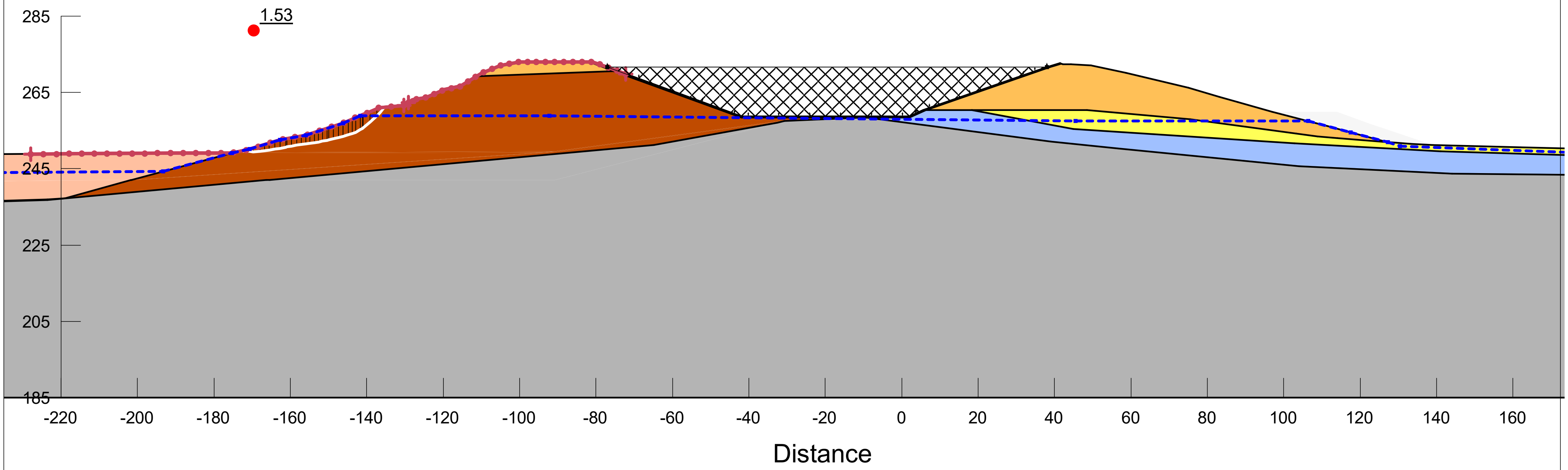
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1. Long-Term Stability - West U/S	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



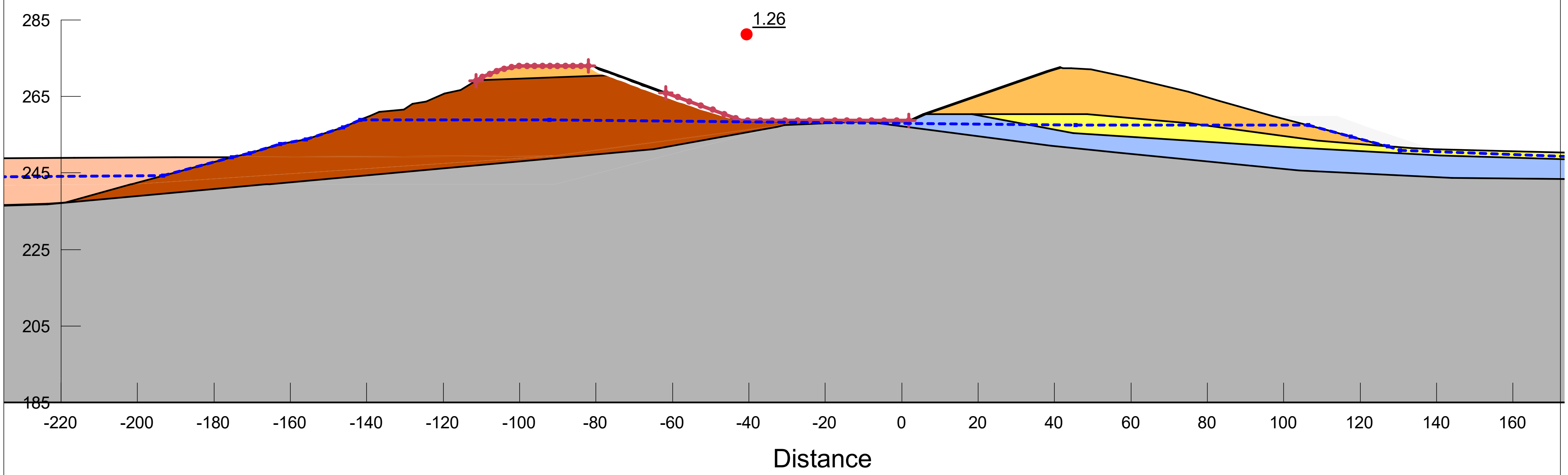
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 1. Long-Term Stability - West D/S		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 0.32g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



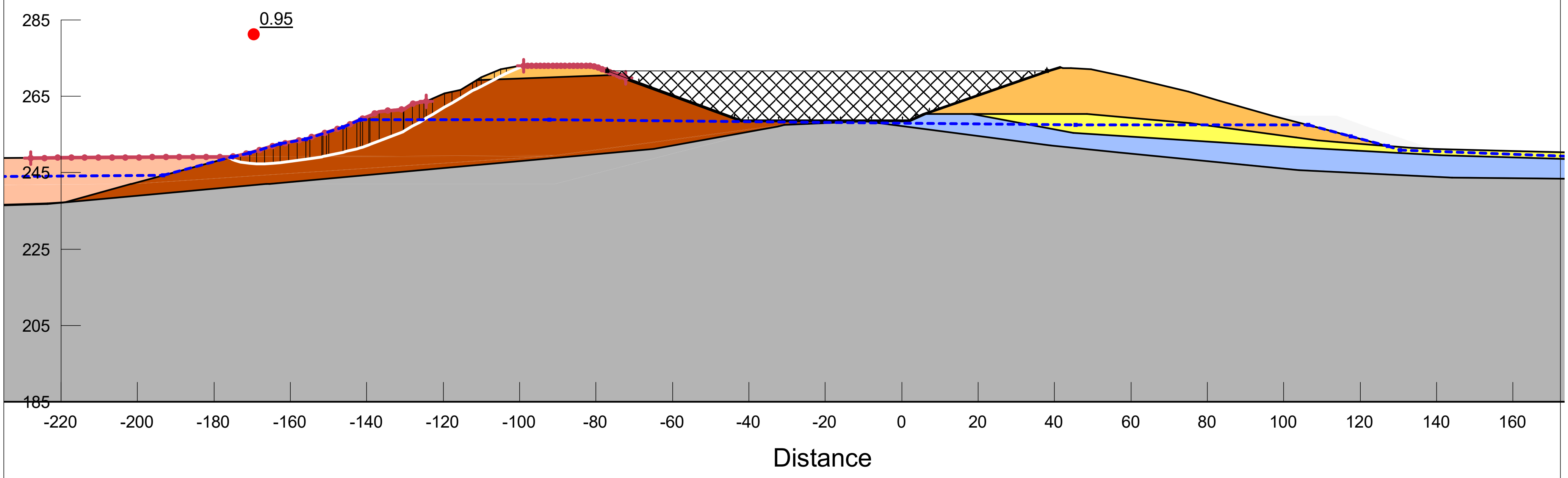
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: 2. Seismic (OBE 1170.5) - West U/S	Analysed by: CHEV
Comments: Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 0.32g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



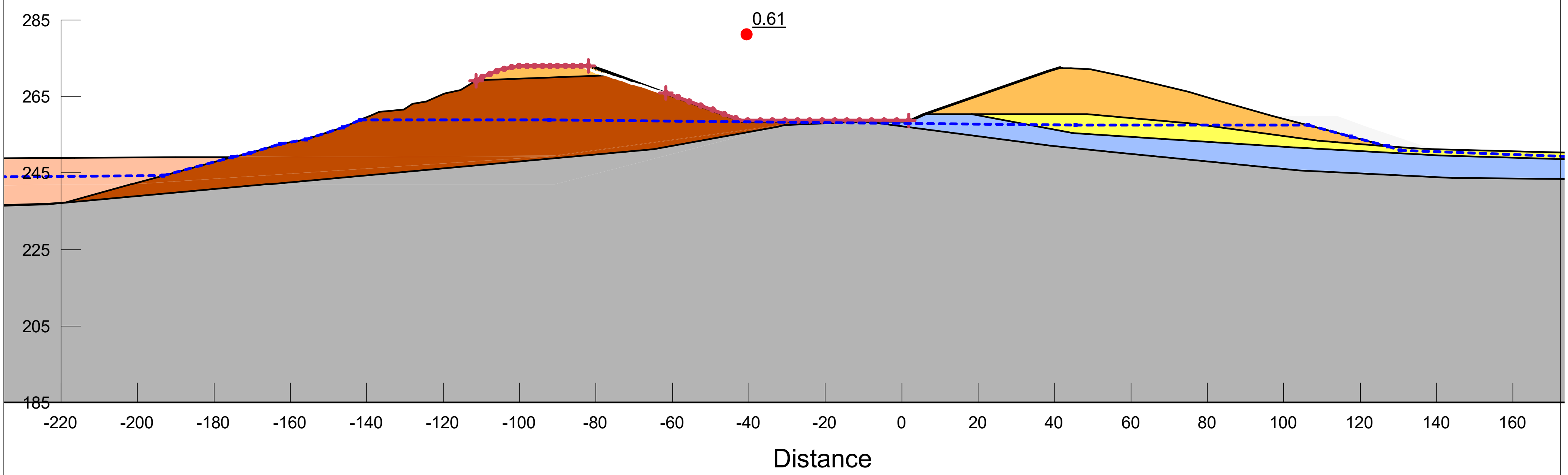
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 2. Seismic (OBE 1170.5) - West D/S		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 3. Seismic (SEE 1170.5) - West U/S	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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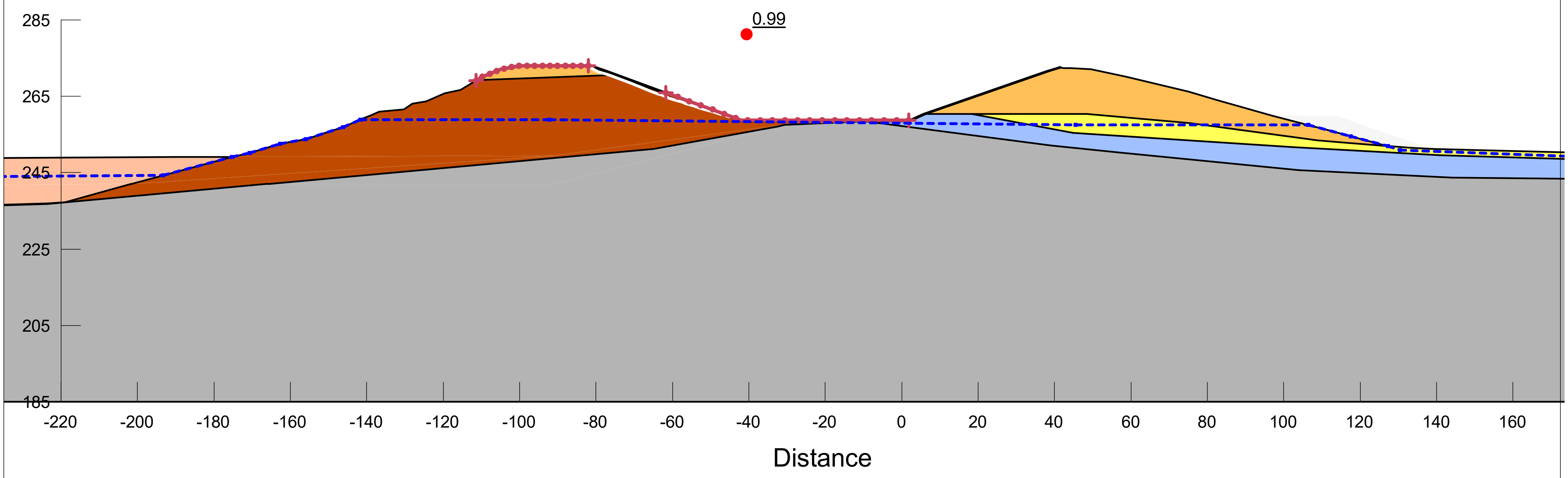
Horz Seismic Coef.: 0.5g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: Seismic Yield - West U/S		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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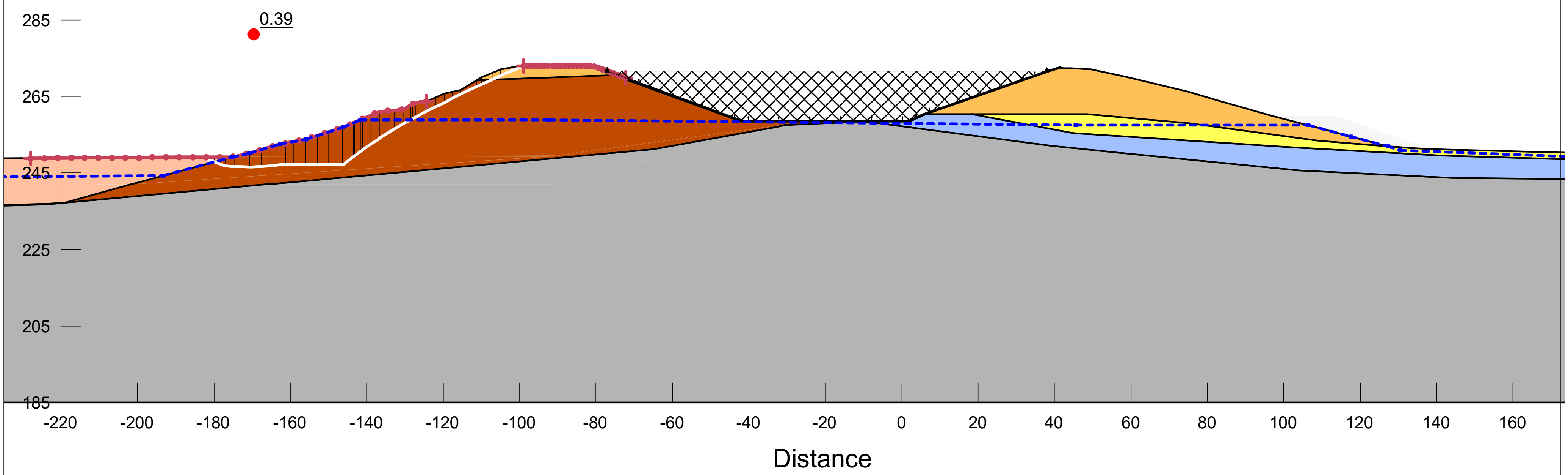
Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: 3. Seismic (SEE 1170.5) - West D/S	Analysed by: CHEV
Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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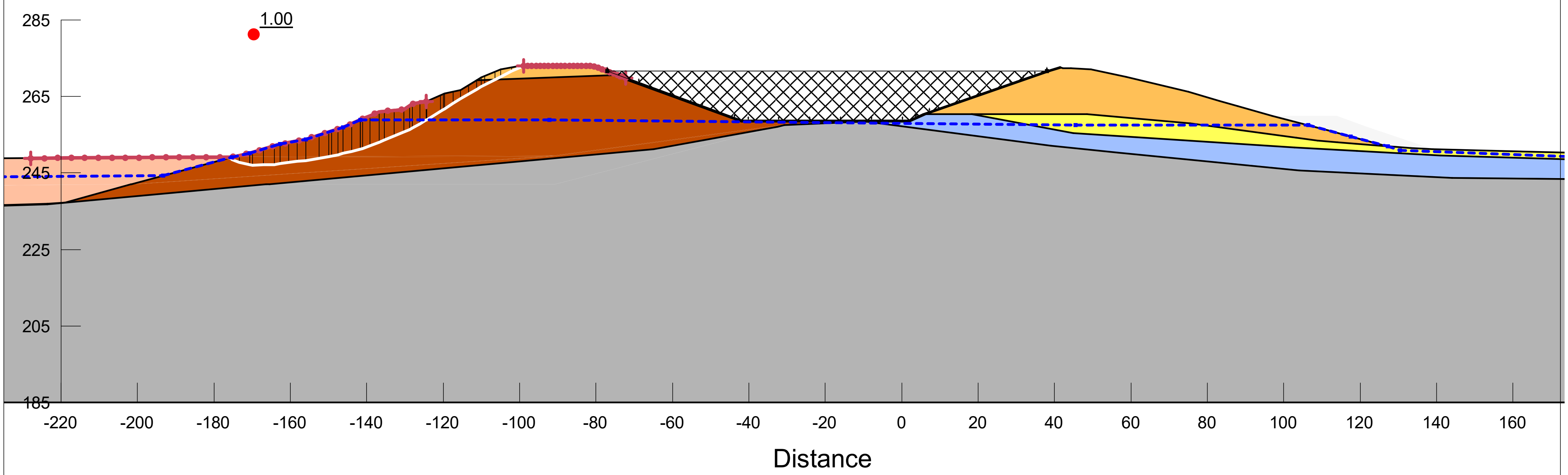
Horz Seismic Coef.: 0.29g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: Seismic Yield - West D/S	Analysed by: CHEV
Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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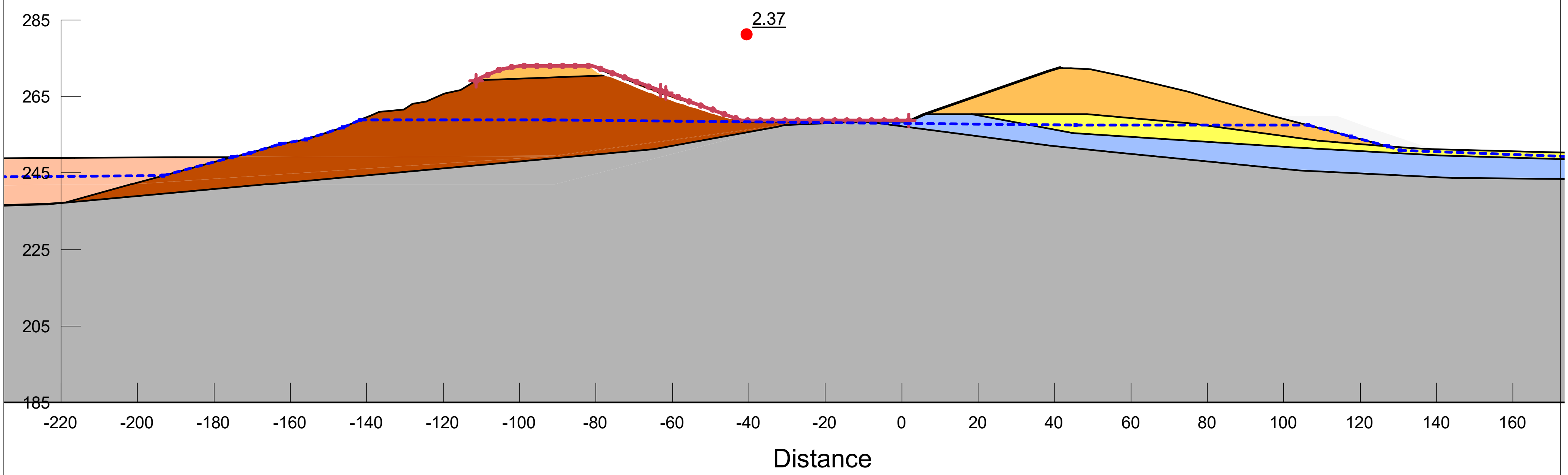
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
	Analysis: 4. Post-Seismic - West U/S		Analysed by: CHEV
	Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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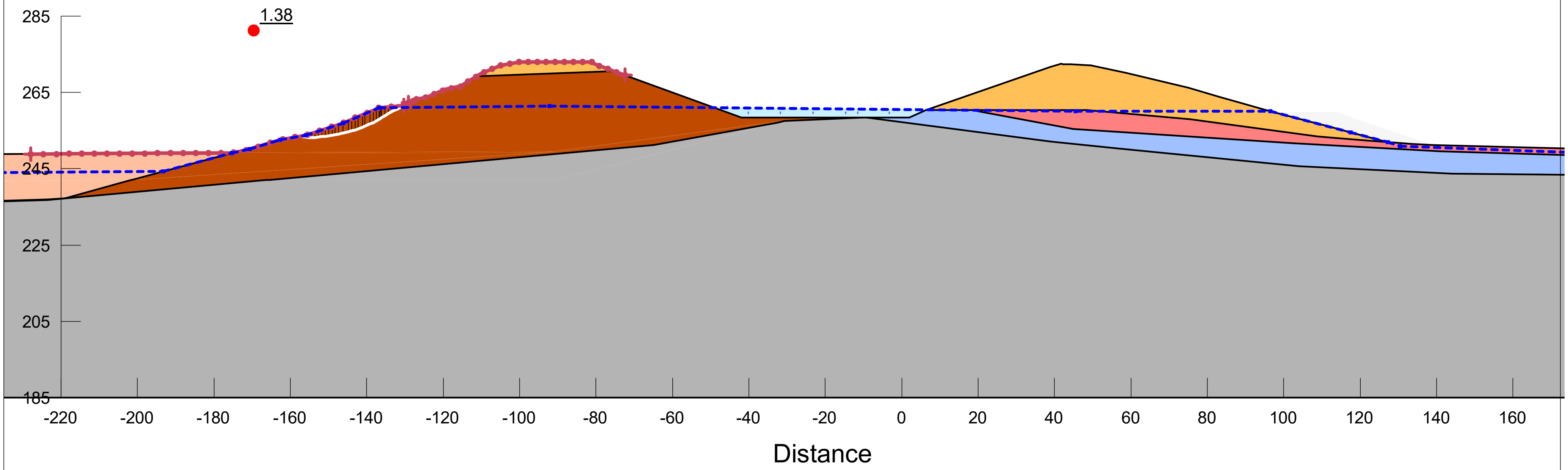
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50				1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 4. Post-Seismic - West D/S	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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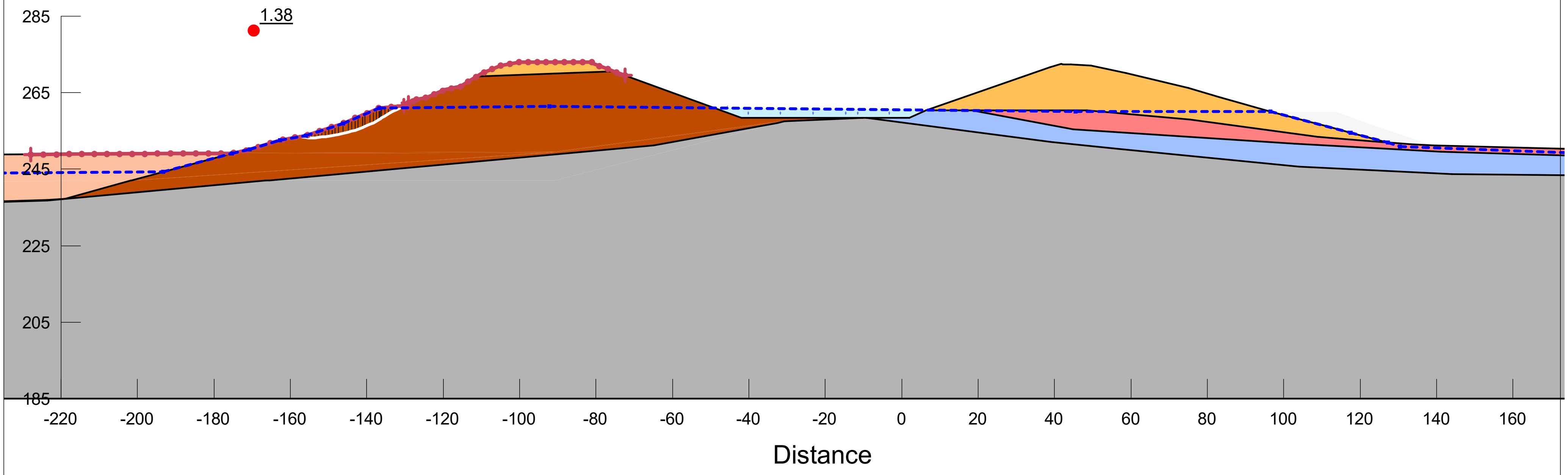
Western Reservoir Rim Stability
Sensitivity Cases
Analysed By: CHEV
Checked By: ABL

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50				1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1a. Long-Term Stability, Liner damage - West D/S	Analysed by: CHEV
	Comments: — Scale: 1:1,500 @ A4	Checked by: ABL

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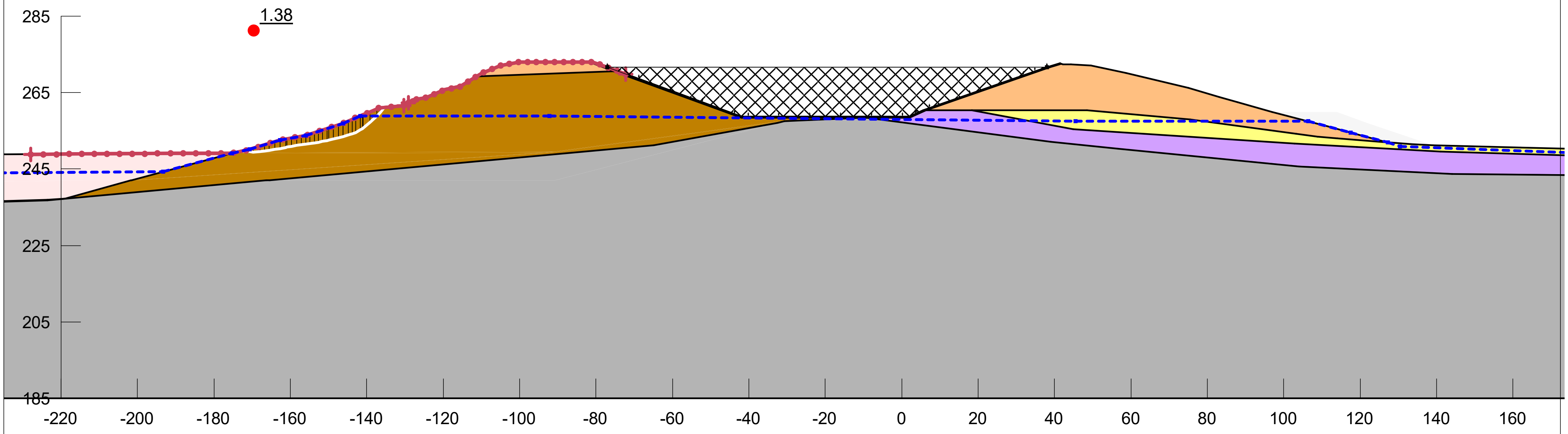
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand) (LB strength)	Mohr-Coulomb	21	0	34	0	1
	Late Pleistocene Alluvium (LB strength)	Mohr-Coulomb	21	0	38	0	1
	Loess (Clayey SILT, some sand) (LB strength)	Mohr-Coulomb	18	1	24	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY) (LB strength)	Mohr-Coulomb	19	3	28	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand) (LB strength)	Mohr-Coulomb	21	0	38	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1b. Long-Term Stability - West D/S (reduced strengths)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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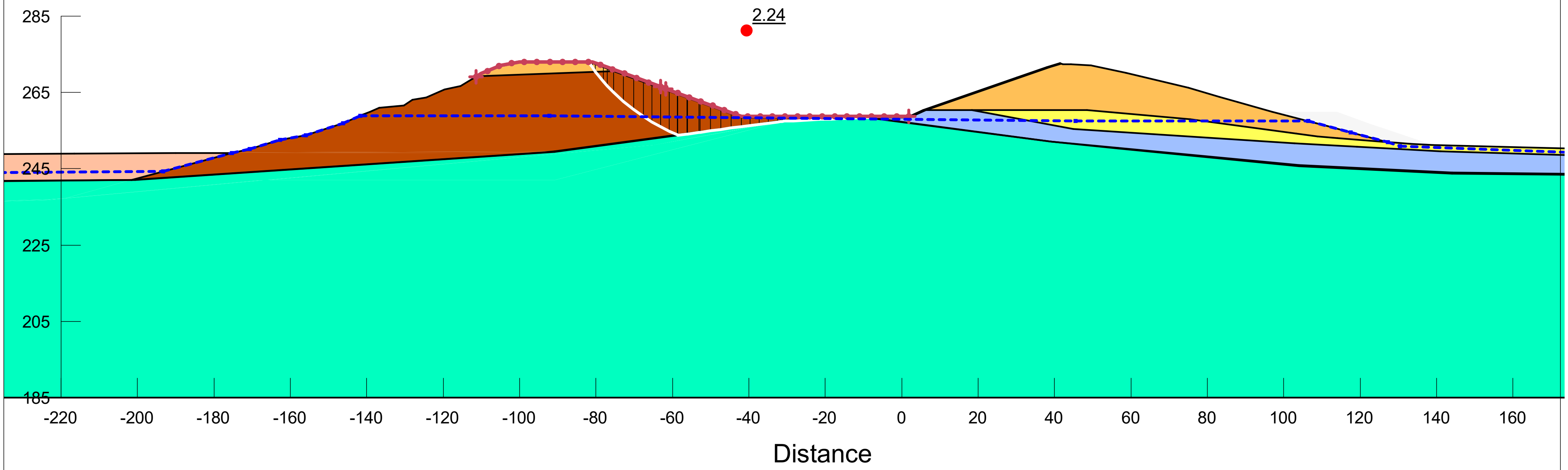
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1c. Long-Term Stability - West U/S (softened layer 7m)	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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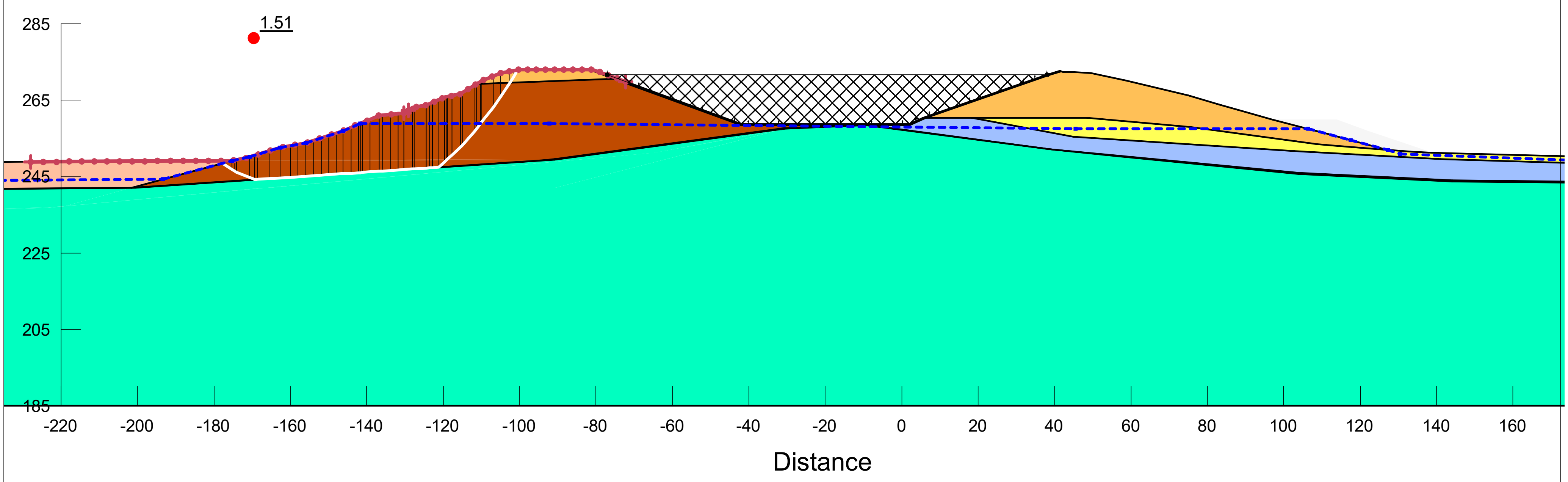
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
	Analysis: 1c. Long-Term Stability - West D/S (softened layer 7m)		Analysed by: CHEV
	Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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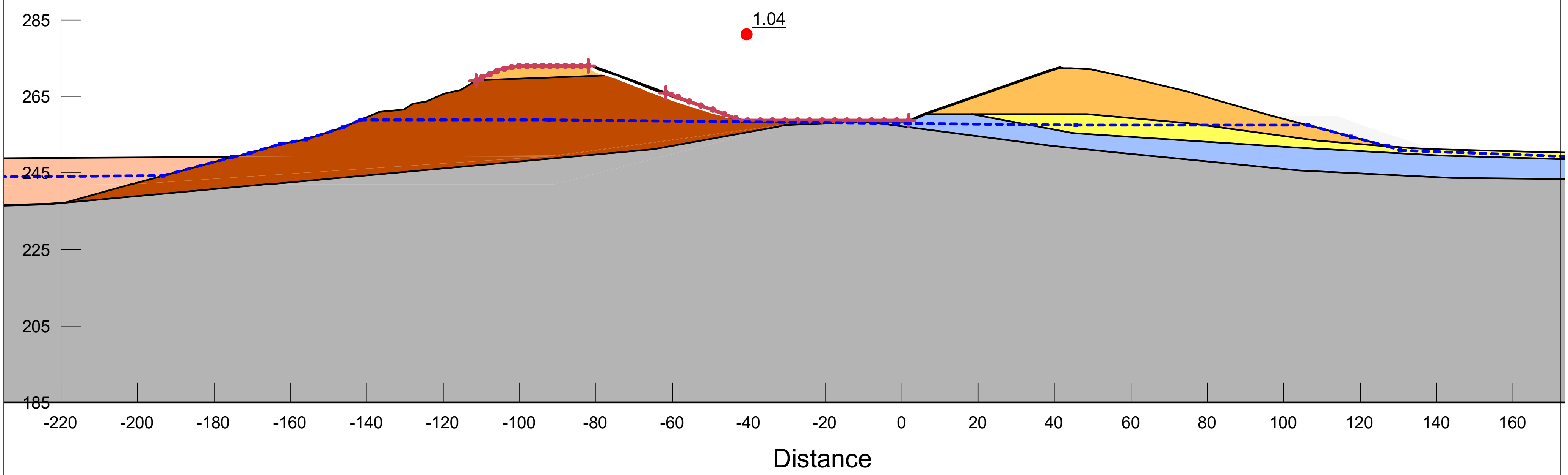
Horz Seismic Coef.: 0.46g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 2a. Seismic (OBE NSHM) - West U/S		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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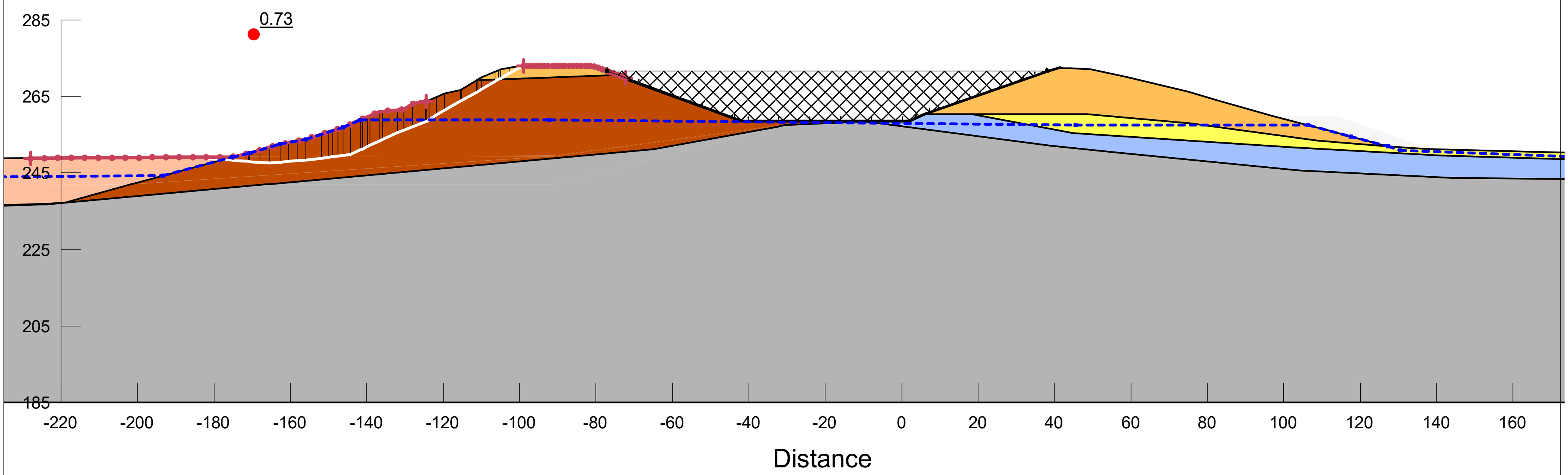
Horz Seismic Coef.: 0.46g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: 2a. Seismic (OBE NSHM) - West D/S	Analysed by: CHEV
Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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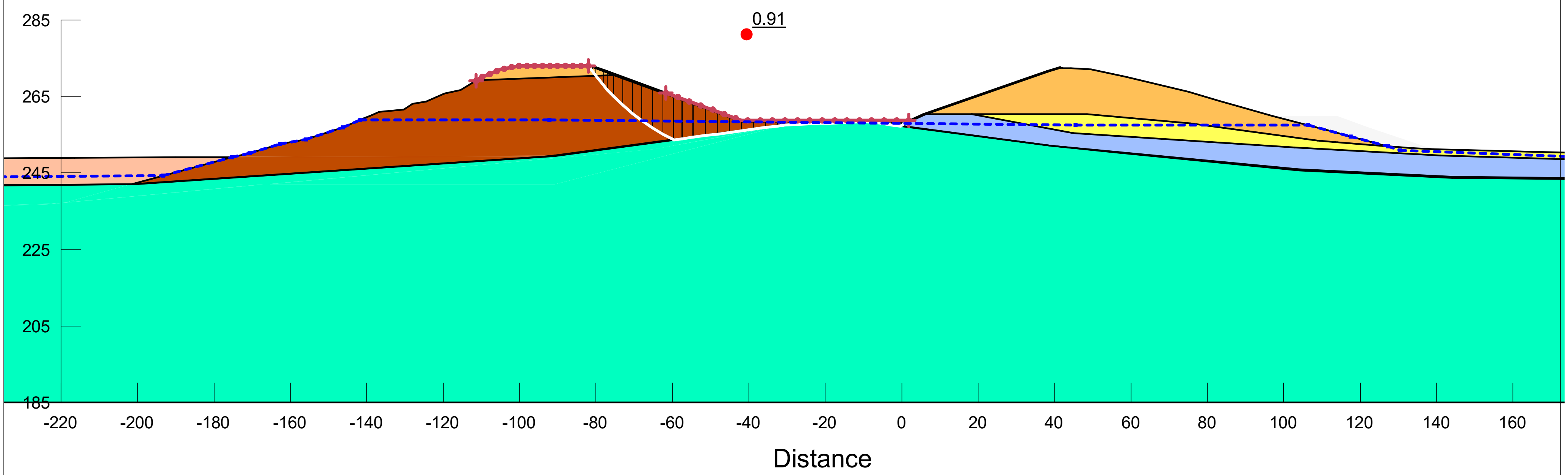
Horz Seismic Coef.: 0.32g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 2b. Seismic (OBE 1170.5) - West U/S (softened layer 7m)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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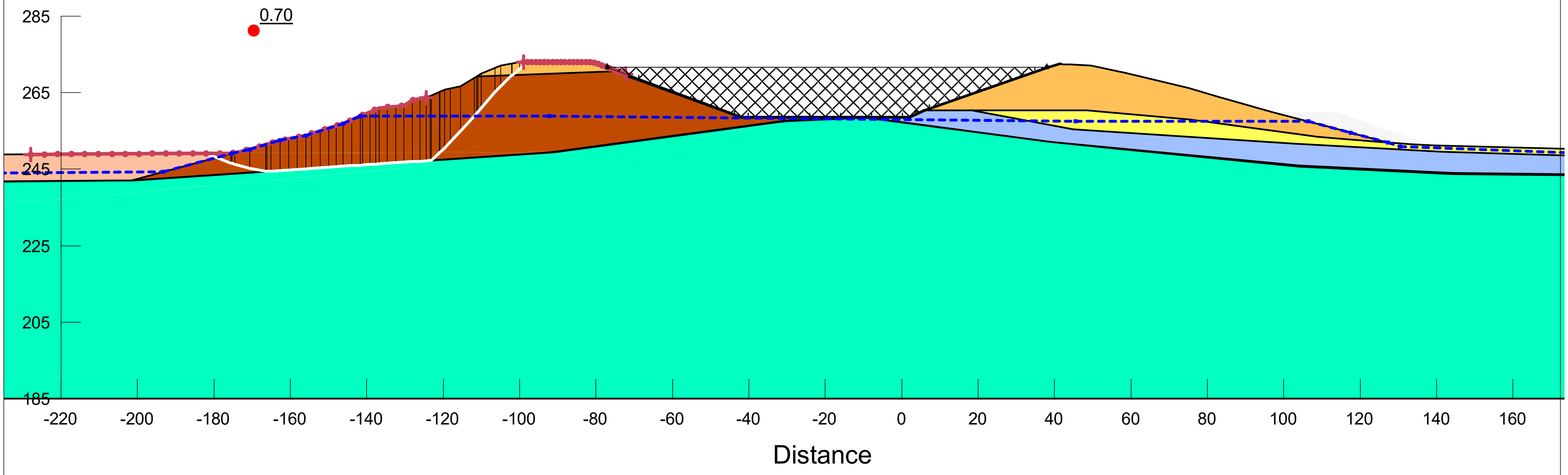
Horz Seismic Coef.: 0.32g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 2b. Seismic (OBE 1170.5) - West D/S (softened layer 7m)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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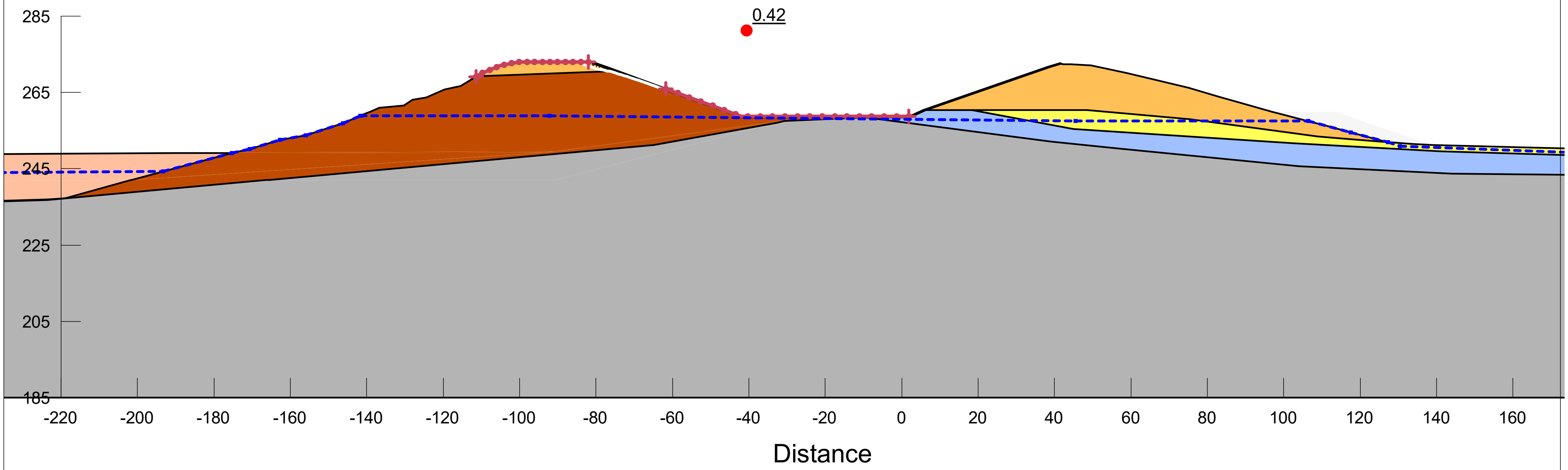
Horz Seismic Coef.: 1.59g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 3a. Seismic (SEE NSHM) - West U/S	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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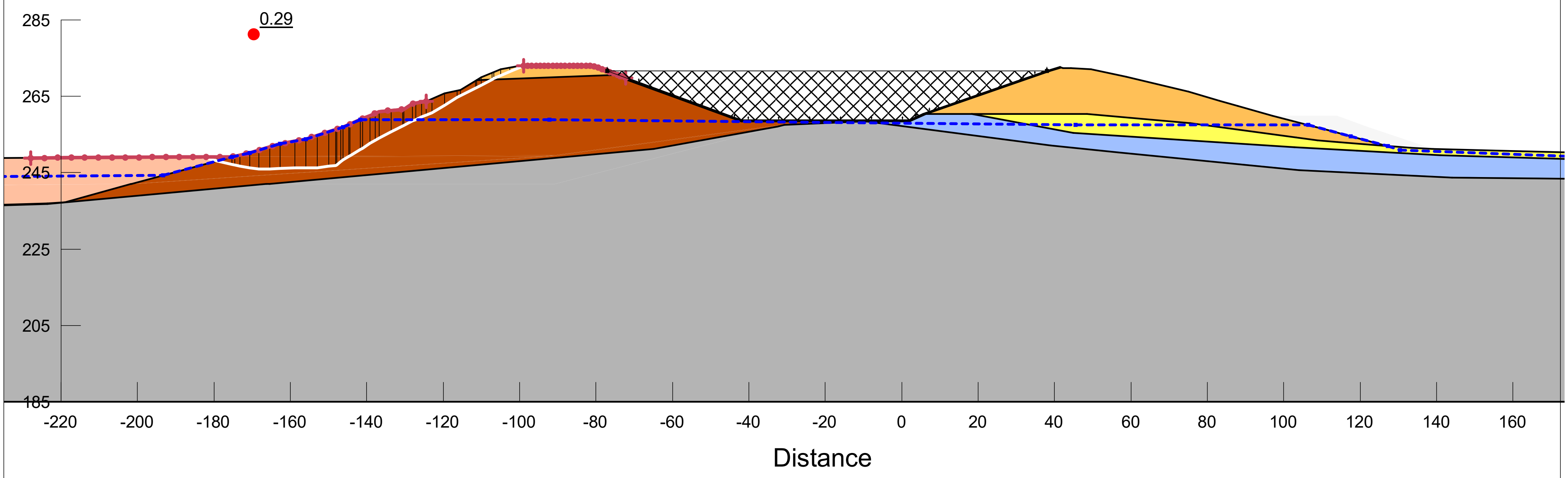
Horz Seismic Coef.: 1.59g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



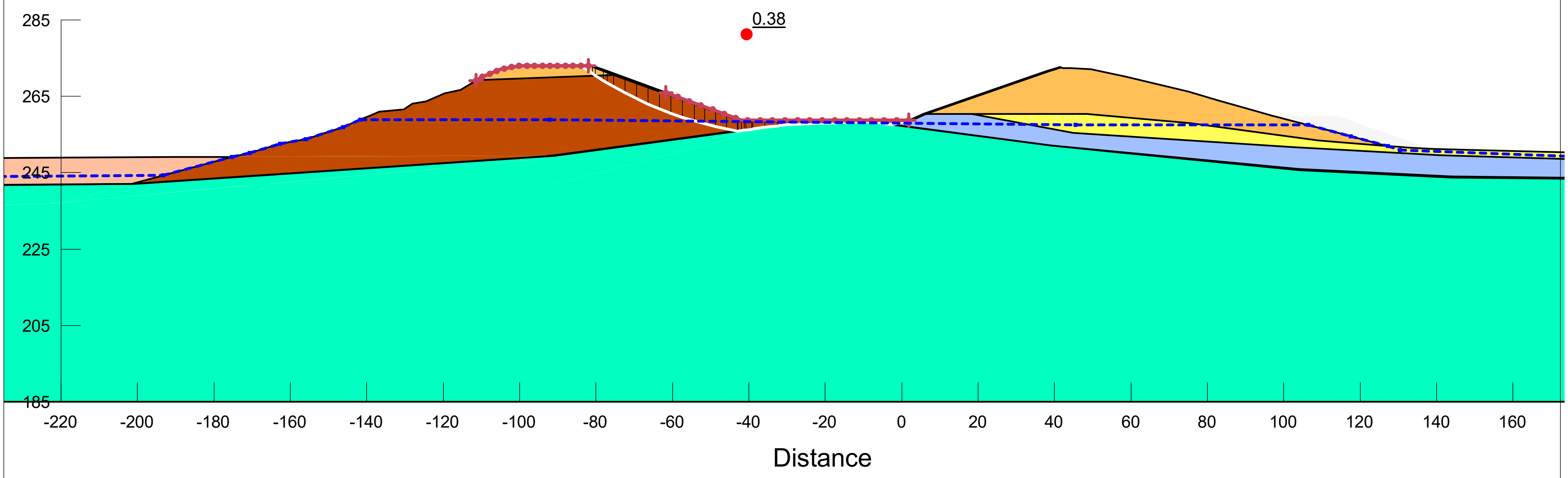
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 3a. Seismic (SEE NSHM) - West D/S		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



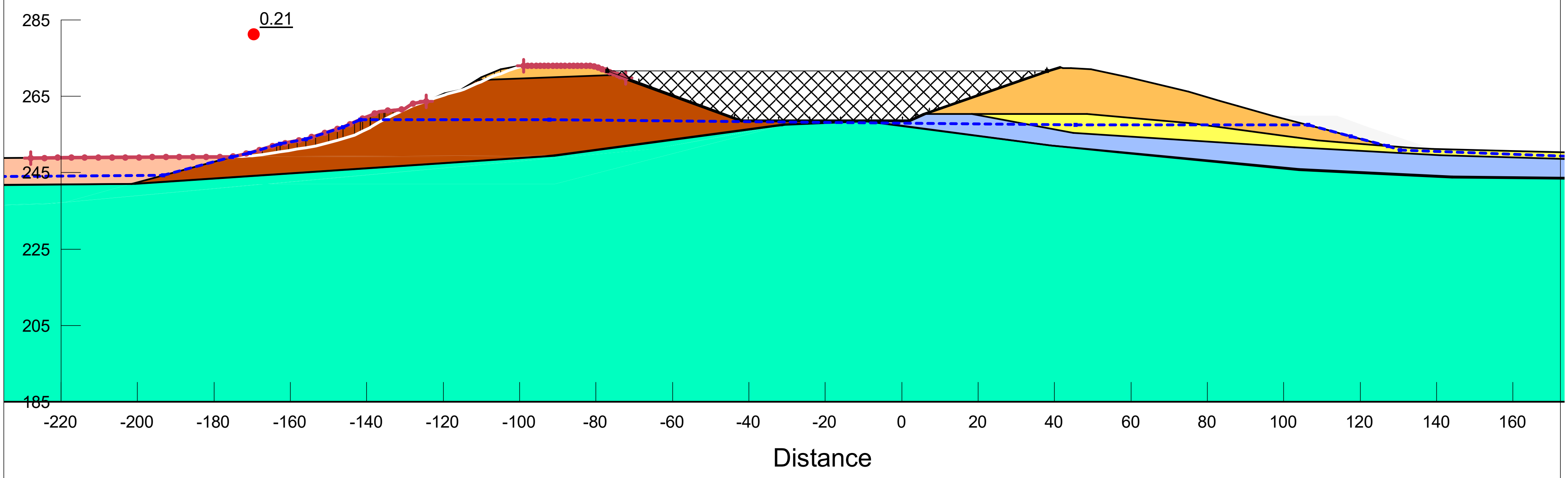
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 3b. Seismic (SEE 1170.5) - West U/S (softened layer 7m)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 3b. Seismic (SEE 1170.5) - West D/S (softened layer 7m)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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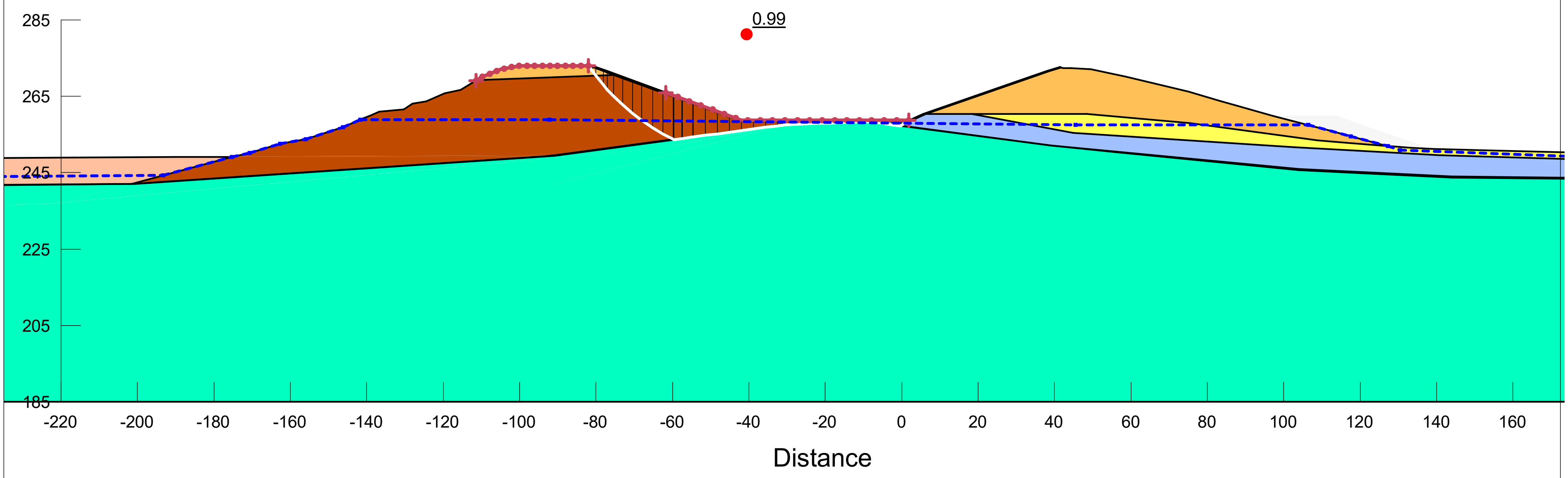
Horz Seismic Coef.: 0.28g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
	Analysis: Seismic Yield - West U/S (softened layer 7m)		Analysed by: CHEV
	Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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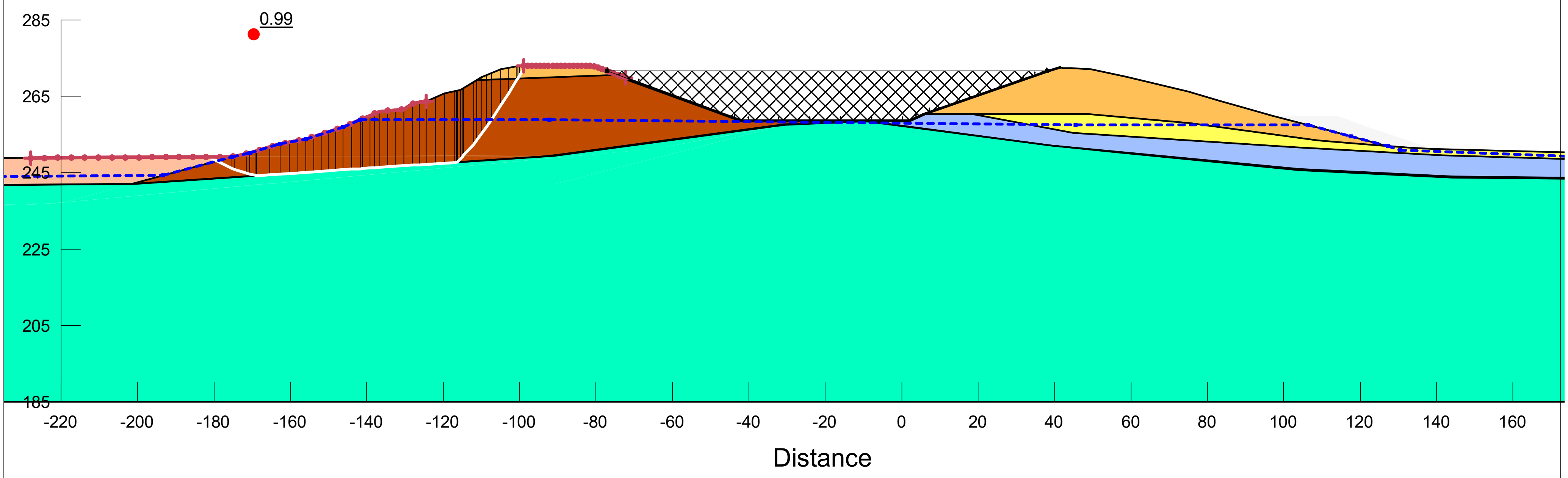
Horz Seismic Coef.: 0.14g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)					1
	Soil-Rock Interface	Mohr-Coulomb	18	0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: Seismic Yield - West D/S (softened layer 7m)	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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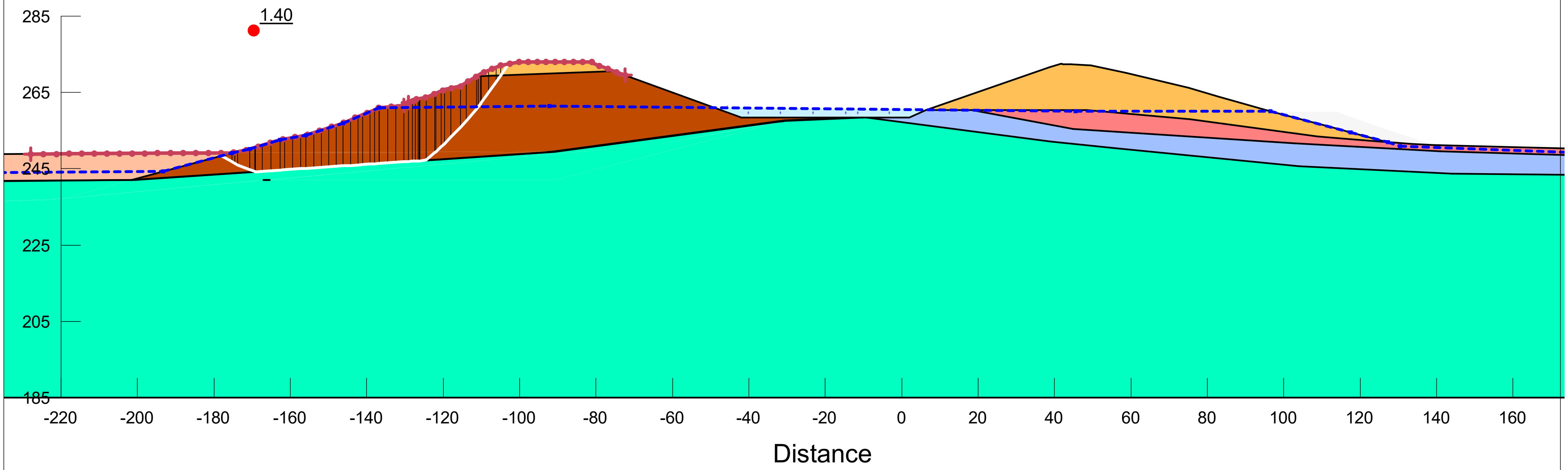
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50				1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	1
	Mangaheia Group (Sandstone/Siltstone) (Impenetrable)	Bedrock (Impenetrable)						1
	Soil-Rock Interface	Mohr-Coulomb	18		0	22	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 4a. Post-Seismic - West D/S (softened layer 7m)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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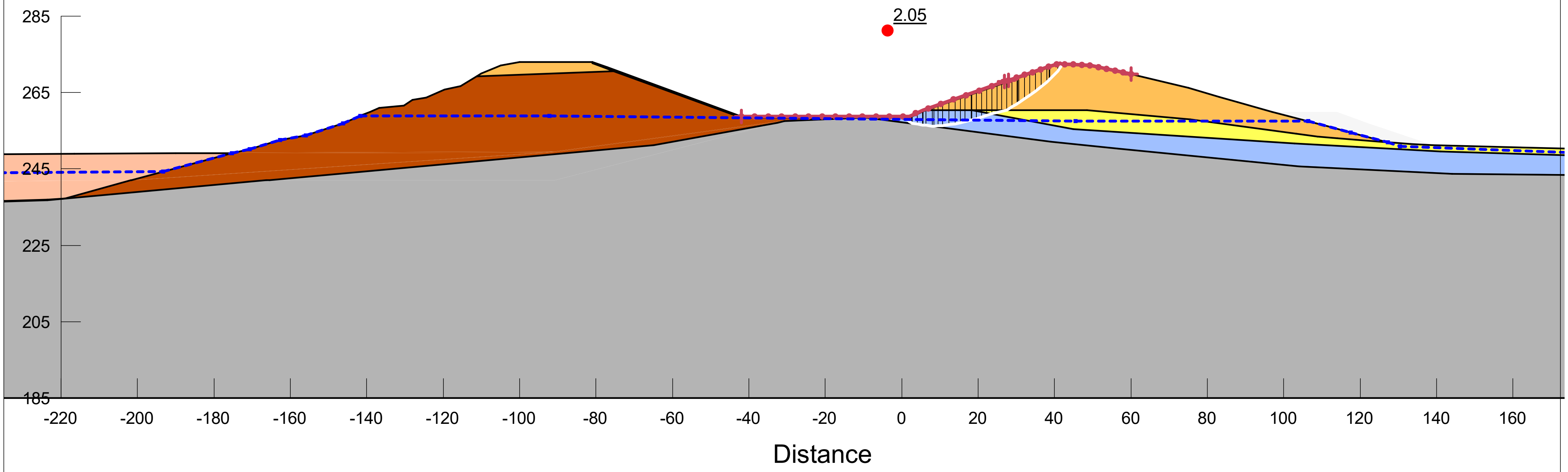
Eastern Embankment Stability
Design Case 01 to 04
Analysed By: CHEV
Checked By: ABL

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1. Long-Term Stability - East U/S	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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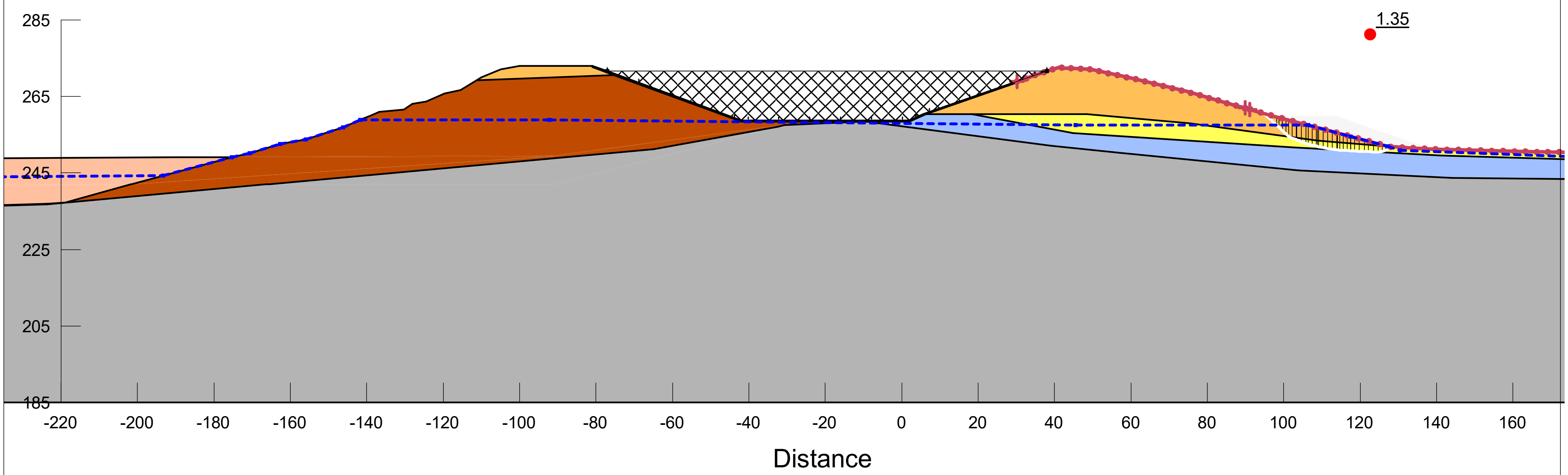
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1. Long-Term Stability - East D/S	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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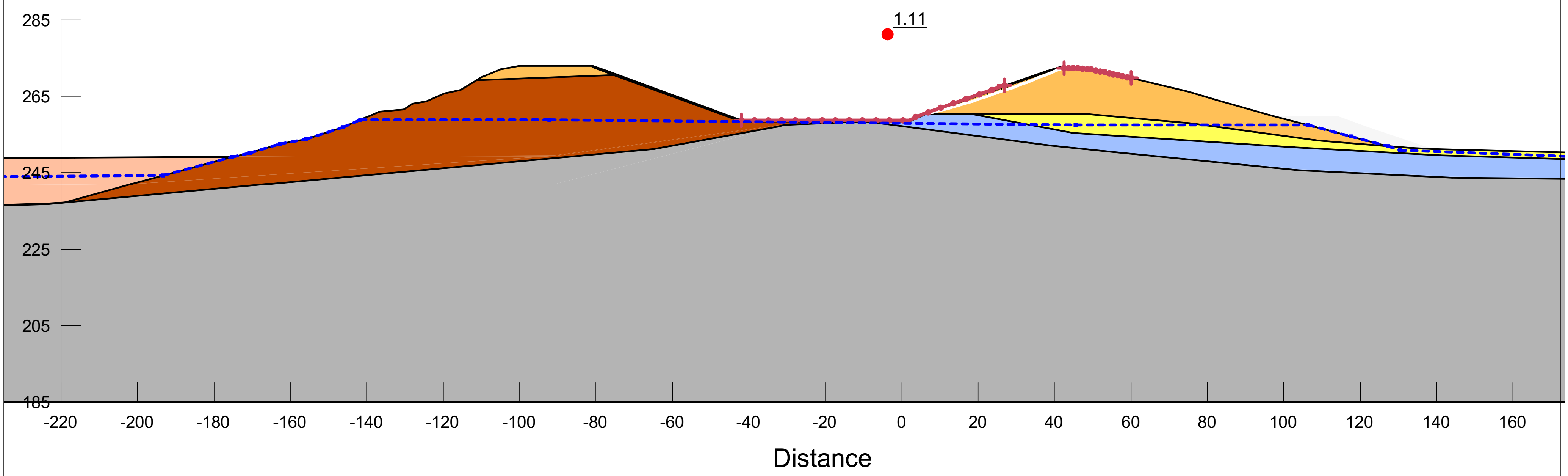
Horz Seismic Coef.: 0.32g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



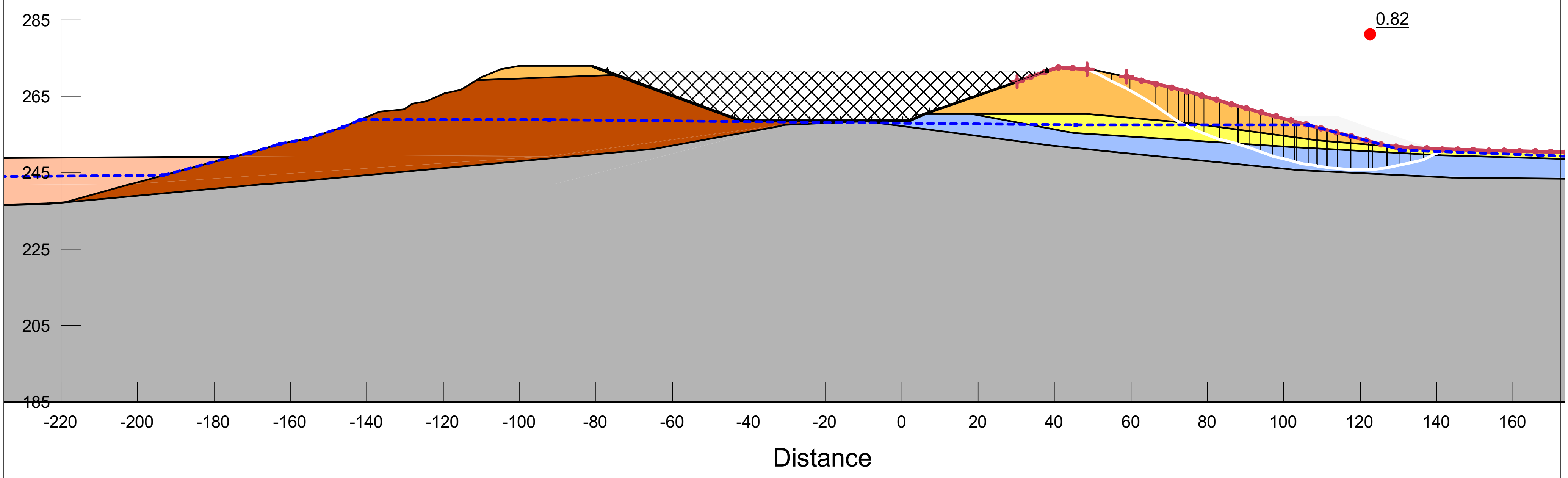
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 2. Seismic (OBE 1170.5) - East U/S		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 0.32g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz

Job Number: 1020688.4000

Analysis: 2. Seismic (OBE 1170.5) - East D/S

Analysed by: CHEV

Comments:

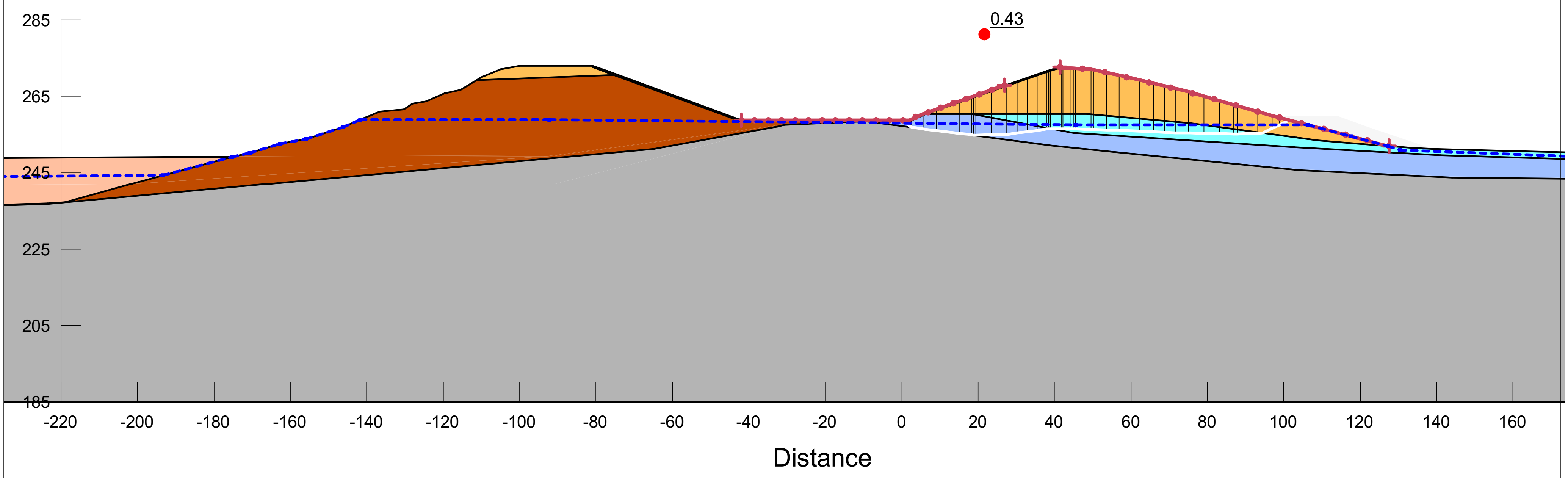
Scale: 1:1,500 @ A4

Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



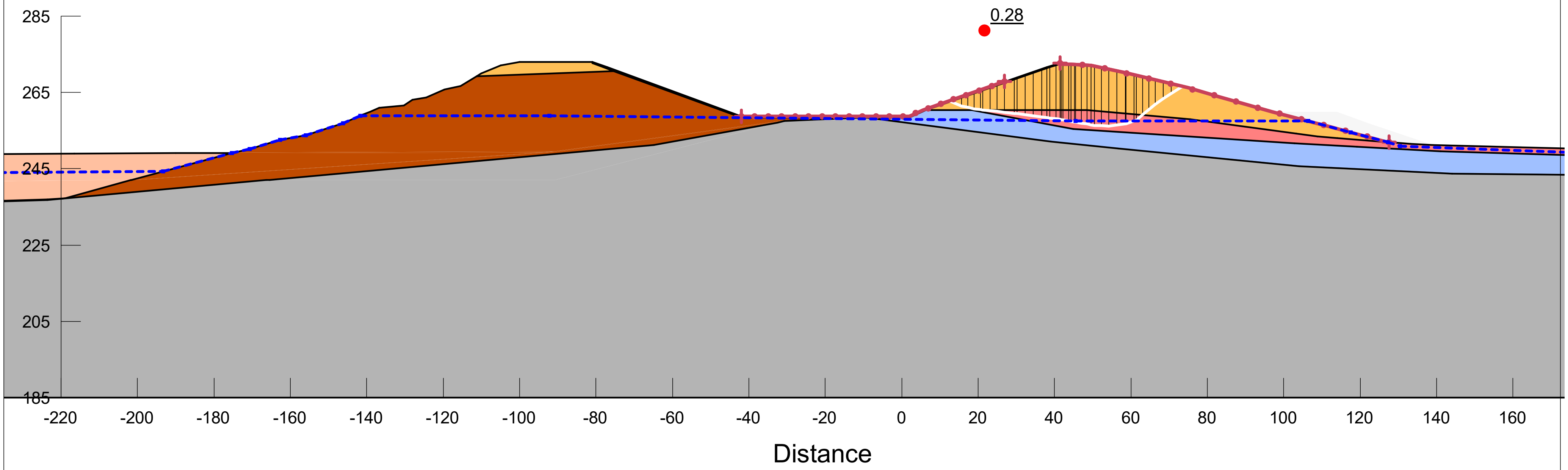
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 3. Seismic (SEE 1170.5) + CS - East U/S (drained)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50						1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	0	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 3. Seismic (SEE 1170.5) + CS - East U/S (undrained)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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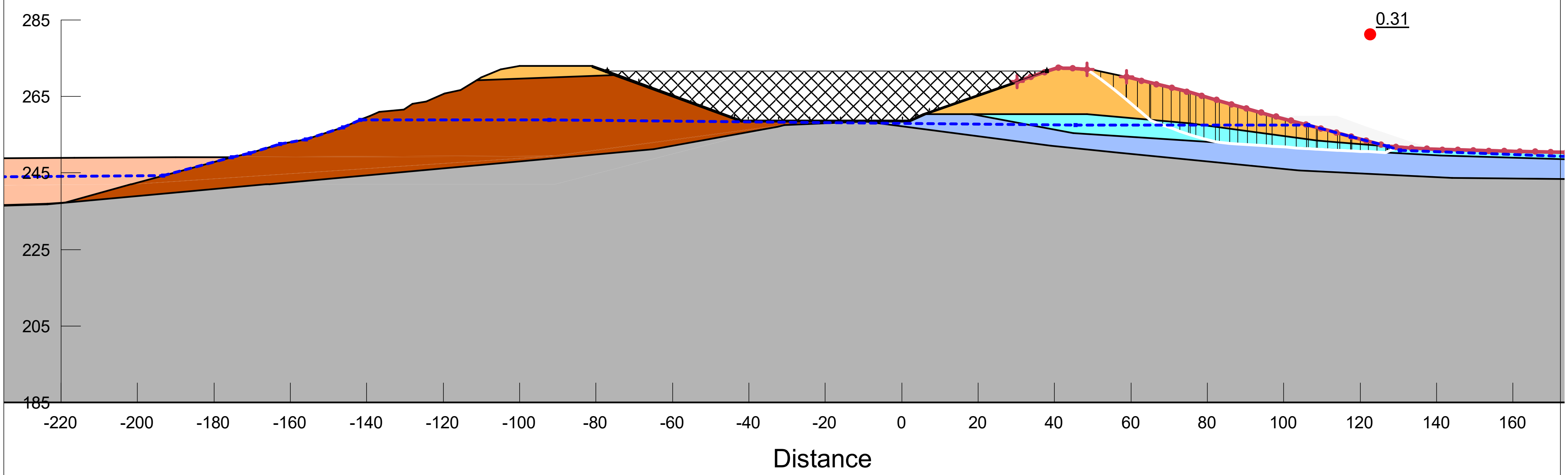
Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1

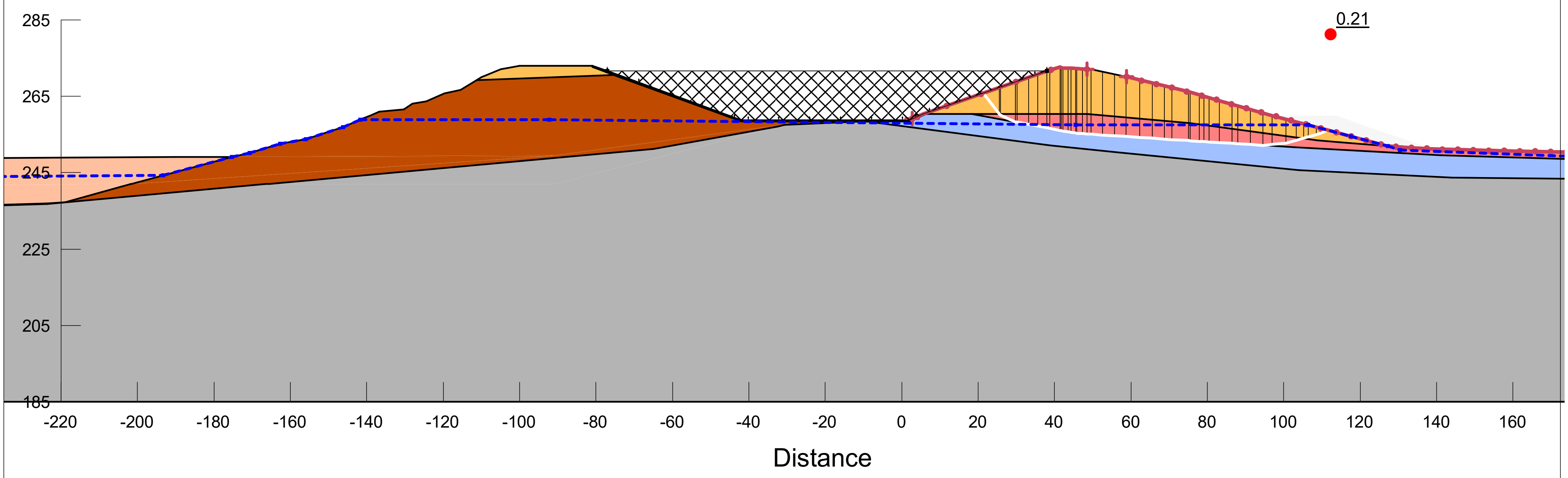


Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 3. Seismic (SEE 1170.5) + CS - East D/S (drained)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50						1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	0	0	1



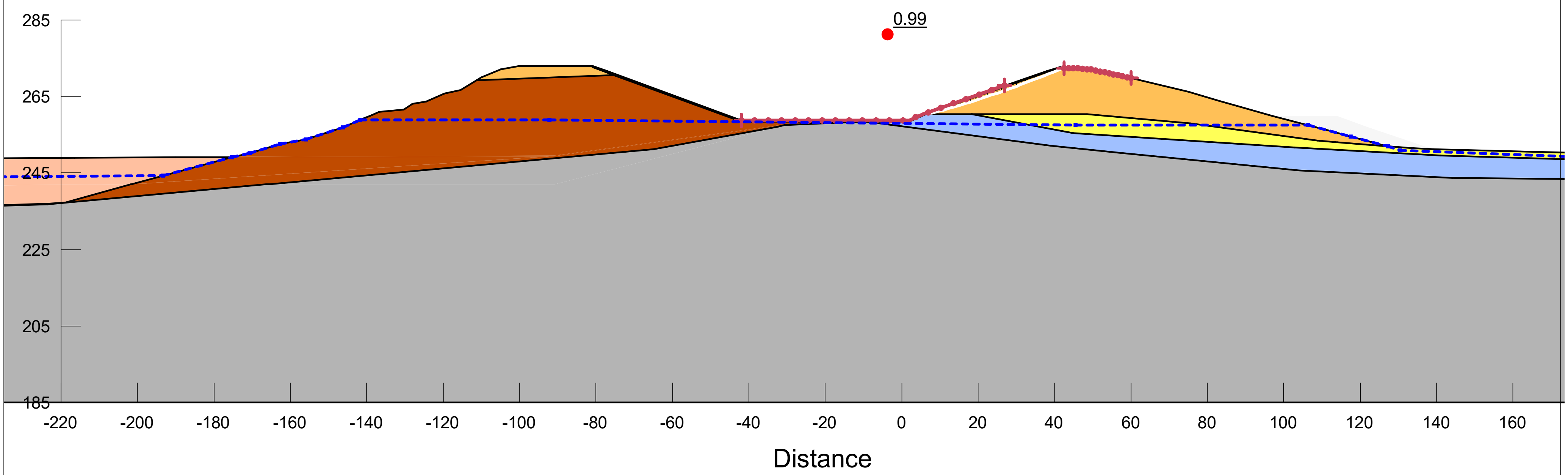
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: 3. Seismic (SEE 1170.5) + CS - East D/S (undrained)	Analysed by: CHEV
Comments: Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 0.4g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz

Analysis: Seismic Yield - East U/S

Comments:

Scale: 1:1,500 @ A4

Job Number: 1020688.4000

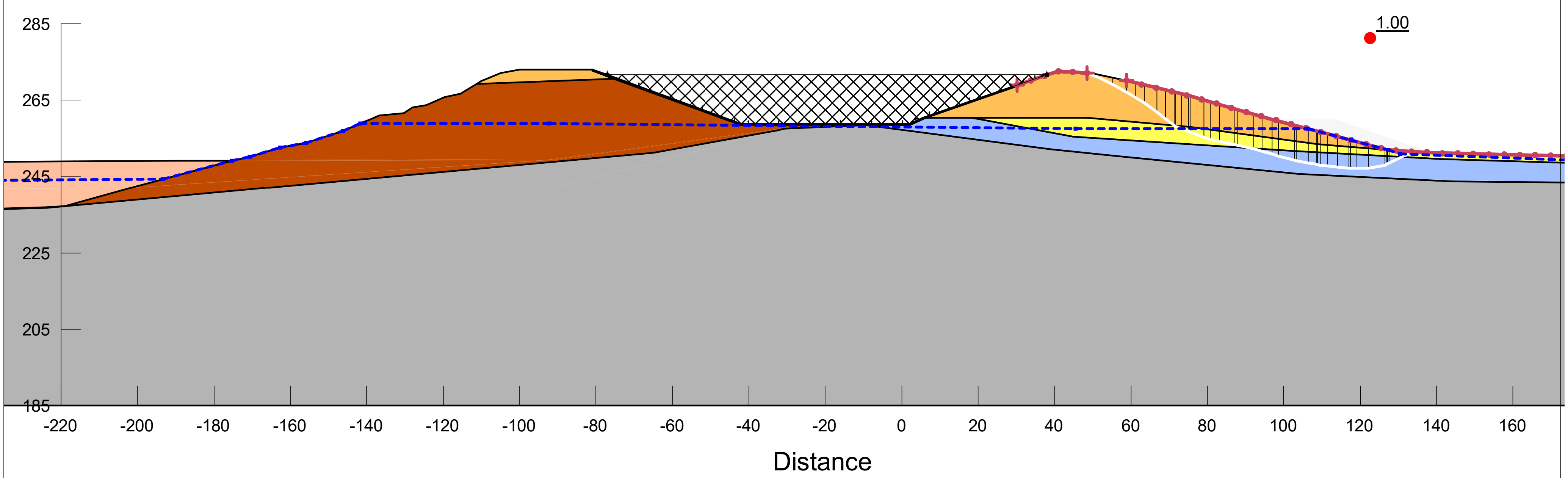
Analysed by: CHEV

Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 0.21g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: Seismic Yield - East D/S	Analysed by: CHEV
Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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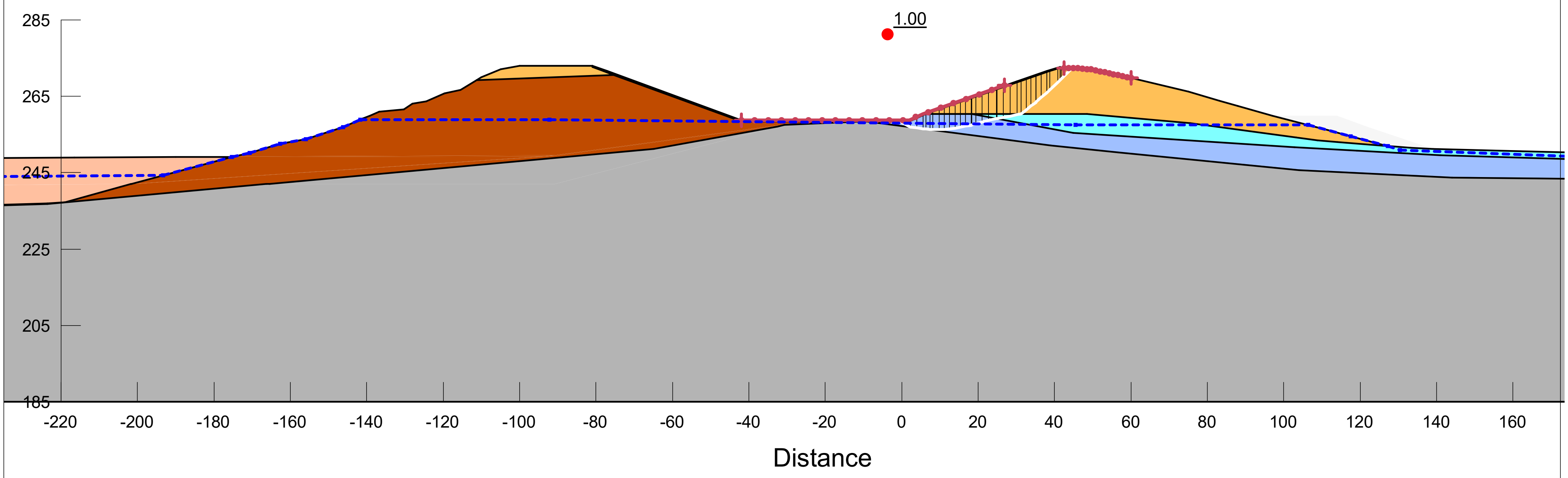
Horz Seismic Coef.: 0.35g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18		2.4	21	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50						1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	0	0	1



Tonkin+Taylor		
	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: Seismic Yield + Cyclic Softening - East U/S (drained)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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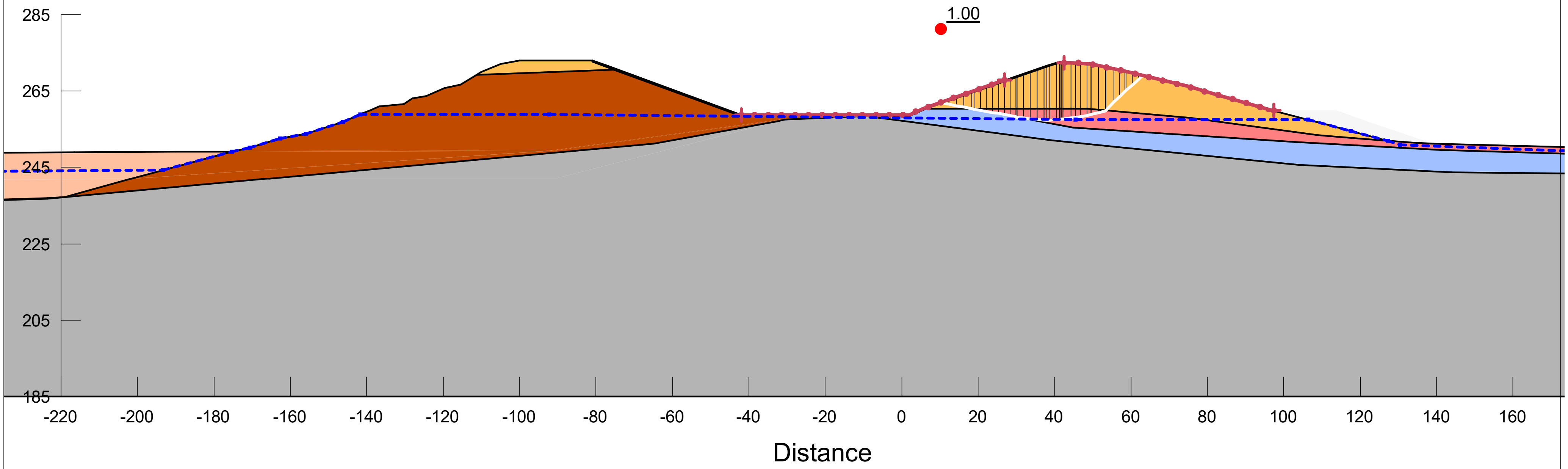
Horz Seismic Coef.: 0.25g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50						1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: Seismic Yield + Cyclic Softening - East U/S (undrained)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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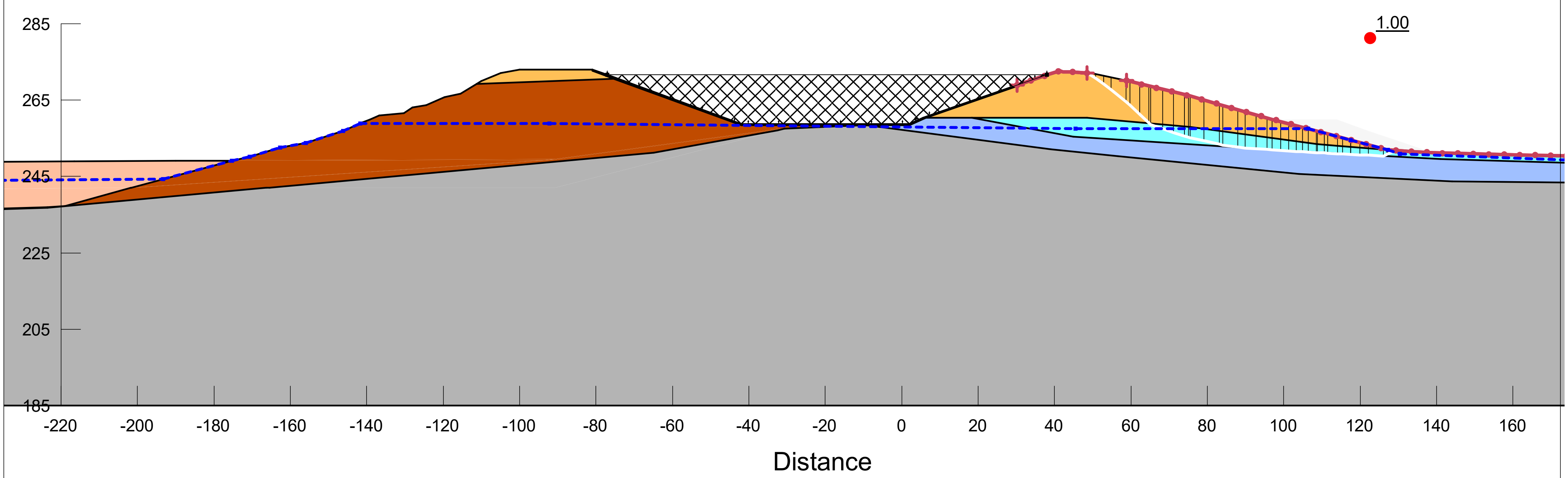
Horz Seismic Coef.: 0.13g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
	Analysis: Seismic Yield + Cyclic Softening - East D/S (drained)		Analysed by: CHEV
	Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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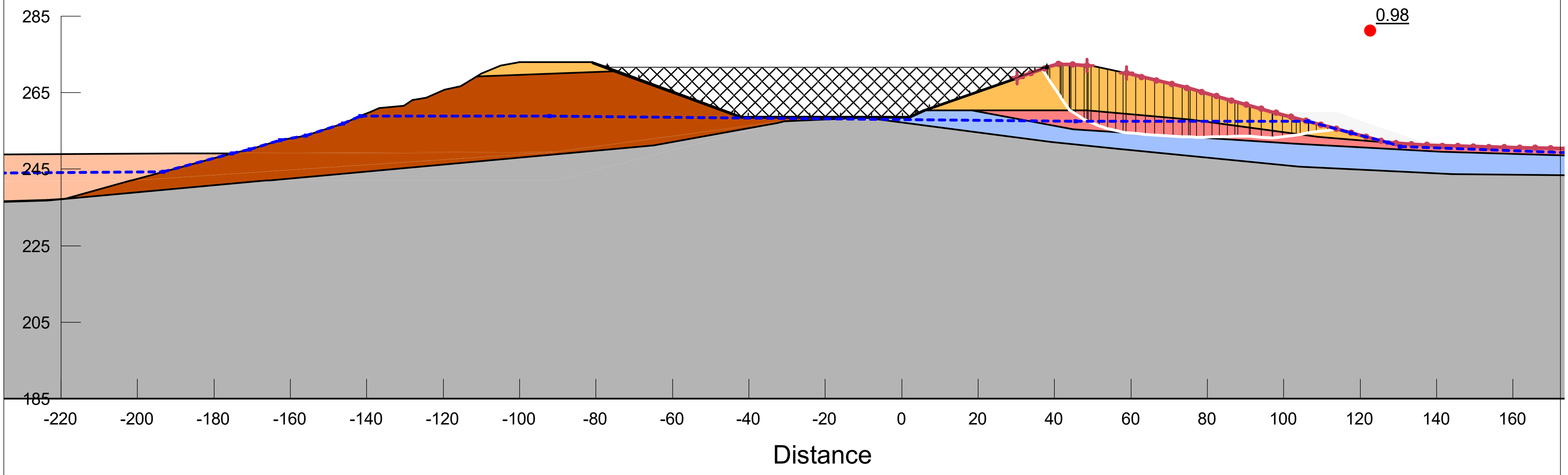
Horz Seismic Coef.: 0.1g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18		2.4	21	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50						1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: Seismic Yield + Cyclic Softening - East D/S (undrained)	Analysed by: CHEV
Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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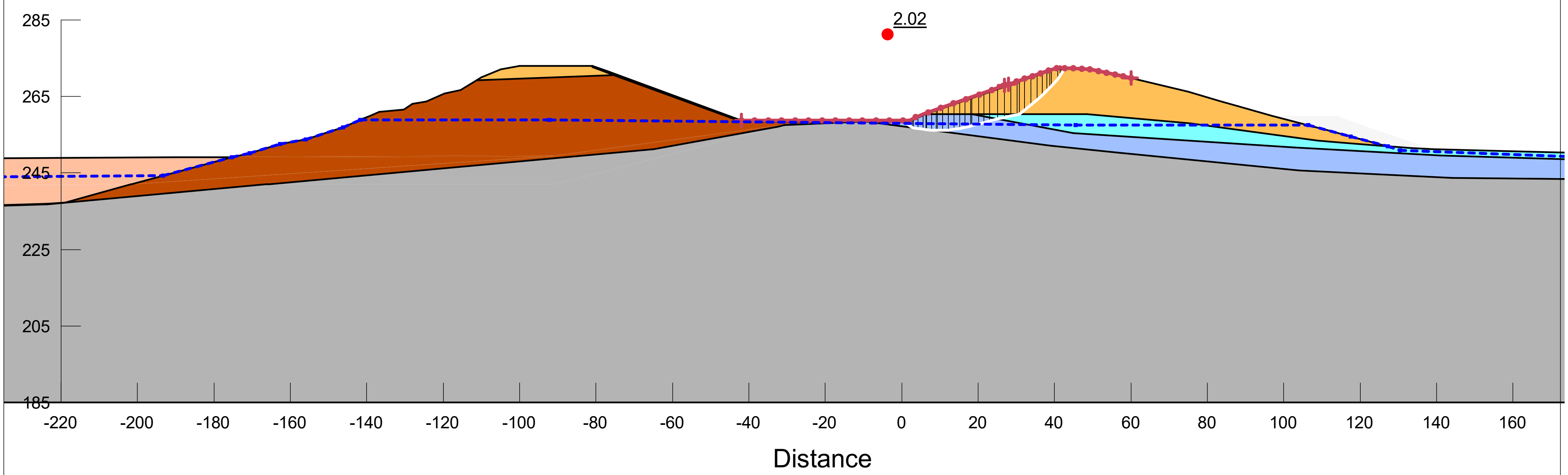
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: 4. Post-Seismic + Cyclic Softening - East U/S (drained)	Analysed by: CHEV
Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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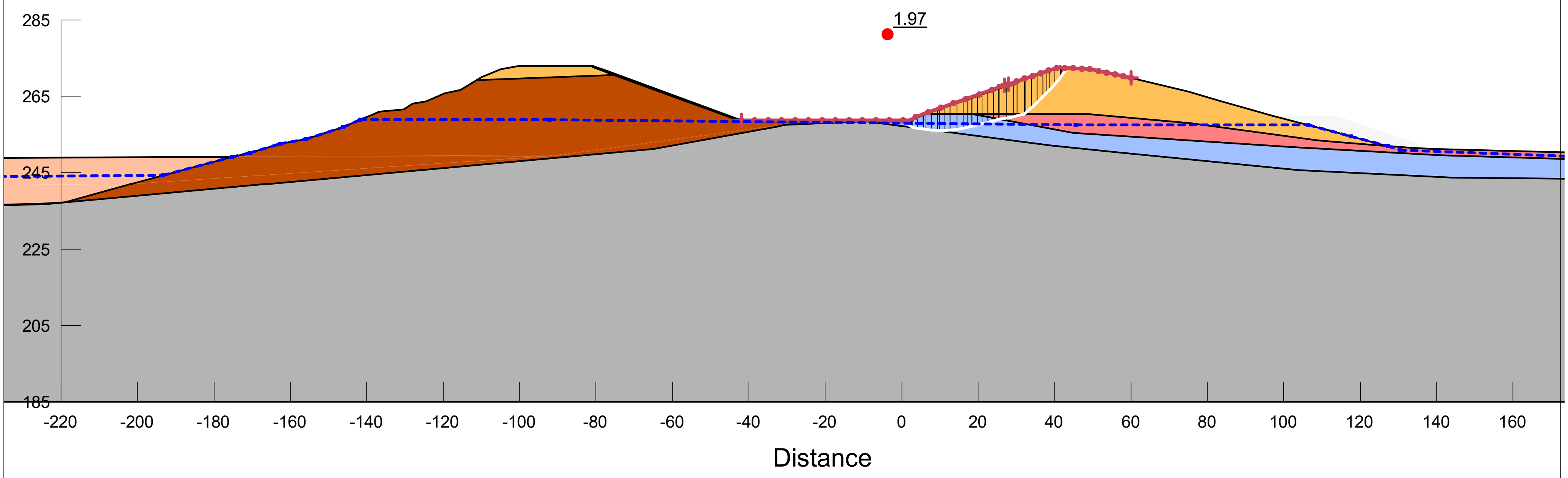
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50				1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 4. Post-Seismic + Cyclic Softening - East U/S (undrained)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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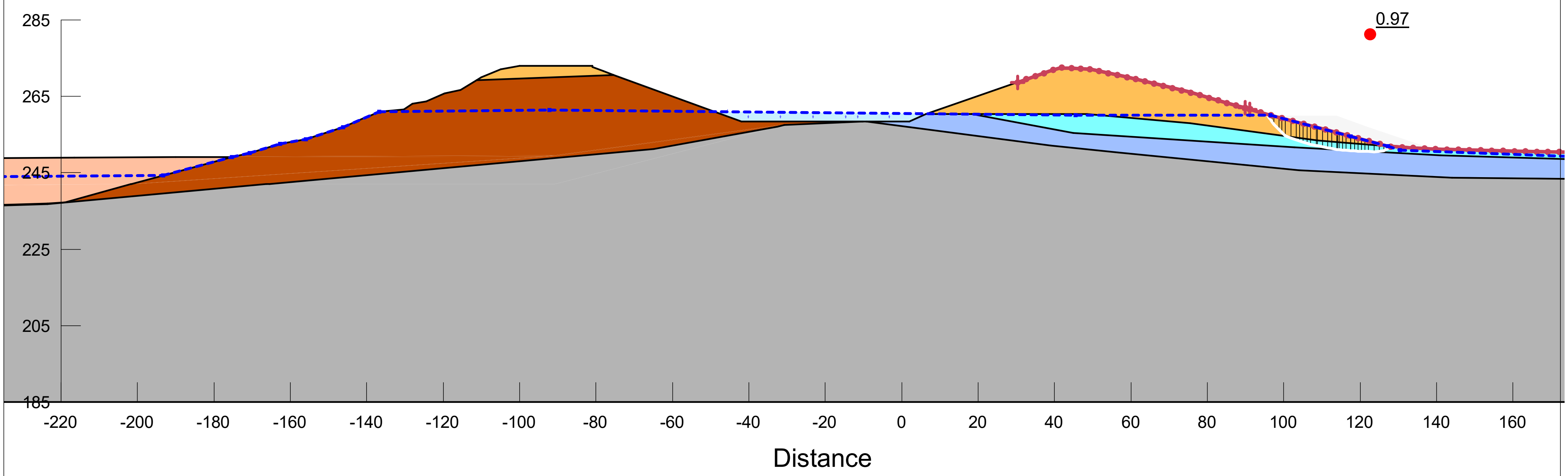
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 4. Post-Seismic + Cyclic Softening - East D/S (drained)	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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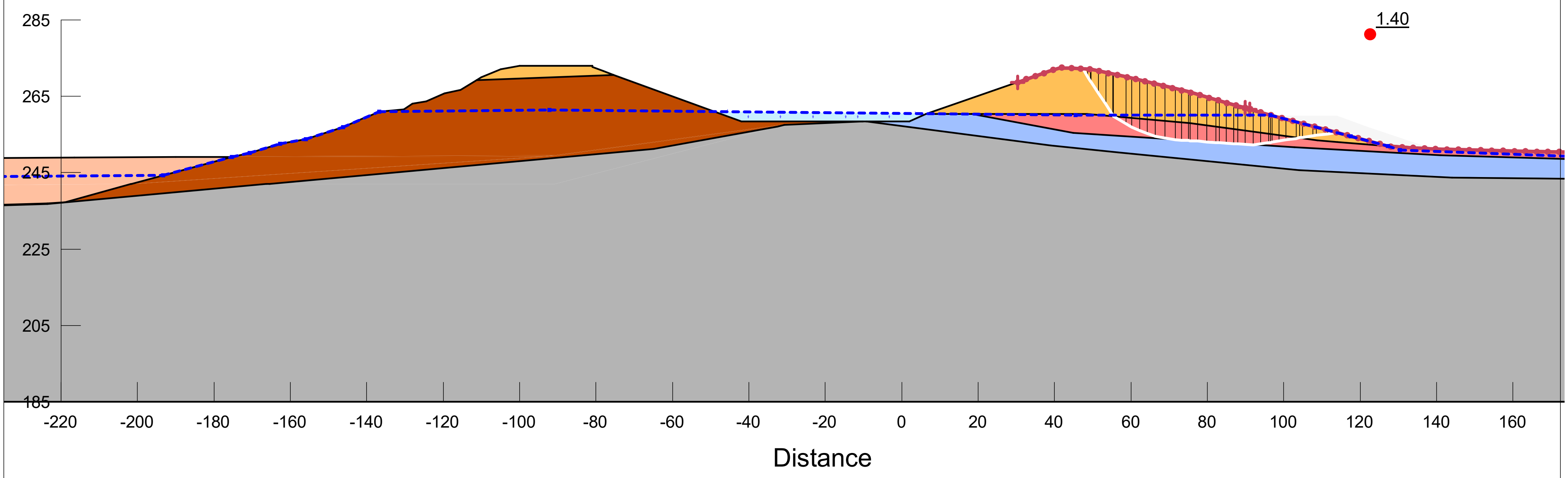
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50				1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 4. Post-Seismic + Cyclic Softening - East D/S (undrained)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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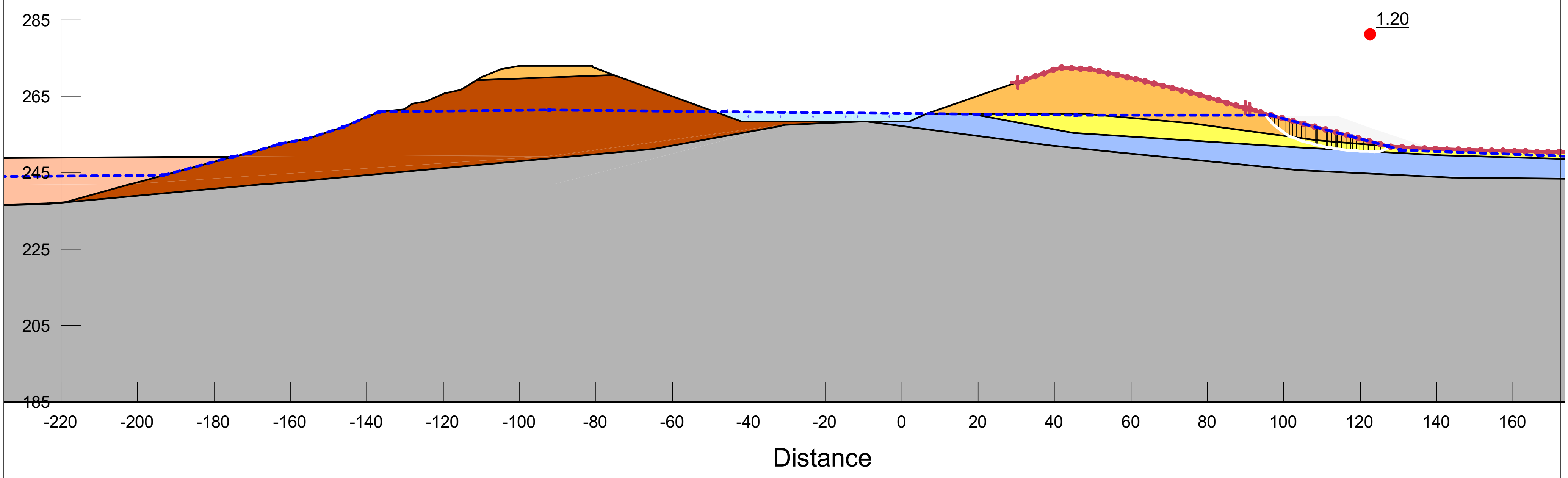
Eastern Embankment Stability
Sensitivity Cases
Analysed By: CHEV
Checked By: ABL

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1a. Long-Term Stability, Liner damage - East D/S	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

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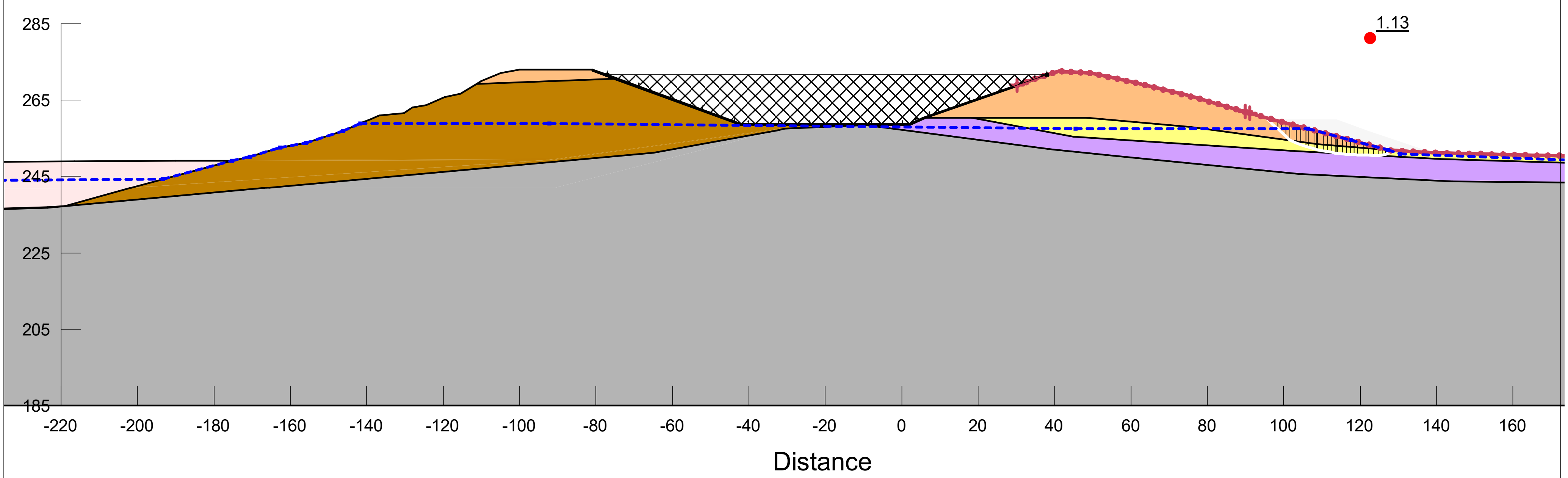
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand) (LB strength)	Mohr-Coulomb	21	0	34	0	1
	Late Pleistocene Alluvium (LB strength)	Mohr-Coulomb	21	0	38	0	1
	Loess (Clayey SILT, some sand) (LB strength)	Mohr-Coulomb	18	1	24	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY) (LB strength)	Mohr-Coulomb	19	3	28	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand) (LB strength)	Mohr-Coulomb	21	0	38	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 1b. Long-Term Stability - East D/S (reduced strengths)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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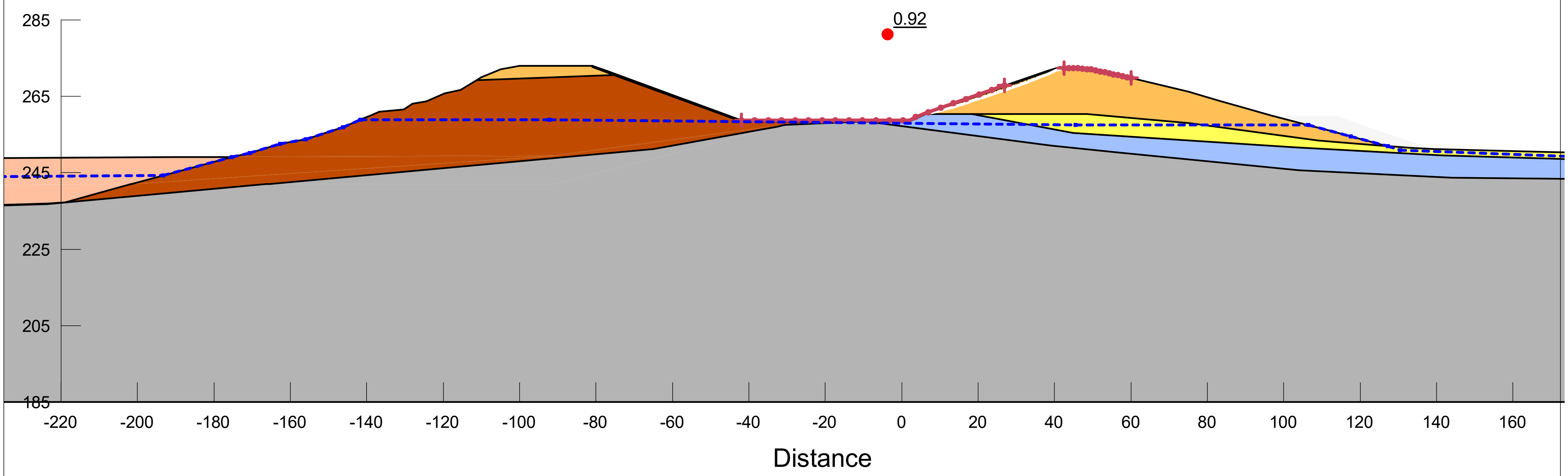
Horz Seismic Coef.: 0.46g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: 2a. Seismic (OBE NSHM) - East U/S	Analysed by: CHEV
Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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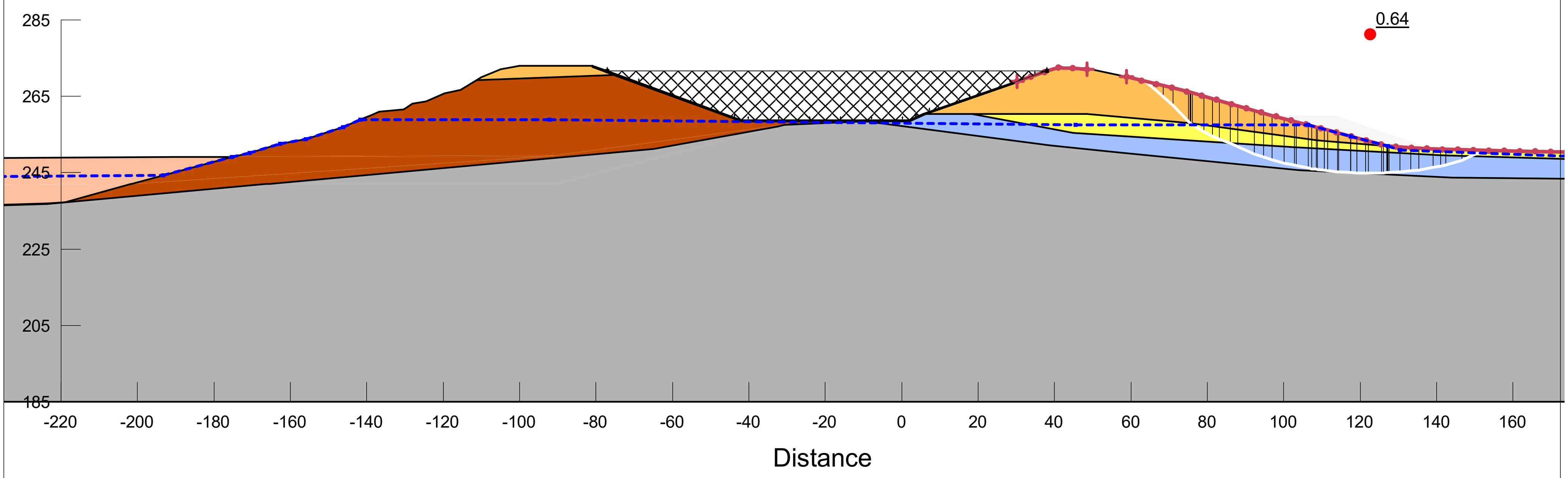
Horz Seismic Coef.: 0.46g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 2a. Seismic (OBE NSHM) - East D/S		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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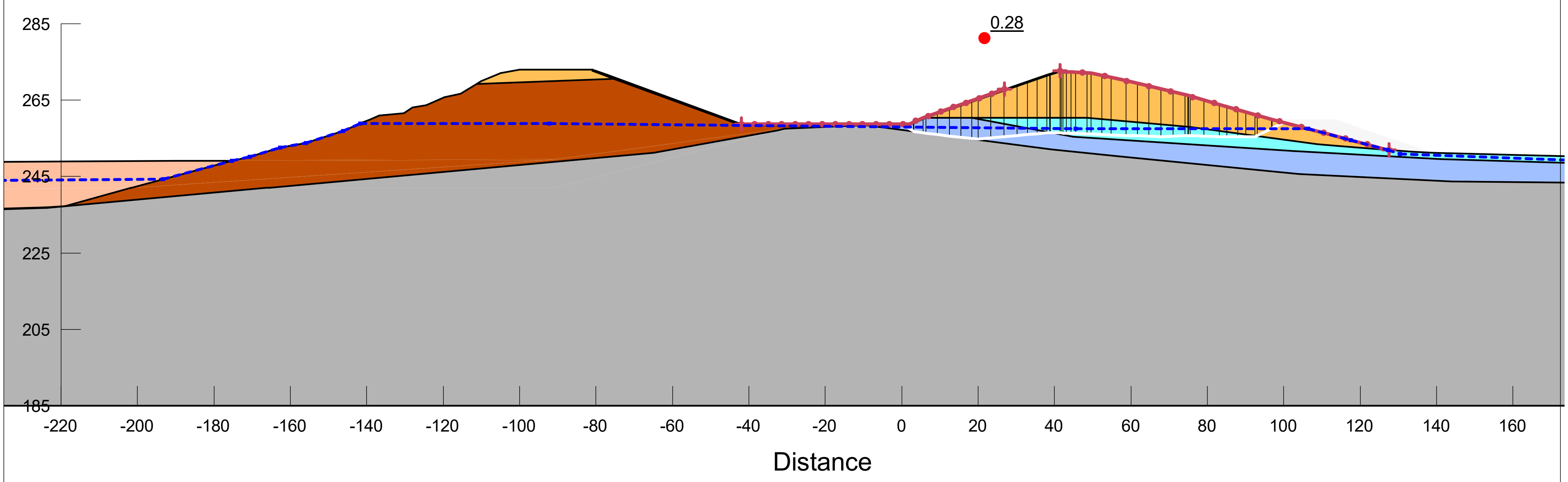
Horz Seismic Coef.: 1.59g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 3a. Seismic (SEE NSHM) + CS - East U/S (drained)	Analysed by: CHEV
	Comments: Scale: 1:1,500 @ A4	Checked by: ABL

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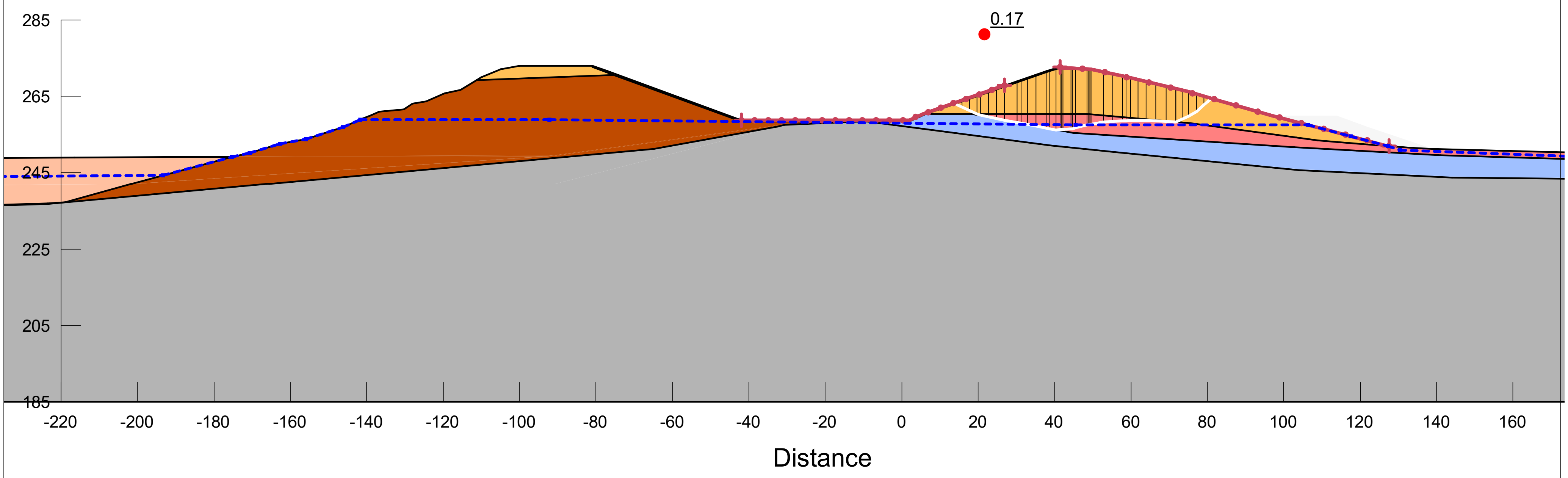
Horz Seismic Coef.: 1.59g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50						1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	0	0	1



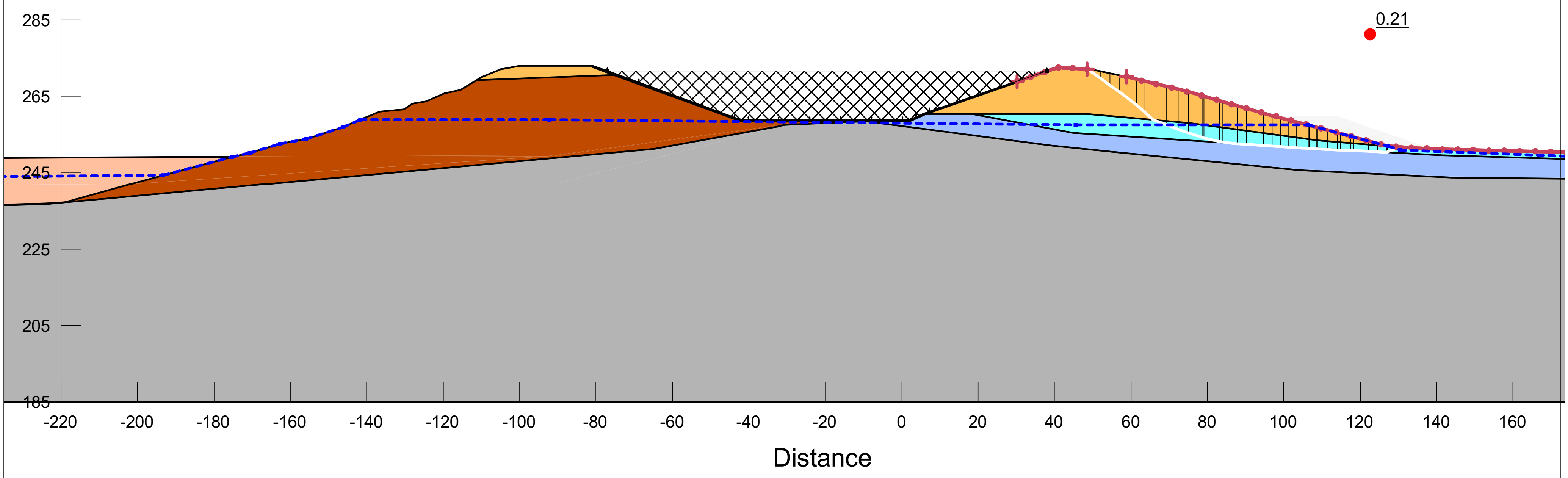
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 3a. Seismic (SEE NSHM) + CS - East U/S (undrained)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.59g

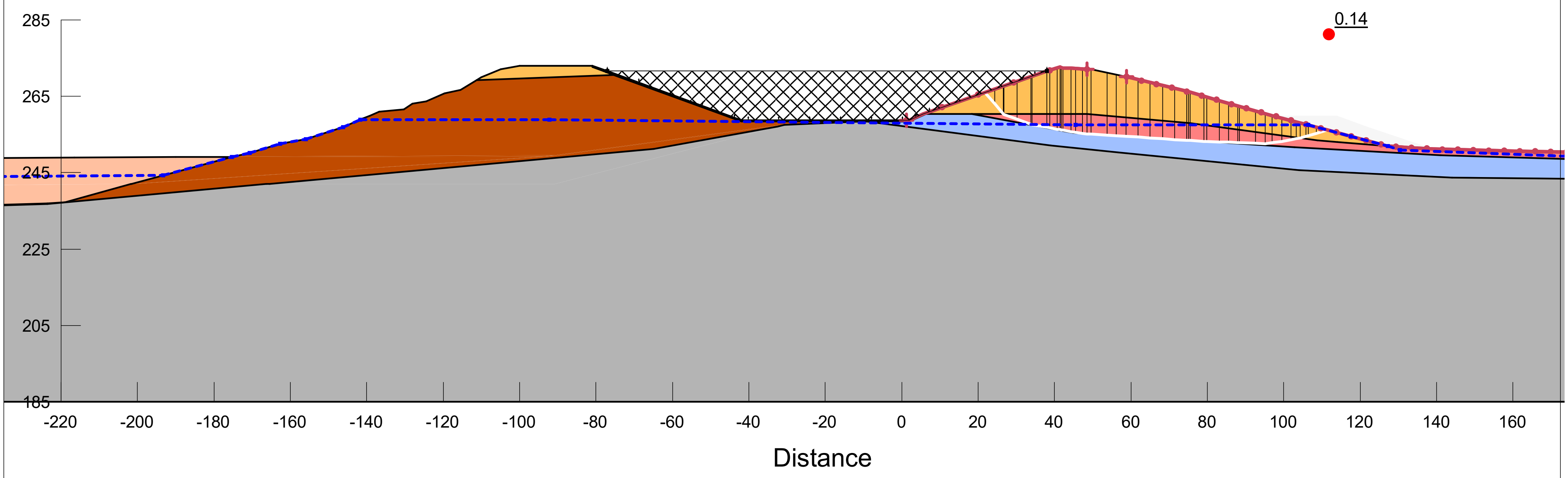
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor	Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
	Analysis: 3a. Seismic (SEE NSHM) + CS - East D/S (drained)	Analysed by: CHEV
	Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

Horz Seismic Coef.: 1.59g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18		3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21		0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21		0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (undrained)	Undrained (Phi=0)	18	50						1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19		6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20		50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21		0	41	0	0	0	1



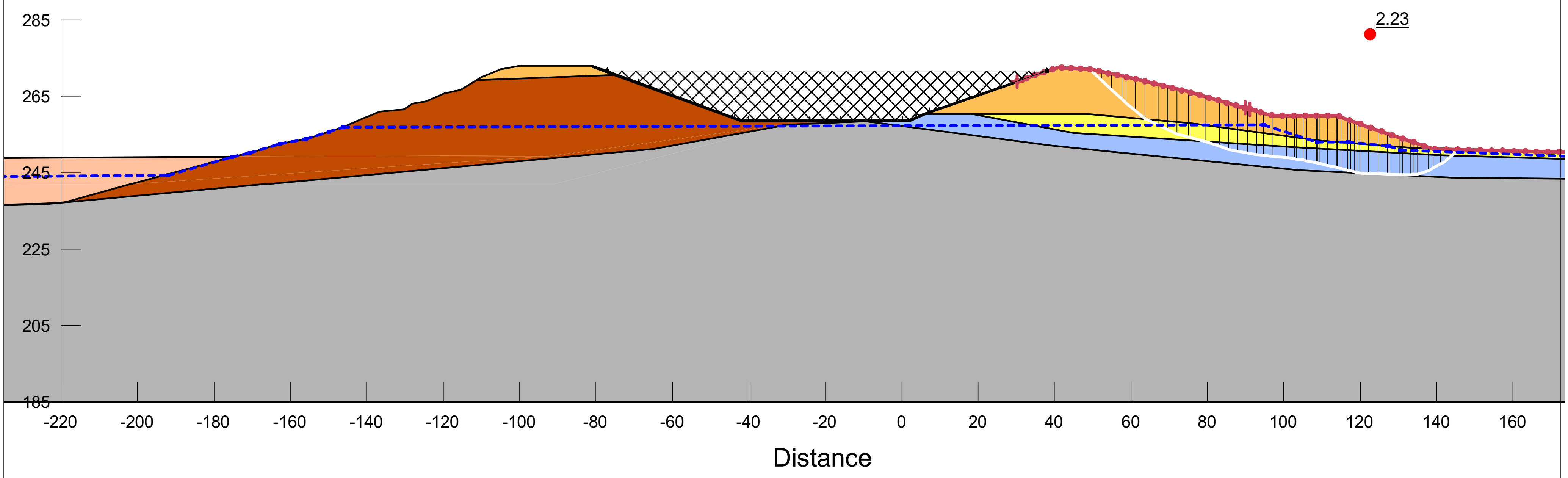
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz	Job Number: 1020688.4000
Analysis: 3a. Seismic (SEE NSHM) + CS - East D/S (undrained)	Analysed by: CHEV
Comments: / Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Eastern Embankment Stability Berm and Drainage Design
Analysed By: CHEV
Checked By: ABL

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 1. Long-Term Stability - East D/S (w drain and berm)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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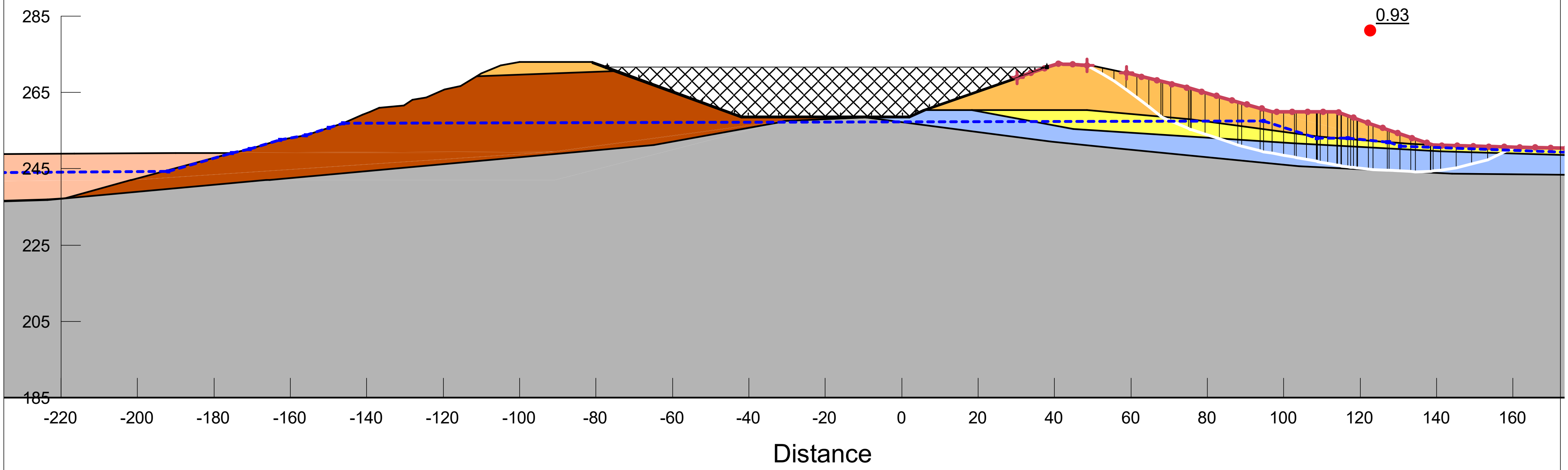
Horz Seismic Coef.: 0.32g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



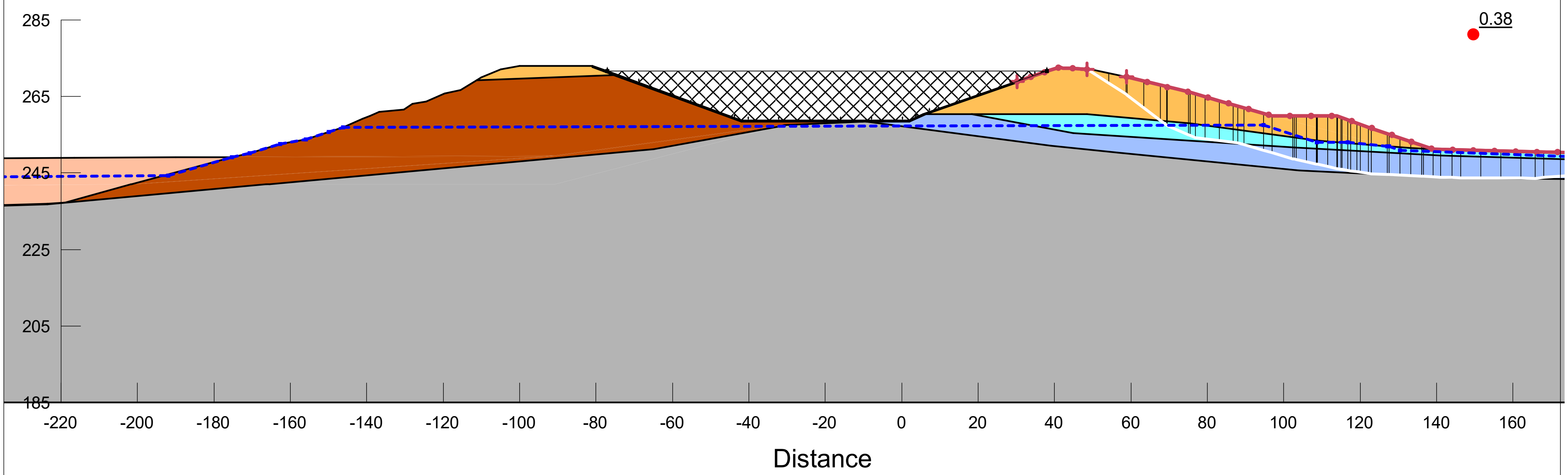
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 2. Seismic (OBE 1170.5) - East D/S (w drain and berm)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 1.01g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz

Analysis: 3. Seismic (SEE 1170.5) + CS - East D/S (w drain, berm)

Comments:

Scale: 1:1,500 @ A4

Job Number: 1020688.4000

Analysed by: CHEV

Checked by: ABL

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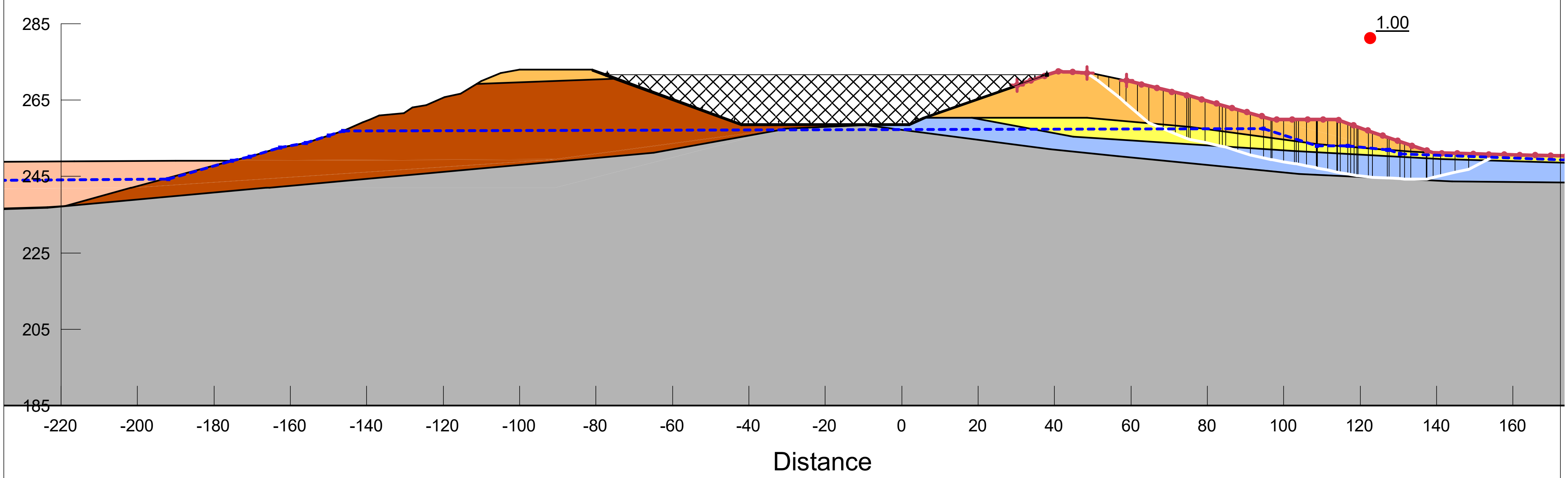
Horz Seismic Coef.: 0.28g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz

Job Number: 1020688.4000

Analysis: Seismic Yield - East D/S (w drain and berm)

Analysed by: CHEV

Comments:

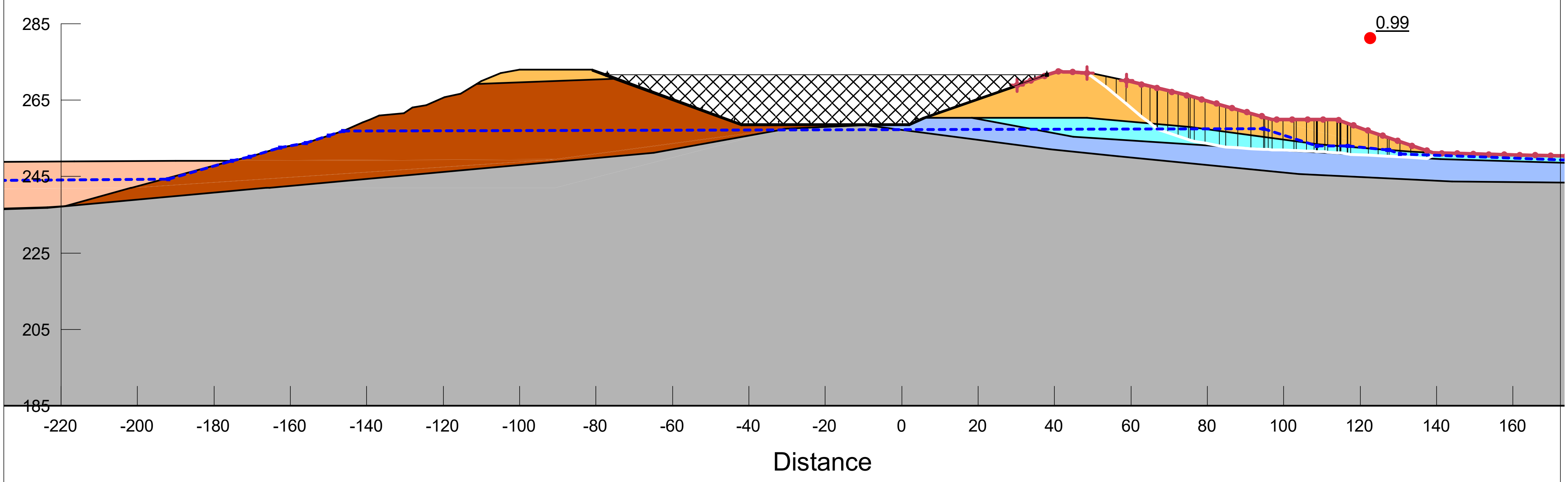
Scale: 1:1,500 @ A4

Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Horz Seismic Coef.: 0.2g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1

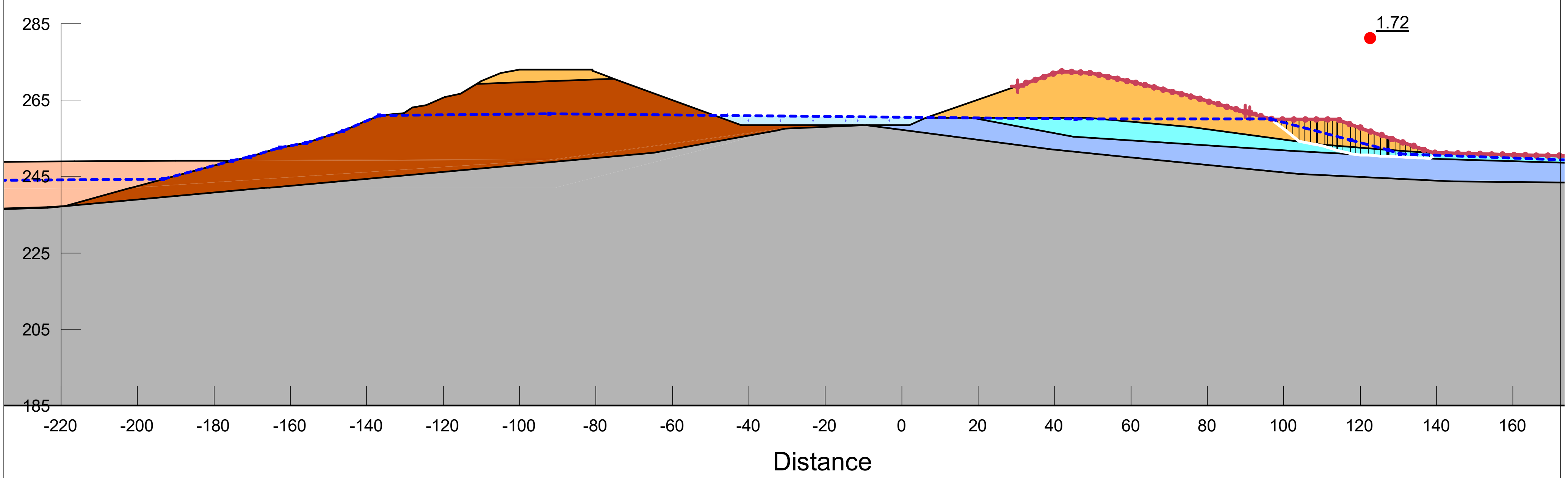


Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: Seismic Yield + CS - East D/S (w drain and berm)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: C:\Users\chev\OneDrive - Tonkin + Taylor Group Ltd\Desktop\

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 4. Post-Seismic + Cyclic Softening - East D/S (w berm)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

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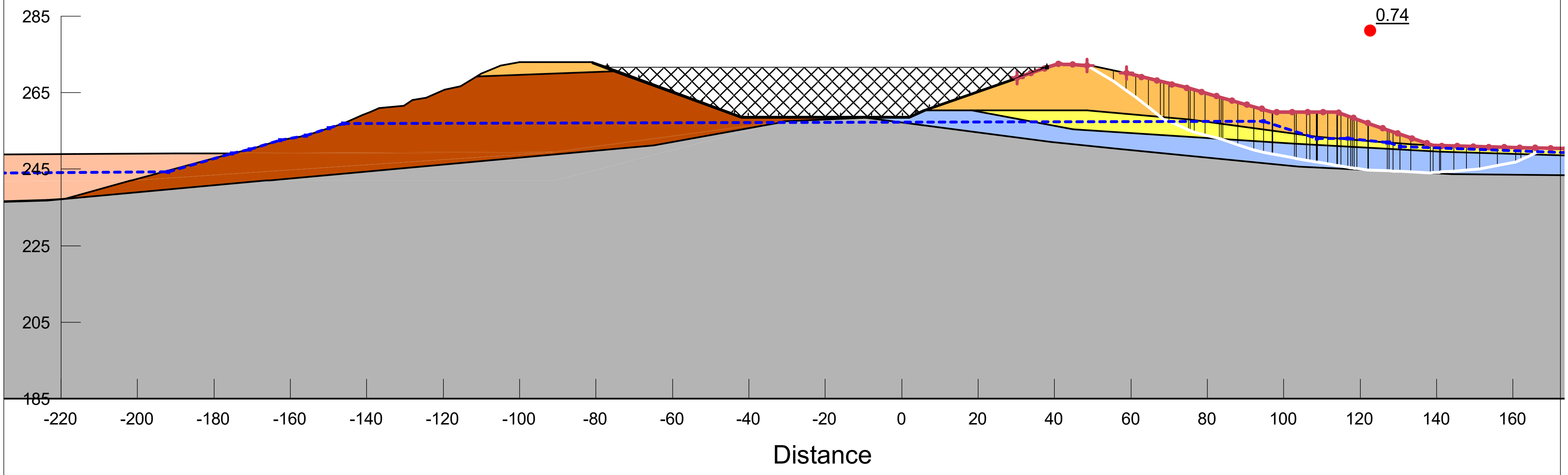
Eastern Embankment Stability
Berm and Drainage Design - Sensitivity Analyses
Analysed By: CHEV
Checked By: ABL

Horz Seismic Coef.: 0.46g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand)	Mohr-Coulomb	18	3	26	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



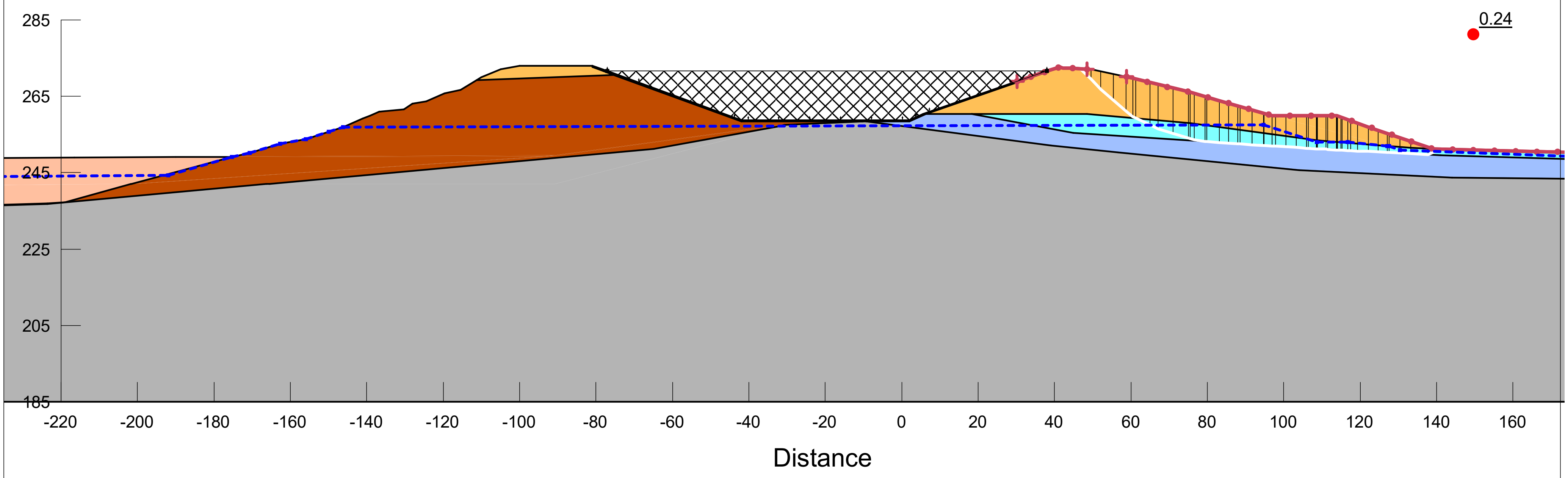
Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz		Job Number: 1020688.4000
Analysis: 2a. Seismic (OBE NSHM) - East D/S (w drain and berm)		Analysed by: CHEV
Comments:	Scale: 1:1,500 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Horz Seismic Coef.: 1.59g

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Clay Liner (Compacted Loess)	Mohr-Coulomb	18	3	26	0	0	0	1
	Embankment Fill (GRAVEL, some sand)	Mohr-Coulomb	21	0	36	0	0	0	1
	Late Pleistocene Alluvium	Mohr-Coulomb	21	0	41	0	0	0	1
	Loess (Clayey SILT, some sand) - residual strength (drained)	Mohr-Coulomb	18	2.4	21	0	0	0	1
	Makirikiri Alluvium (Silty, gravelly CLAY)	Mohr-Coulomb	19	6	30	0	0	0	1
	Mangaheia Group (Sandstone/Siltstone)	Mohr-Coulomb	20	50	35	0	0	0	1
	Tamaki Alluvium (GRAVEL, some sand)	Mohr-Coulomb	21	0	41	0	0	0	1



Tonkin+Taylor

Title: Dannevirke Dam_Rev4.gsz

Analysis: 3a. Seismic (SEE NSHM) + CS - East D/S (w drain, berm)

Comments:

Scale: 1:1,500 @ A4

Job Number: 1020688.4000

Analysed by: CHEV

Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Western and Eastern Seismic Slope Displacement Estimates
Analysed By: CHEV
Checked By: ABL

WESTERN DAM EMBANKMENT - DOWNSTREAM SLOPE
OBE NZS1170.5 Ky=0.29g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes

by Jorge Macedo, Jonathan D. Bray, and Chenying Liu

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.290		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	0.562	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

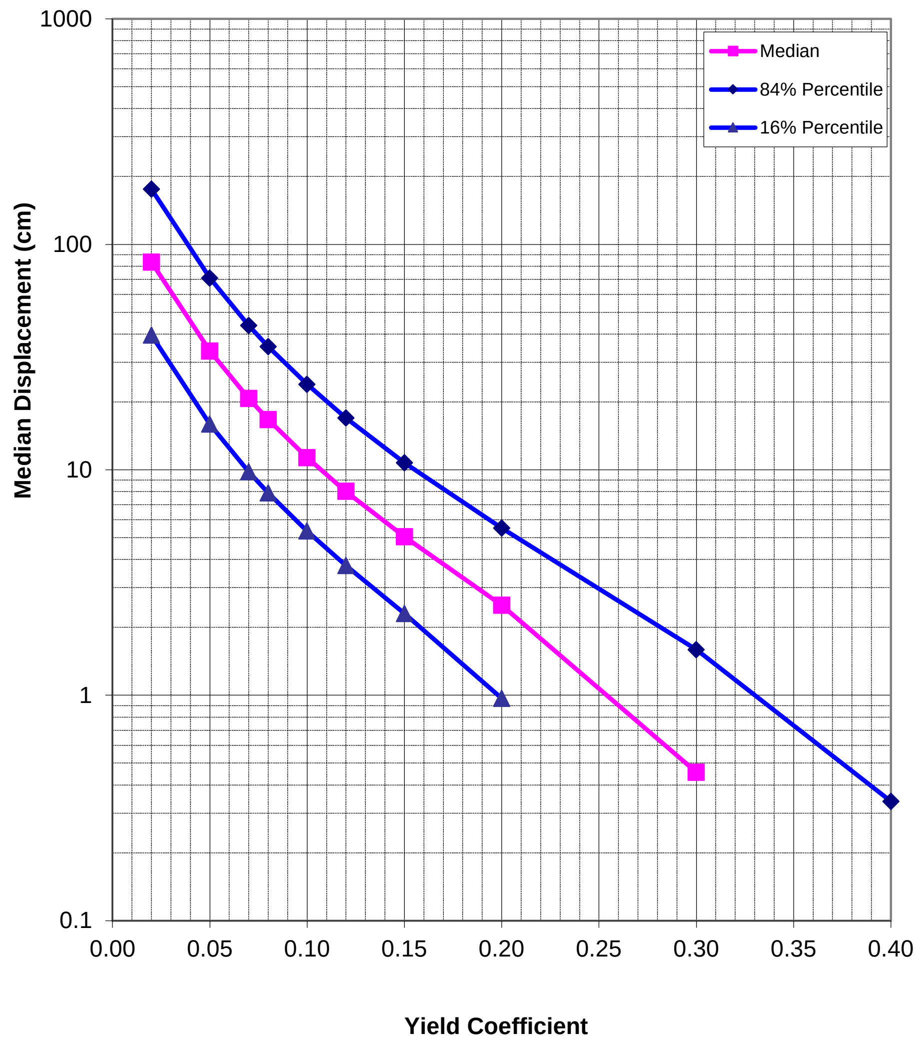
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	1.11	cm	eq. 3a or 3b	0.1026417
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.38**		eq. 2
D1	**<0.5**	cm	calc. using eq. 1
D2	**0.6**	cm	calc. using eq. 1
D3	**1.8**	cm	calc. using eq. 1
P(D>d_threshold)	**0.00**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	83.5	83.5	176.0	39.6
0.05	0.000	33.6	33.6	70.8	15.9
0.07	0.001	20.7	20.7	43.6	9.8
0.08	0.001	16.7	16.7	35.2	7.9
0.1	0.003	11.4	11.3	23.9	5.3
0.12	0.007	8.1	8.0	17.0	3.8
0.15	0.021	5.1	5.0	10.7	2.3
0.2	0.084	2.7	2.5	5.5	1.0
0.3	0.417	1.0	0.5	1.6	<1
0.4	0.761	0.5	<1	0.3	<1



Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

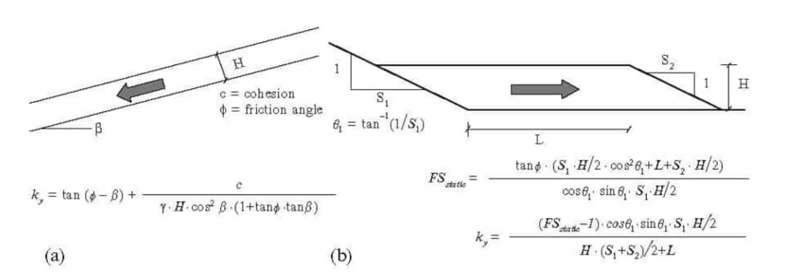


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

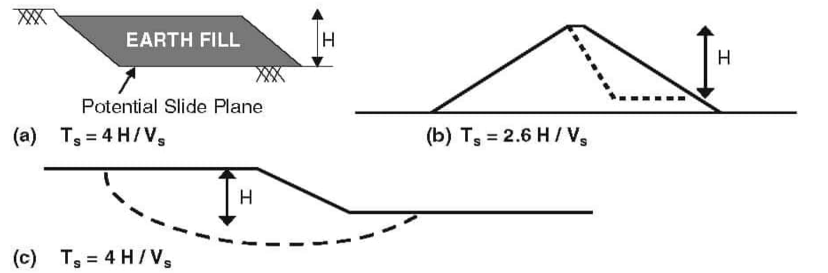


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

WESTERN DAM EMBANKMENT - UPSTREAM SLOPE
SEE NZS1170.5 Ky=0.5g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes
by Jorge Macedo, Jonathan D. Bray, and Chenying Liu
Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET
If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.500		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	1.734	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

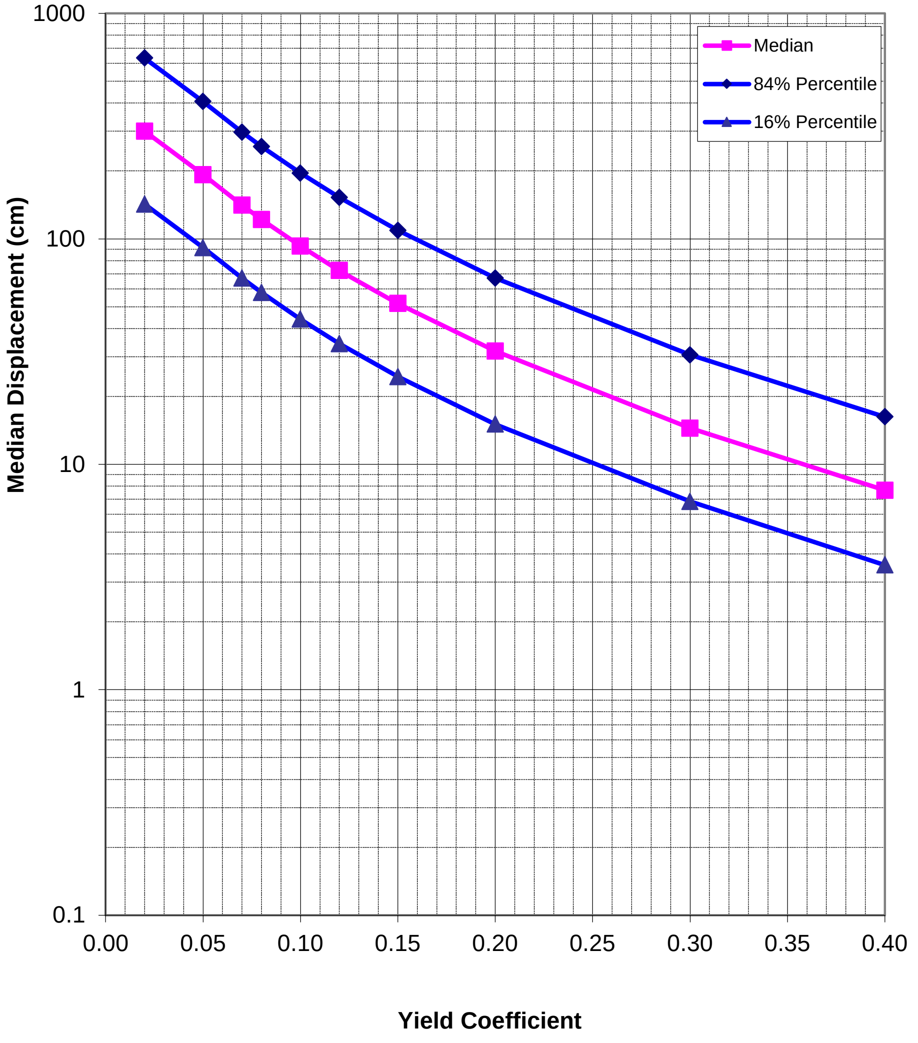
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	4.57	cm	eq. 3a or 3b	1.5201089
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.03**		eq. 2
D1	**2.0**	cm	calc. using eq. 1
D2	**4.4**	cm	calc. using eq. 1
D3	**9.5**	cm	calc. using eq. 1
P(D>d_threshold)	**0.01**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	300.7	300.7	633.9	142.6
0.05	0.000	192.8	192.8	406.4	91.4
0.07	0.000	141.0	141.0	297.4	66.9
0.08	0.000	121.9	121.9	256.9	57.8
0.1	0.000	92.8	92.8	195.6	44.0
0.12	0.000	72.3	72.3	152.5	34.3
0.15	0.000	51.7	51.7	108.9	24.5
0.2	0.000	31.8	31.8	67.0	15.1
0.3	0.002	14.5	14.5	30.5	6.8
0.4	0.010	7.7	7.7	16.2	3.6



Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

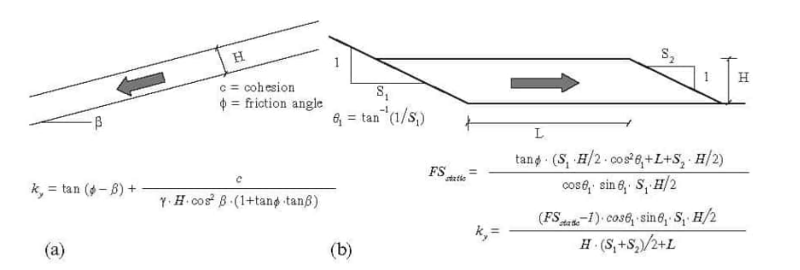


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

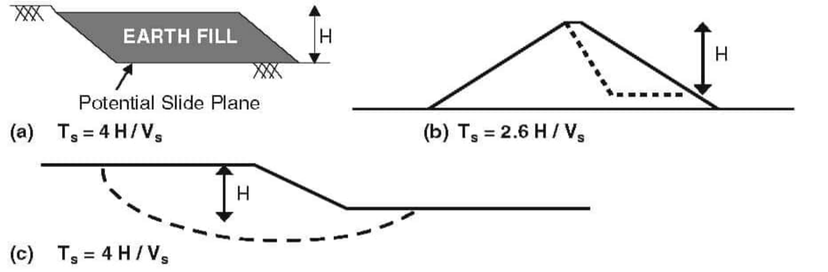


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

WESTERN DAM EMBANKMENT - DOWNSTREAM SLOPE
SEE NZS1170.5 Ky=0.29g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes
by Jorge Macedo, Jonathan D. Bray, and Chenying Liu
Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET
If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.290		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	1.734	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

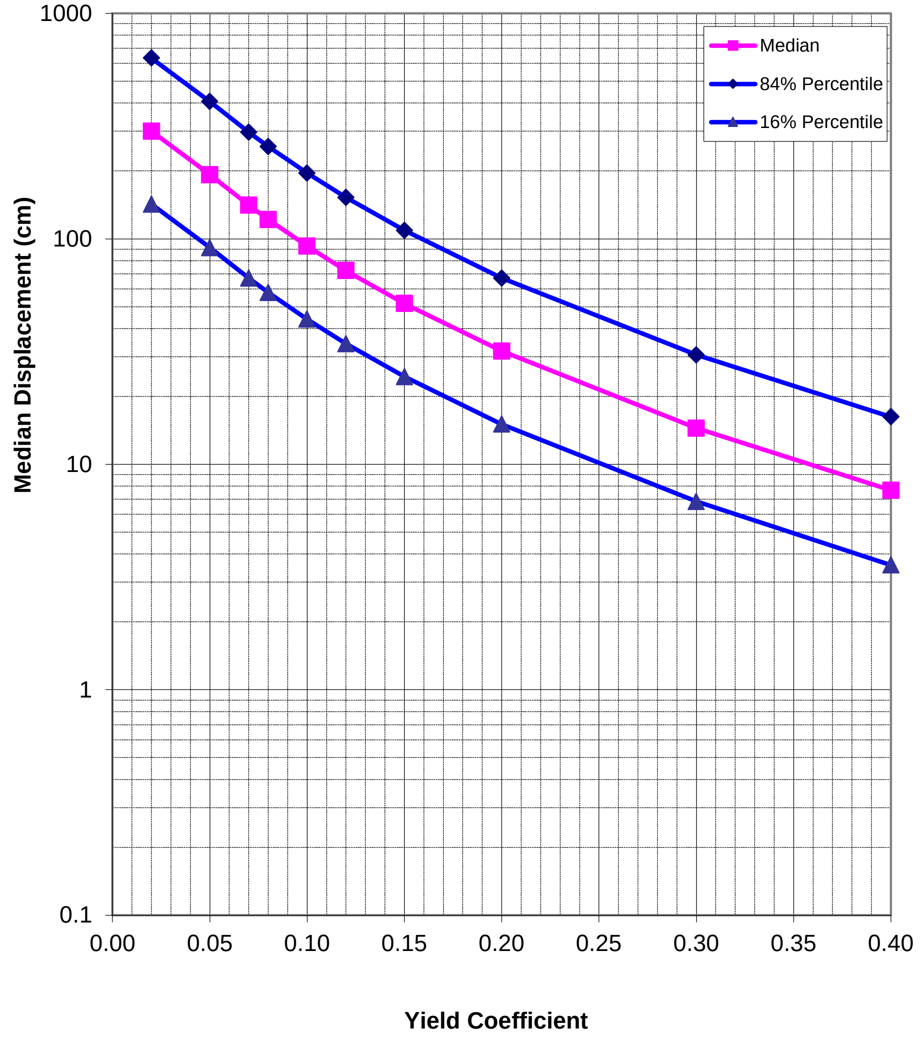
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	15.55	cm	eq. 3a or 3b	2.7443065
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**7.3**	cm	calc. using eq. 1
D2	**15.5**	cm	calc. using eq. 1
D3	**32.8**	cm	calc. using eq. 1
P(D>d_threshold)	**0.19**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	300.7	300.7	633.9	142.6
0.05	0.000	192.8	192.8	406.4	91.4
0.07	0.000	141.0	141.0	297.4	66.9
0.08	0.000	121.9	121.9	256.9	57.8
0.1	0.000	92.8	92.8	195.6	44.0
0.12	0.000	72.3	72.3	152.5	34.3
0.15	0.000	51.7	51.7	108.9	24.5
0.2	0.000	31.8	31.8	67.0	15.1
0.3	0.002	14.5	14.5	30.5	6.8
0.4	0.010	7.7	7.7	16.2	3.6



Notes

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3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

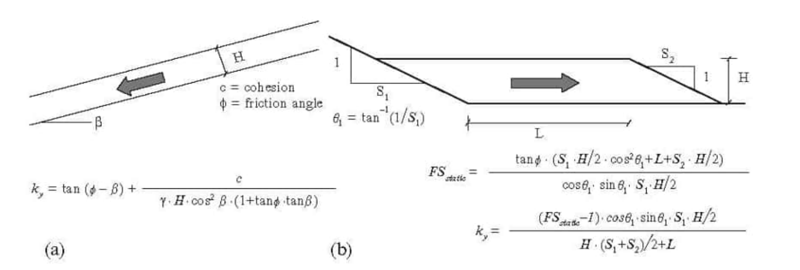

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

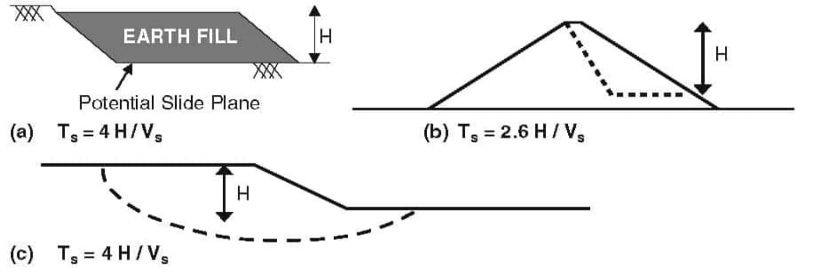

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

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WESTERN DAM EMBANKMENT - DOWNSTREAM SLOPE
OBE NHSM Ky=0.29g

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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.290		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	0.878	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	3.29	cm	eq. 3a or 3b	1.1922985
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.06**		eq. 2
D1	**1.3**	cm	calc. using eq. 1
D2	**3.1**	cm	calc. using eq. 1
D3	**6.7**	cm	calc. using eq. 1
P(D>d_threshold)	**0.00**		eq. 1

Notes
1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

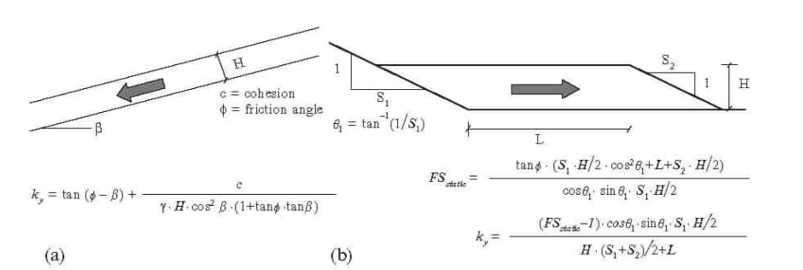


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

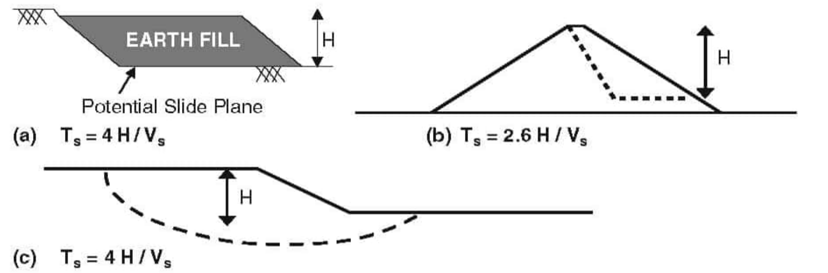
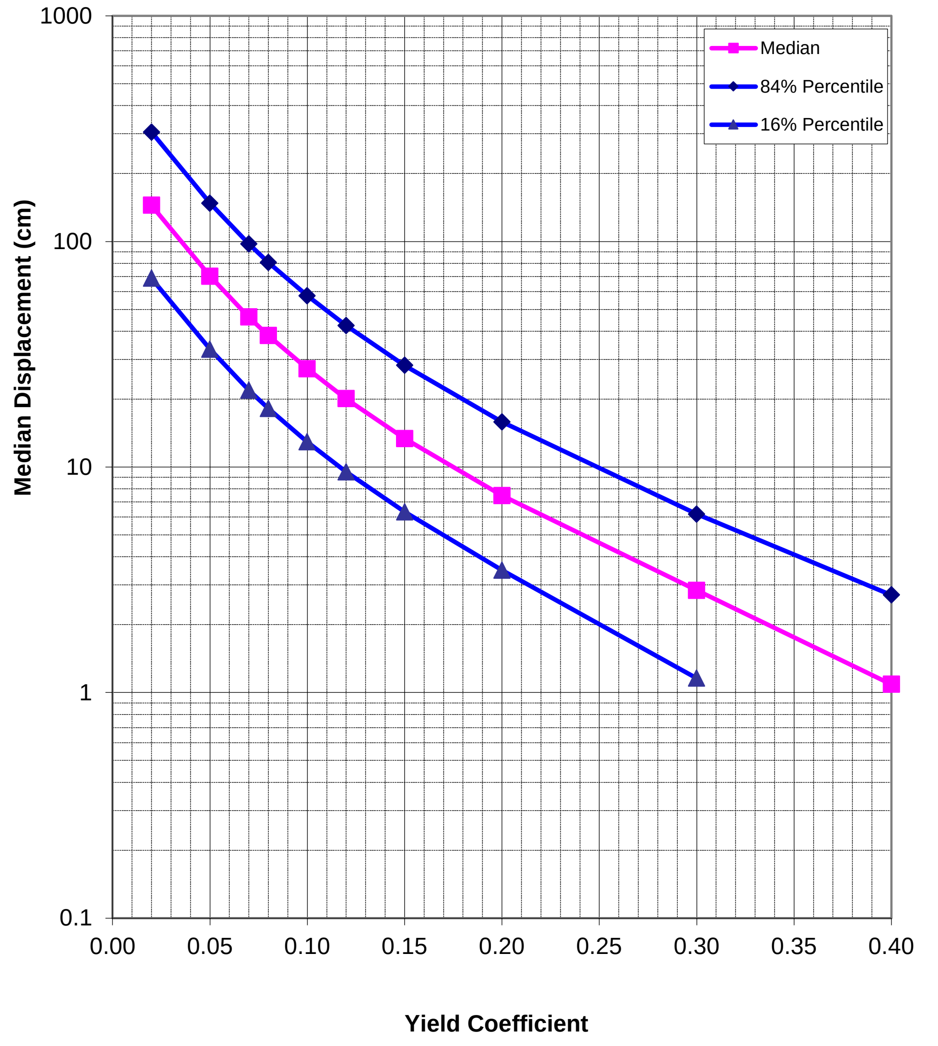


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	144.8	144.8	305.3	68.7
0.05	0.000	70.1	70.1	147.7	33.2
0.07	0.000	46.2	46.2	97.5	21.9
0.08	0.000	38.3	38.3	80.8	18.2
0.1	0.000	27.3	27.3	57.5	12.9
0.12	0.001	20.1	20.1	42.3	9.5
0.15	0.002	13.4	13.4	28.2	6.3
0.2	0.009	7.5	7.5	15.8	3.5
0.3	0.068	3.0	2.8	6.2	1.2
0.4	0.244	1.5	1.1	2.7	<1



WESTERN DAM EMBANKMENT - UPSTREAM SLOPE
SEE NHSM Ky=0.5g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes
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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.500		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.9		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	3.134	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

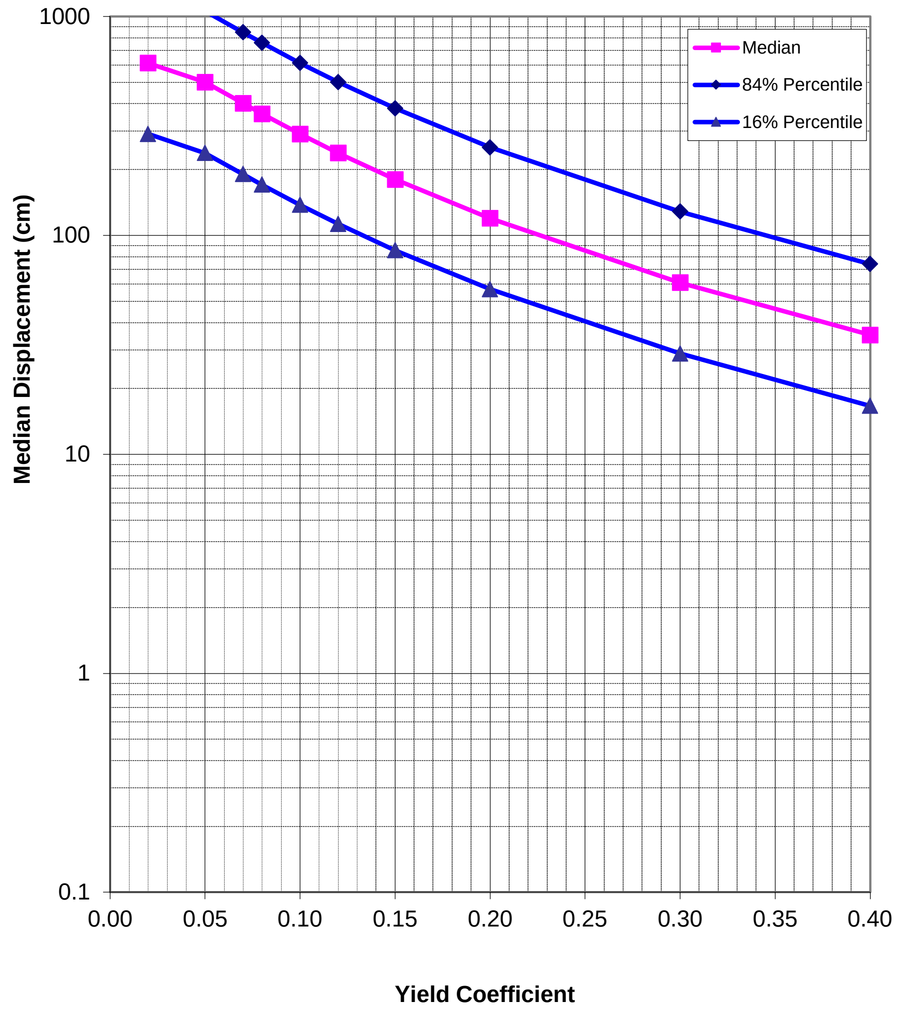
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	22.03	cm	eq. 3a or 3b	3.0922104
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**10.4**	cm	calc. using eq. 1
D2	**22.0**	cm	calc. using eq. 1
D3	**46.4**	cm	calc. using eq. 1
P(D>d_threshold)	**0.34**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	612.9	612.9	1292.2	290.7
0.05	0.000	502.0	502.0	1058.3	238.1
0.07	0.000	401.8	401.8	847.1	190.6
0.08	0.000	359.7	359.7	758.4	170.6
0.1	0.000	290.7	290.7	613.0	137.9
0.12	0.000	238.0	238.0	501.8	112.9
0.15	0.000	180.4	180.4	380.4	85.6
0.2	0.000	119.8	119.8	252.6	56.8
0.3	0.000	60.9	60.9	128.5	28.9
0.4	0.000	35.1	35.1	74.1	16.7



Notes
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2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
(e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

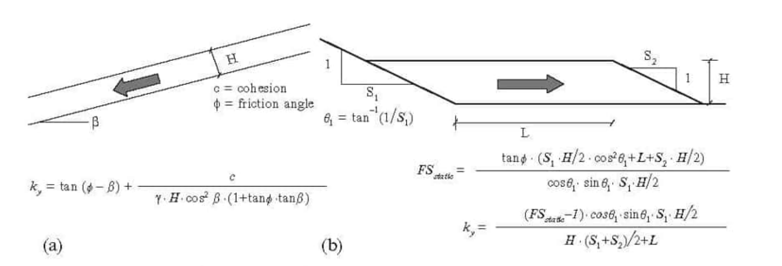

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

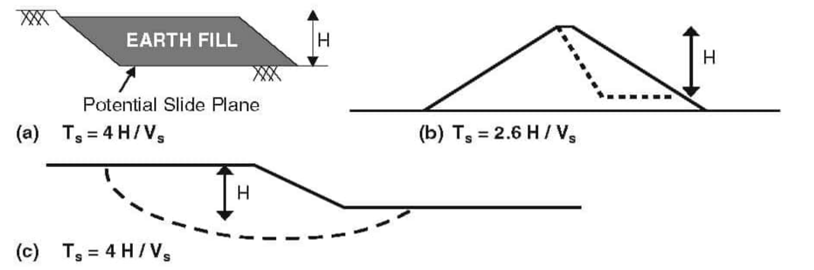

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) “Chapter 14: Simplified Seismic Slope Displacement Procedures,” Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

WESTERN DAM EMBANKMENT - DOWNSTREAM SLOPE
SEE NHSM Ky=0.29g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes

by Jorge Macedo, Jonathan D. Bray, and Chenying Liu

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.290		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.9		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	3.134	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

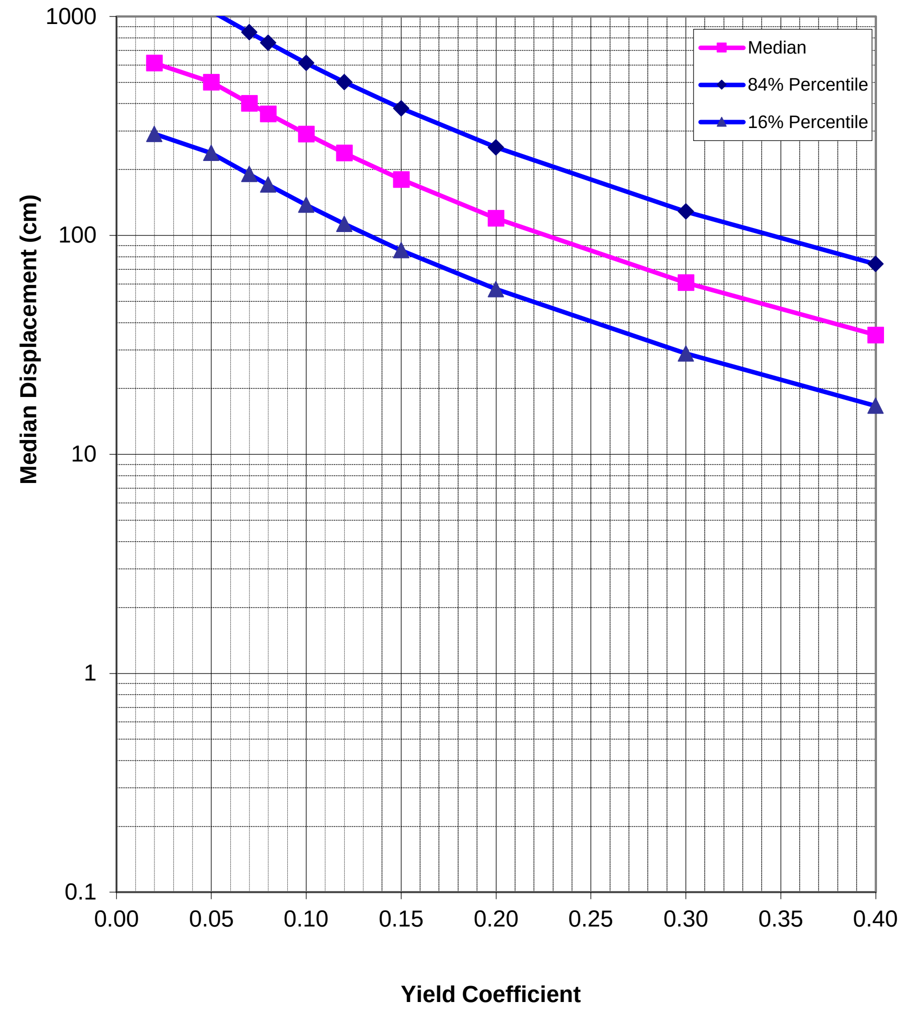
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	64.77	cm	eq. 3a or 3b 4.1708799
Standard Deviation of Non-Zero Seismic D	0.75		

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**30.7**	cm	calc. using eq. 1
D2	**64.8**	cm	calc. using eq. 1
D3	**136.5**	cm	calc. using eq. 1
P(D>d_threshold)	**0.85**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	612.9	612.9	1292.2	290.7
0.05	0.000	502.0	502.0	1058.3	238.1
0.07	0.000	401.8	401.8	847.1	190.6
0.08	0.000	359.7	359.7	758.4	170.6
0.1	0.000	290.7	290.7	613.0	137.9
0.12	0.000	238.0	238.0	501.8	112.9
0.15	0.000	180.4	180.4	380.4	85.6
0.2	0.000	119.8	119.8	252.6	56.8
0.3	0.000	60.9	60.9	128.5	28.9
0.4	0.000	35.1	35.1	74.1	16.7



Notes

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2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

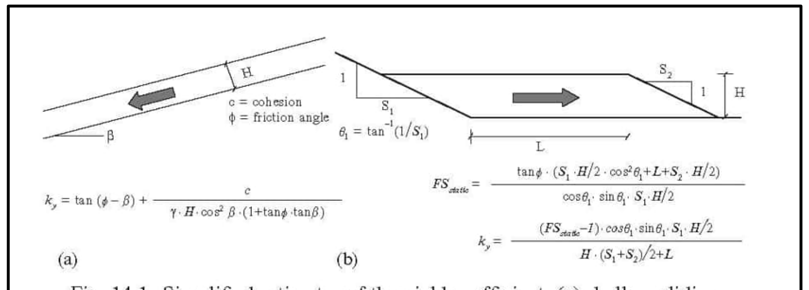

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

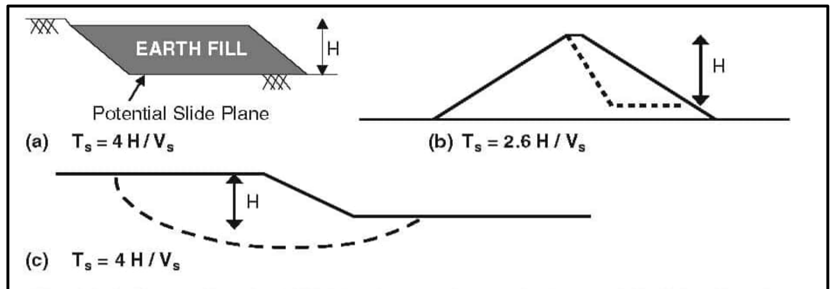

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) “Chapter 14: Simplified Seismic Slope Displacement Procedures,” Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

WESTERN DAM EMBANKMENT - UPSTREAM SLOPE -SOFTENED SURFACE
OBE NZS1170.5 Ky=0.28g

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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.280		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	0.562	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	1.21	cm	eq. 3a or 3b	0.1921456
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.33**		eq. 2
D1	**<0.5**	cm	calc. using eq. 1
D2	**0.7**	cm	calc. using eq. 1
D3	**2.1**	cm	calc. using eq. 1
P(D>d_threshold)	**0.00**		eq. 1

Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

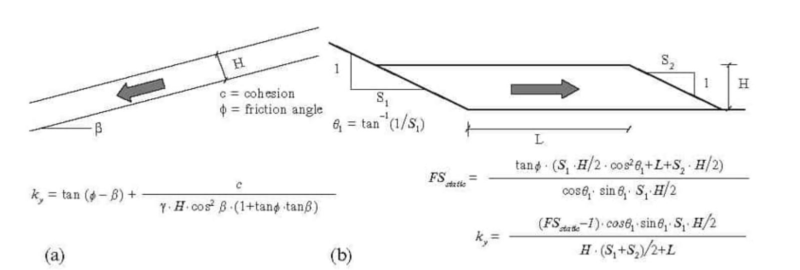


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

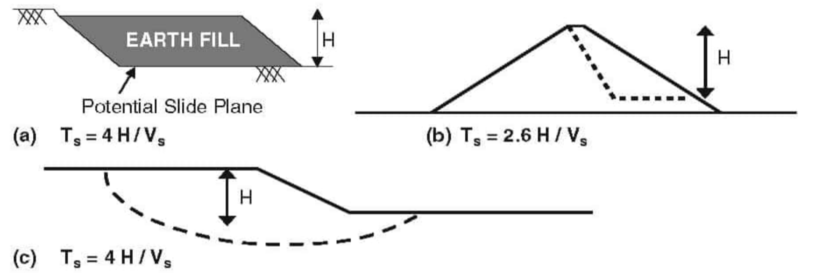
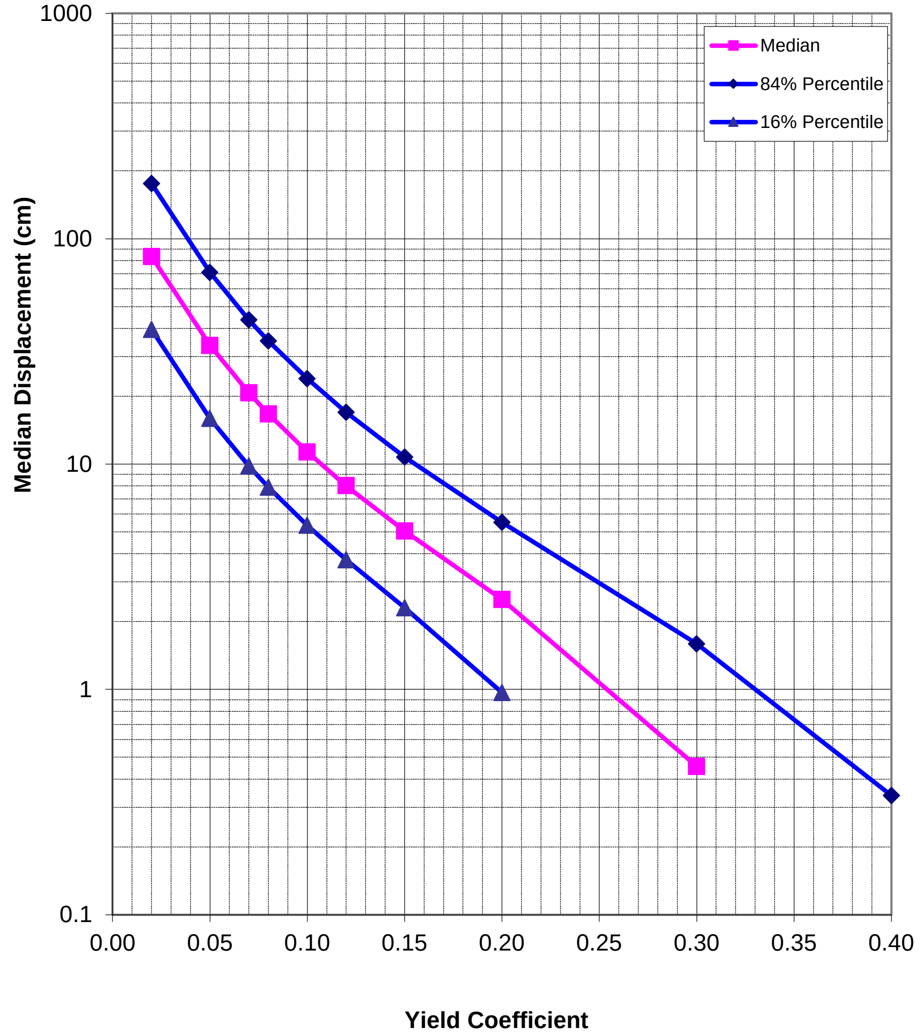


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

Dependence on ky

ky	**P(D="0")**	**D (cm)**	**Dmedian (cm)**	**D-84% (cm)**	**D-16% (cm)**
0.020	0.000	83.5	83.5	176.0	39.6
0.05	0.000	33.6	33.6	70.8	15.9
0.07	0.001	20.7	20.7	43.6	9.8
0.08	0.001	16.7	16.7	35.2	7.9
0.1	0.003	11.4	11.3	23.9	5.3
0.12	0.007	8.1	8.0	17.0	3.8
0.15	0.021	5.1	5.0	10.7	2.3
0.2	0.084	2.7	2.5	5.5	1.0
0.3	0.417	1.0	0.5	1.6	<1
0.4	0.761	0.5	<1	0.3	<1



WESTERN DAM EMBANKMENT - DOWNSTREAM SLOPE -SOFTENED SURFACE
OBE NZS1170.5 Ky=0.14g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes
by Jorge Macedo, Jonathan D. Bray, and Chenying Liu
Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET
If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.140		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	0.562	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

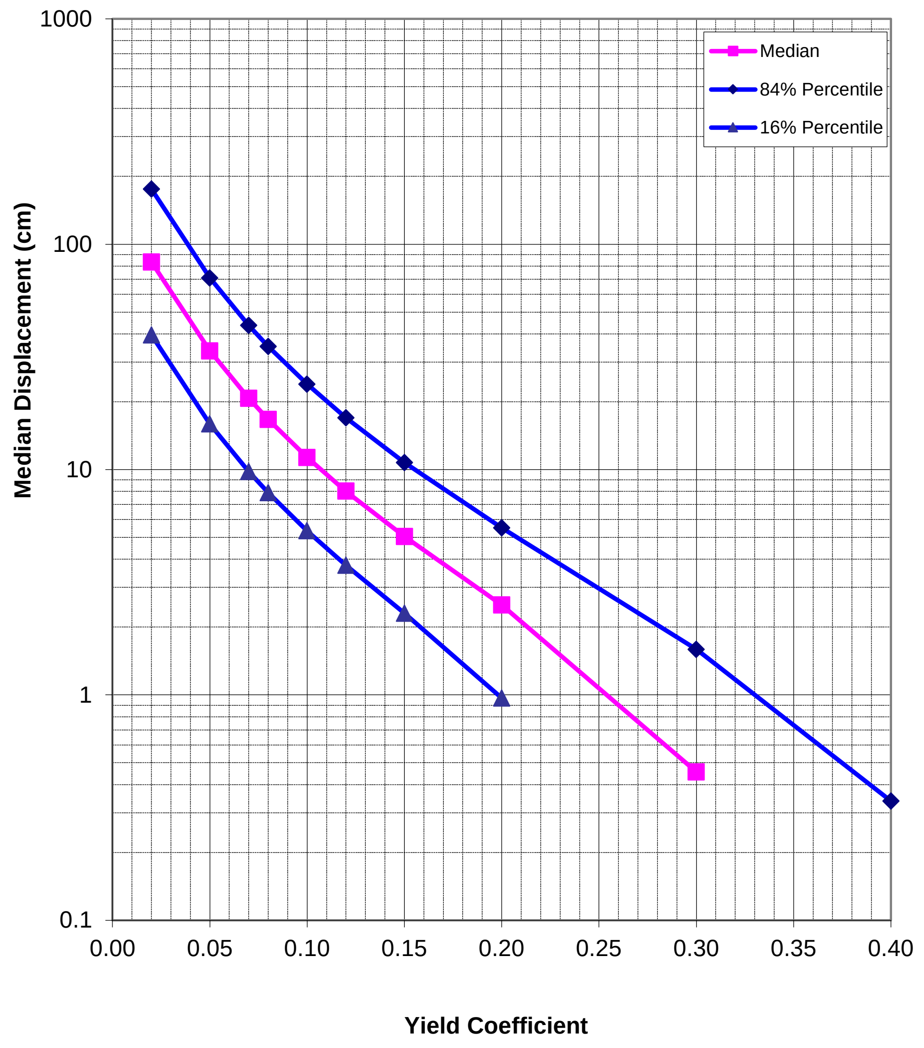
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	5.94	cm	eq. 3a or 3b 1.7813467
Standard Deviation of Non-Zero Seismic D	0.75		

Results			
Probability of Negligible Displ. (P(D=0))	**0.02**		eq. 2
D1	**2.7**	cm	calc. using eq. 1
D2	**5.9**	cm	calc. using eq. 1
D3	**12.4**	cm	calc. using eq. 1
P(D>d_threshold)	**0.02**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	83.5	83.5	176.0	39.6
0.05	0.000	33.6	33.6	70.8	15.9
0.07	0.001	20.7	20.7	43.6	9.8
0.08	0.001	16.7	16.7	35.2	7.9
0.1	0.003	11.4	11.3	23.9	5.3
0.12	0.007	8.1	8.0	17.0	3.8
0.15	0.021	5.1	5.0	10.7	2.3
0.2	0.084	2.7	2.5	5.5	1.0
0.3	0.417	1.0	0.5	1.6	<1
0.4	0.761	0.5	<1	0.3	<1



Notes

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3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

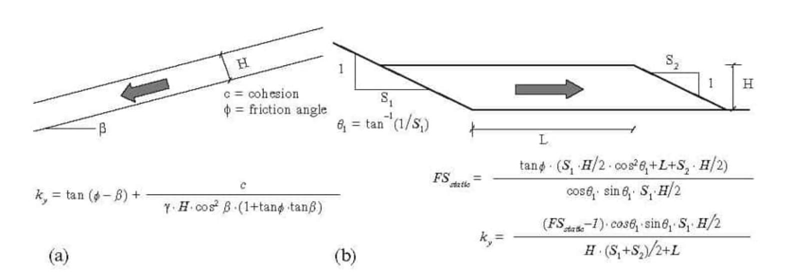

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

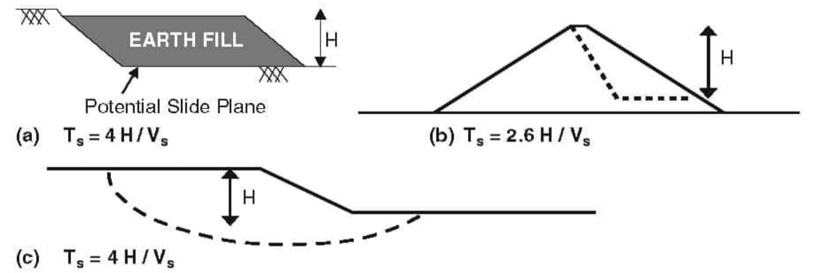

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

WESTERN DAM EMBANKMENT - UPSTREAM SLOPE - SOFTENED SURFACE
SEE NZS1170.5 Ky=0.28g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes

by Jorge Macedo, Jonathan D. Bray, and Chenying Liu

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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.280		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	1.734	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

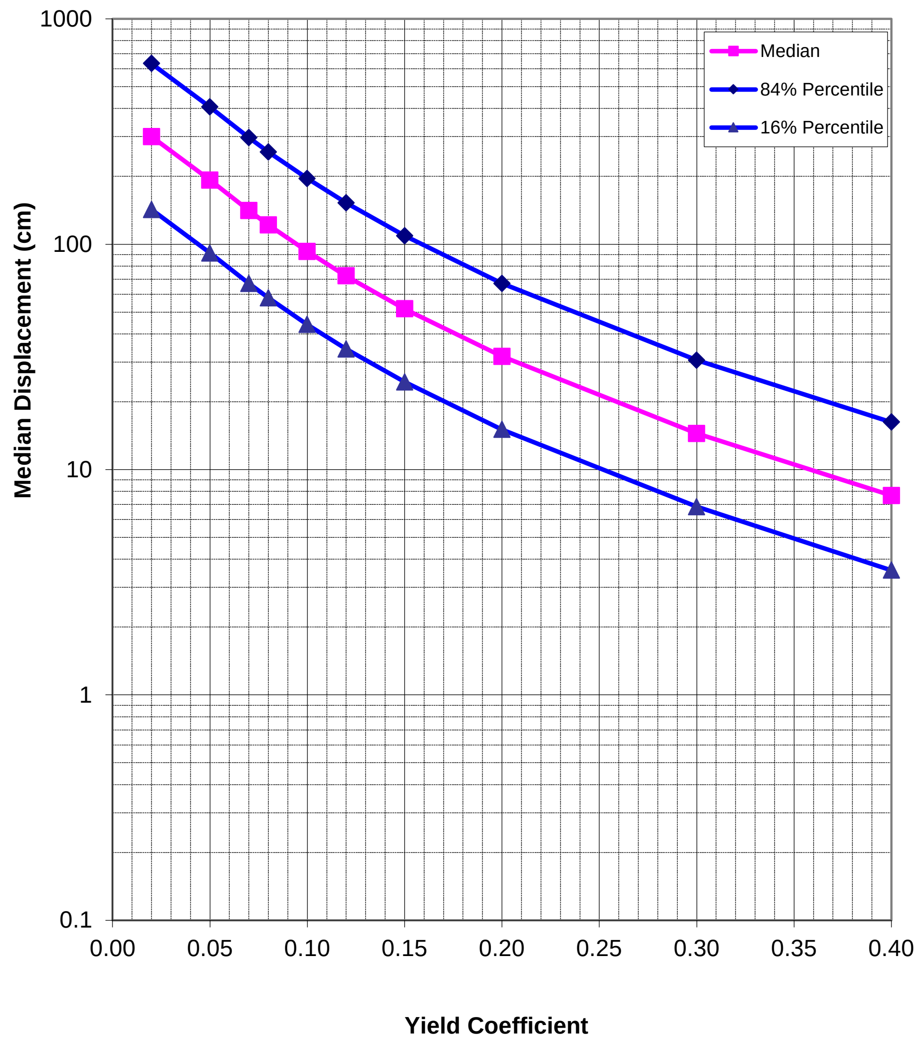
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	16.71	cm	eq. 3a or 3b 2.8159646
Standard Deviation of Non-Zero Seismic D	0.75		

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**7.9**	cm	calc. using eq. 1
D2	**16.7**	cm	calc. using eq. 1
D3	**35.2**	cm	calc. using eq. 1
P(D>d_threshold)	**0.22**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	300.7	300.7	633.9	142.6
0.05	0.000	192.8	192.8	406.4	91.4
0.07	0.000	141.0	141.0	297.4	66.9
0.08	0.000	121.9	121.9	256.9	57.8
0.1	0.000	92.8	92.8	195.6	44.0
0.12	0.000	72.3	72.3	152.5	34.3
0.15	0.000	51.7	51.7	108.9	24.5
0.2	0.000	31.8	31.8	67.0	15.1
0.3	0.002	14.5	14.5	30.5	6.8
0.4	0.010	7.7	7.7	16.2	3.6



Notes

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2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

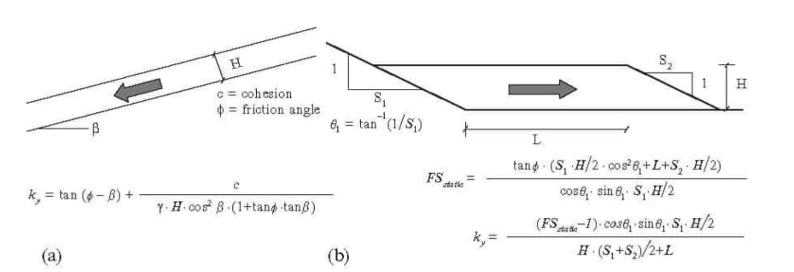

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

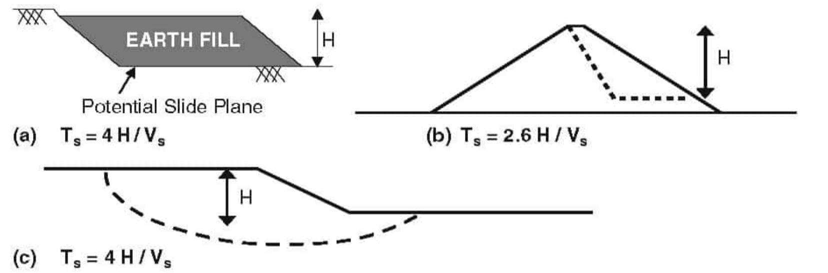

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

WESTERN DAM EMBANKMENT - DOWNSTREAM SLOPE - SOFTENED SURFACE
SEE NZS1170.5 Ky=0.14g

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by Jorge Macedo, Jonathan D. Bray, and Chenying Liu
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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.140		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	1.734	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

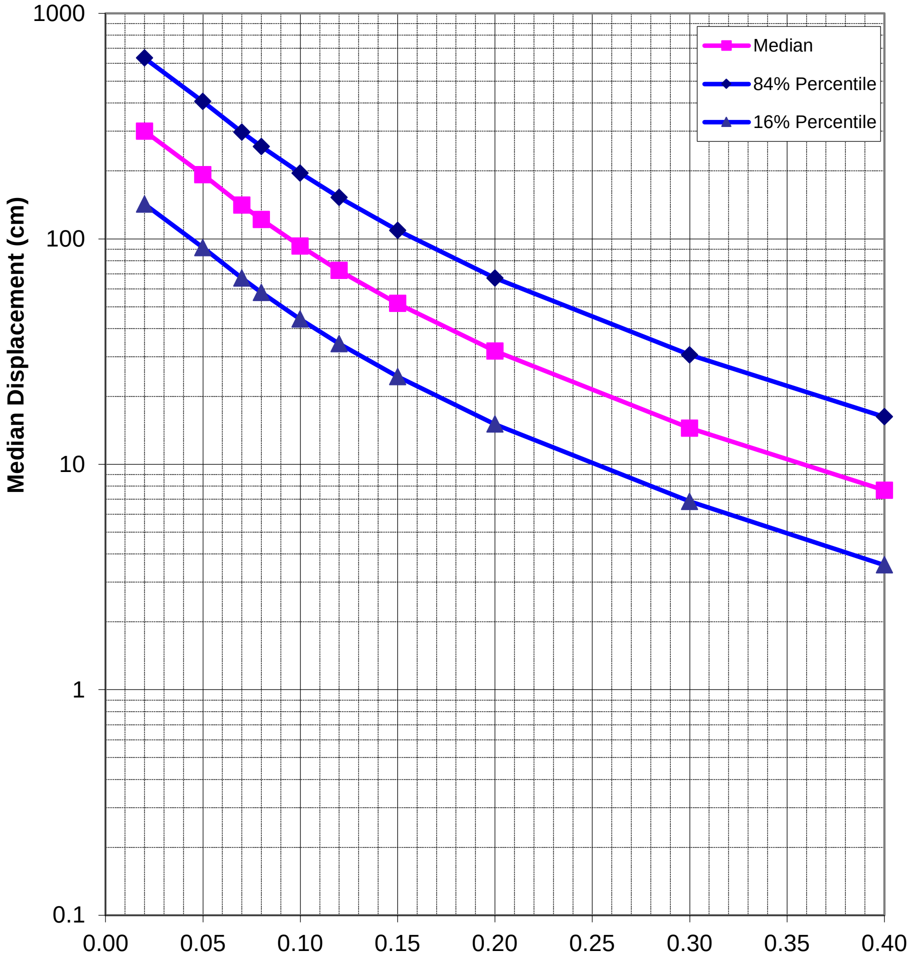
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	57.55	cm	eq. 3a or 3b	4.052663
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**27.3**	cm	calc. using eq. 1
D2	**57.5**	cm	calc. using eq. 1
D3	**121.3**	cm	calc. using eq. 1
P(D>d_threshold)	**0.81**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	300.7	300.7	633.9	142.6
0.05	0.000	192.8	192.8	406.4	91.4
0.07	0.000	141.0	141.0	297.4	66.9
0.08	0.000	121.9	121.9	256.9	57.8
0.1	0.000	92.8	92.8	195.6	44.0
0.12	0.000	72.3	72.3	152.5	34.3
0.15	0.000	51.7	51.7	108.9	24.5
0.2	0.000	31.8	31.8	67.0	15.1
0.3	0.002	14.5	14.5	30.5	6.8
0.4	0.010	7.7	7.7	16.2	3.6



Notes

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3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
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4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

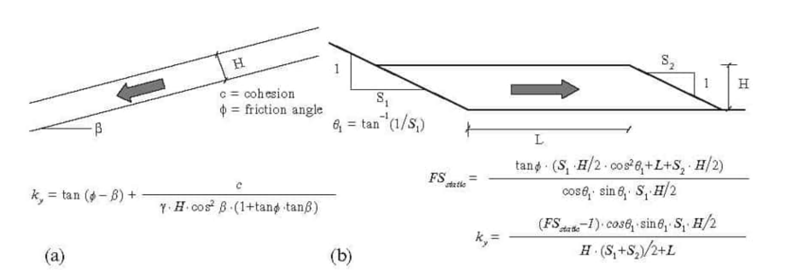

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

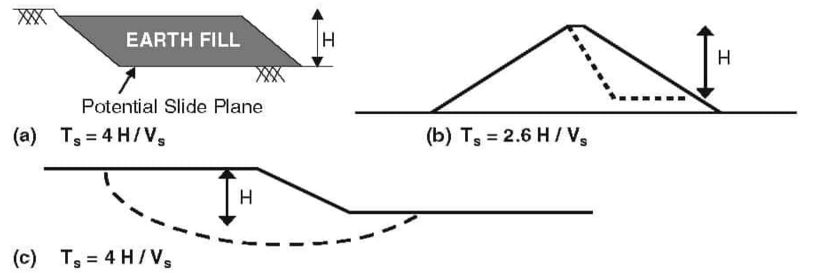

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT - DOWNSTREAM SLOPE
OBE NZS1170.5 Ky=0.21g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes

by Jorge Macedo, Jonathan D. Bray, and Chenying Liu

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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.210		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	0.562	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

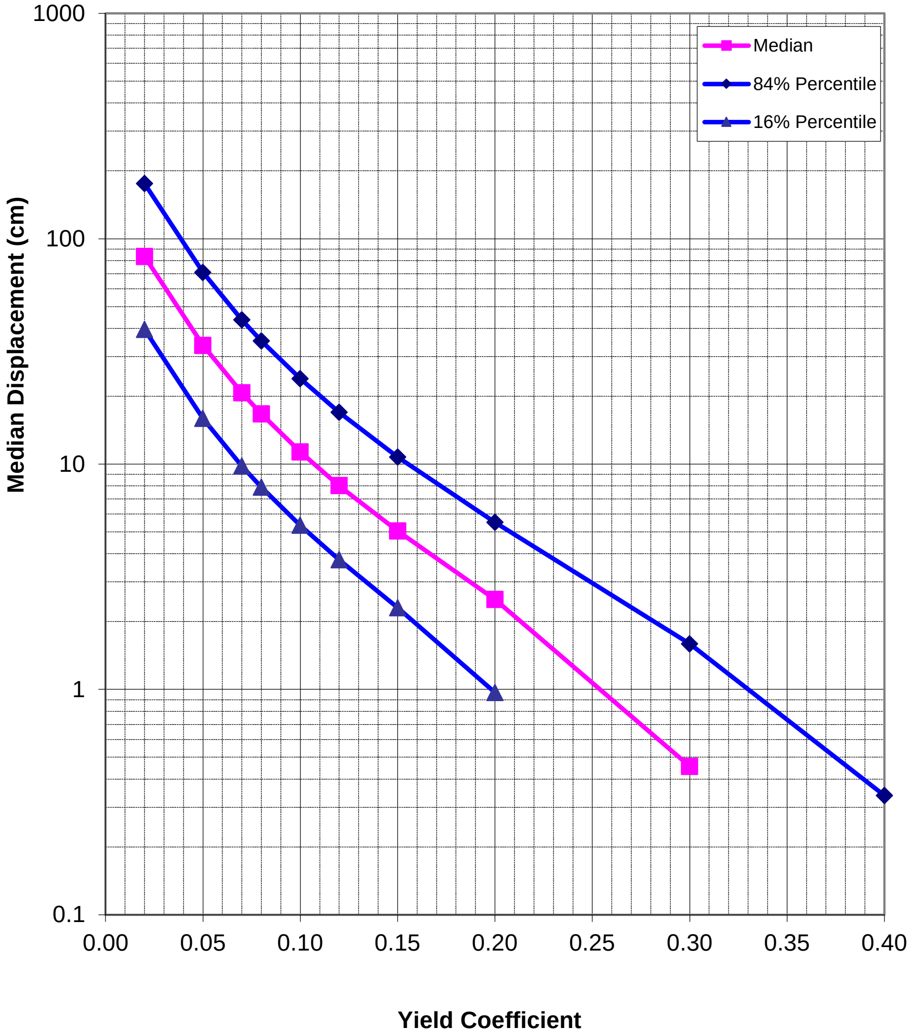
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	2.44	cm	eq. 3a or 3b	0.8930274
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.10**		eq. 2
D1	**0.8**	cm	calc. using eq. 1
D2	**2.2**	cm	calc. using eq. 1
D3	**4.9**	cm	calc. using eq. 1
P(D>d_threshold)	**0.00**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	83.5	83.5	176.0	39.6
0.05	0.000	33.6	33.6	70.8	15.9
0.07	0.001	20.7	20.7	43.6	9.8
0.08	0.001	16.7	16.7	35.2	7.9
0.1	0.003	11.4	11.3	23.9	5.3
0.12	0.007	8.1	8.0	17.0	3.8
0.15	0.021	5.1	5.0	10.7	2.3
0.2	0.084	2.7	2.5	5.5	1.0
0.3	0.417	1.0	0.5	1.6	<1
0.4	0.761	0.5	<1	0.3	<1



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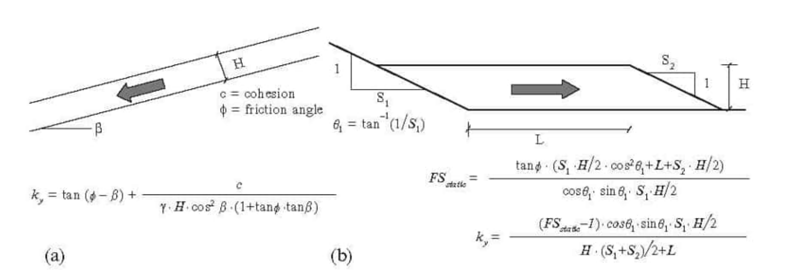
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)



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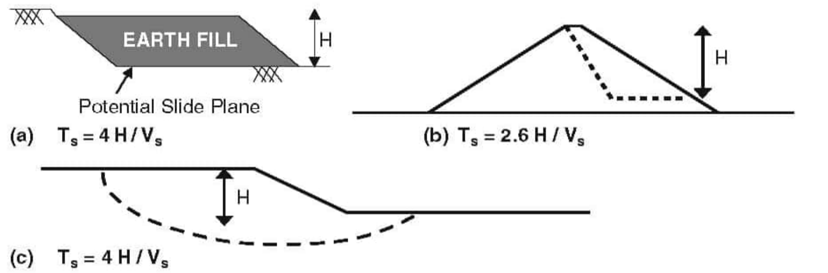

Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT - UPSTREAM SLOPE - CYCLIC SOFTENING (UNDRAINED)
SEE NZS1170.5 Ky=0.25g

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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.250		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	1.734	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

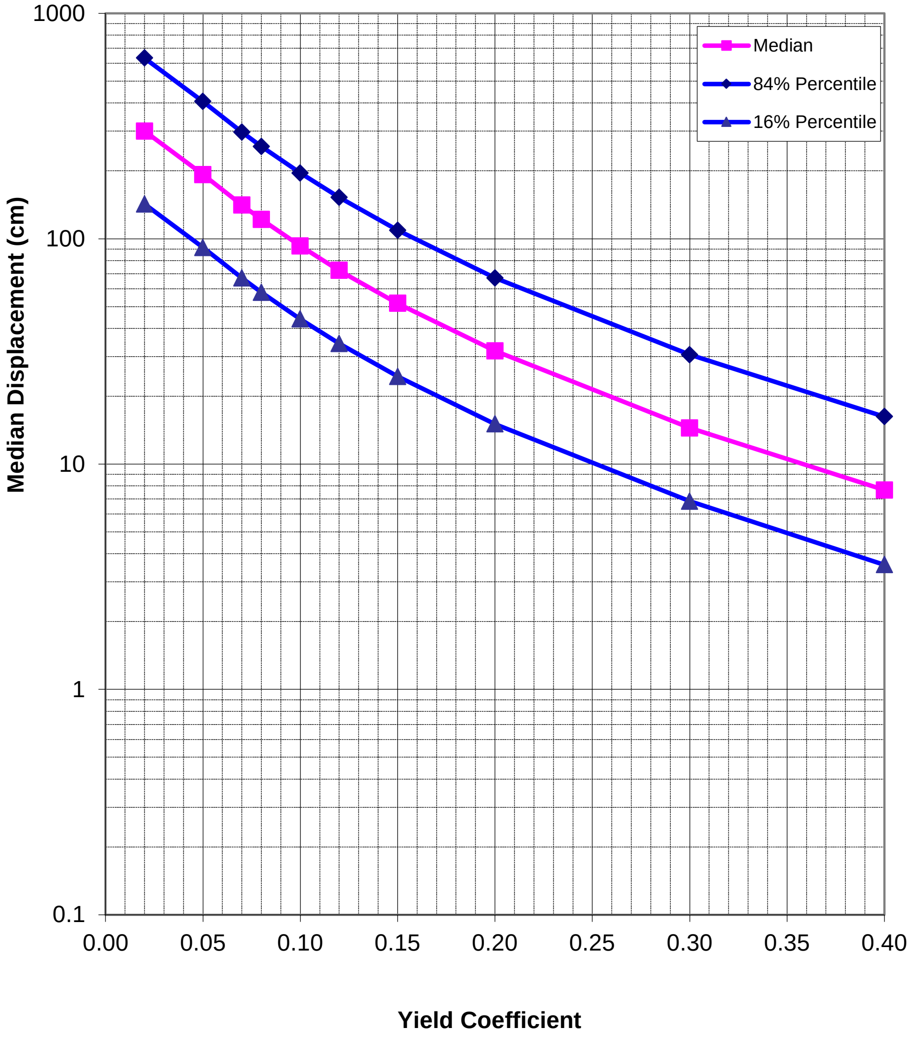
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	20.94	cm	eq. 3a or 3b	3.041431
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**9.9**	cm	calc. using eq. 1
D2	**20.9**	cm	calc. using eq. 1
D3	**44.1**	cm	calc. using eq. 1
P(D>d_threshold)	**0.32**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	300.7	300.7	633.9	142.6
0.05	0.000	192.8	192.8	406.4	91.4
0.07	0.000	141.0	141.0	297.4	66.9
0.08	0.000	121.9	121.9	256.9	57.8
0.1	0.000	92.8	92.8	195.6	44.0
0.12	0.000	72.3	72.3	152.5	34.3
0.15	0.000	51.7	51.7	108.9	24.5
0.2	0.000	31.8	31.8	67.0	15.1
0.3	0.002	14.5	14.5	30.5	6.8
0.4	0.010	7.7	7.7	16.2	3.6



Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

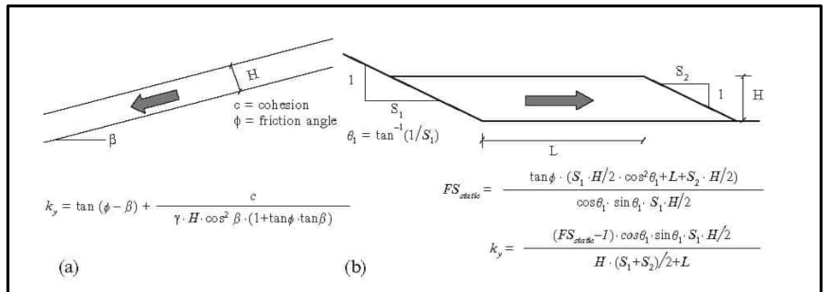


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

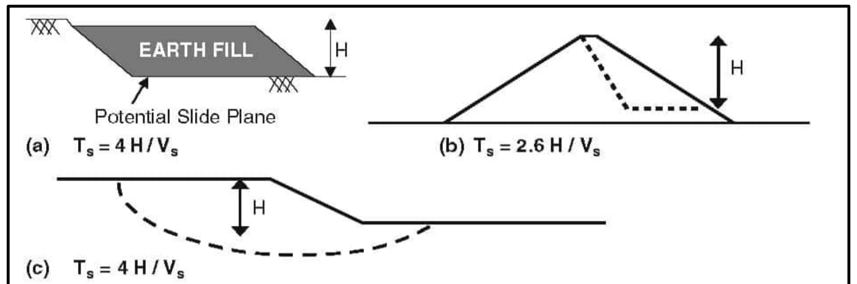


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT - DOWNSTREAM SLOPE - CYCLIC SOFTENING (UNDRAINED)
SEE NZS1170.5 Ky=0.10g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes
by Jorge Macedo, Jonathan D. Bray, and Chenying Liu
Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET
If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.100		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	1.734	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

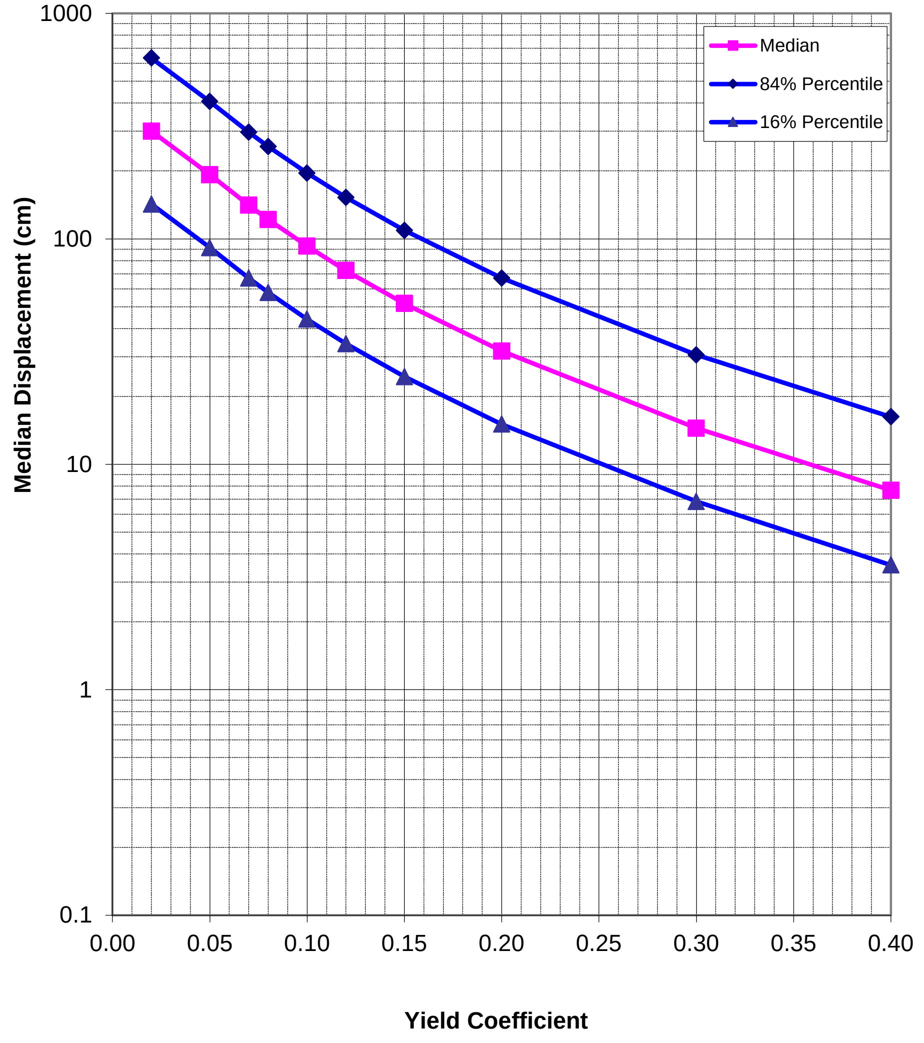
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	92.79	cm	eq. 3a or 3b 4.5303165
Standard Deviation of Non-Zero Seismic D	0.75		

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**44.0**	cm	calc. using eq. 1
D2	**92.8**	cm	calc. using eq. 1
D3	**195.6**	cm	calc. using eq. 1
P(D>d_threshold)	**0.93**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	300.7	300.7	633.9	142.6
0.05	0.000	192.8	192.8	406.4	91.4
0.07	0.000	141.0	141.0	297.4	66.9
0.08	0.000	121.9	121.9	256.9	57.8
0.1	0.000	92.8	92.8	195.6	44.0
0.12	0.000	72.3	72.3	152.5	34.3
0.15	0.000	51.7	51.7	108.9	24.5
0.2	0.000	31.8	31.8	67.0	15.1
0.3	0.002	14.5	14.5	30.5	6.8
0.4	0.010	7.7	7.7	16.2	3.6



Notes

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2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

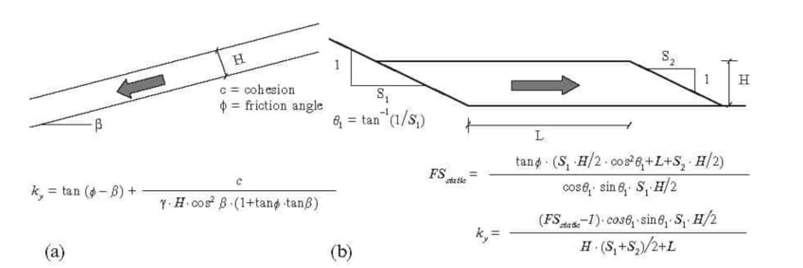


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

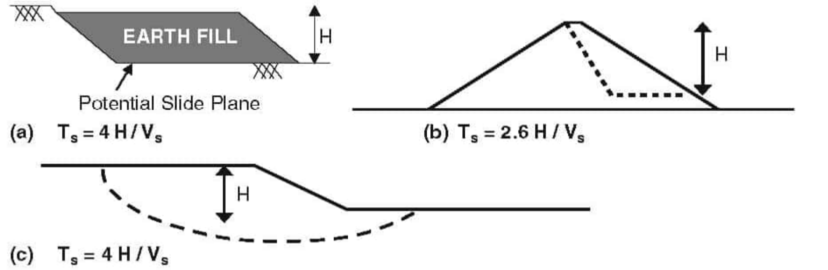


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT - UPSTREAM SLOPE
OBE NHSM Ky=0.4g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes

by Jorge Macedo, Jonathan D. Bray, and Chenying Liu

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.400		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	0.878	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

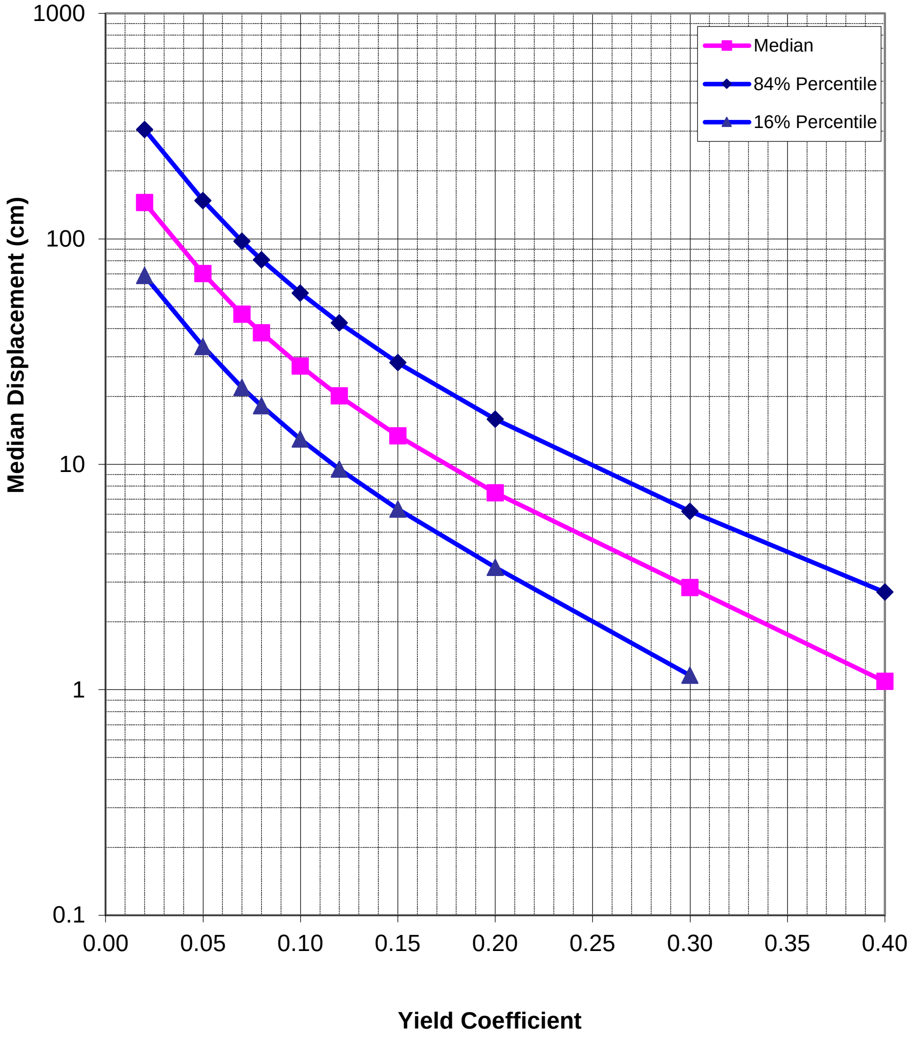
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	1.49	cm	eq. 3a or 3b	0.3962116
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.24**		eq. 2
D1	**<0.5**	cm	calc. using eq. 1
D2	**1.1**	cm	calc. using eq. 1
D3	**2.7**	cm	calc. using eq. 1
P(D>d_threshold)	**0.00**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	144.8	144.8	305.3	68.7
0.05	0.000	70.1	70.1	147.7	33.2
0.07	0.000	46.2	46.2	97.5	21.9
0.08	0.000	38.3	38.3	80.8	18.2
0.1	0.000	27.3	27.3	57.5	12.9
0.12	0.001	20.1	20.1	42.3	9.5
0.15	0.002	13.4	13.4	28.2	6.3
0.2	0.009	7.5	7.5	15.8	3.5
0.3	0.068	3.0	2.8	6.2	1.2
0.4	0.244	1.5	1.1	2.7	<1



Notes

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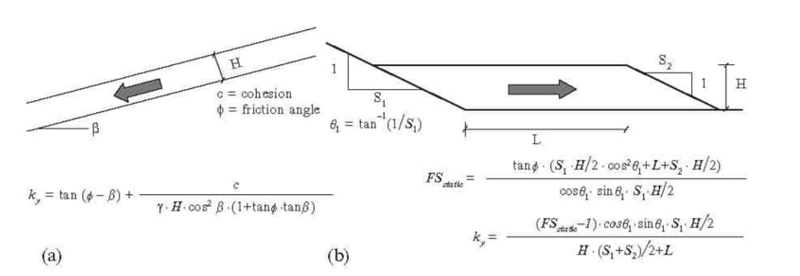
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)



$$k_y = \tan(\phi - \beta) + \frac{c}{\gamma \cdot H \cdot \cos^2\beta \cdot (1 + \tan\phi \cdot \tan\beta)}$$

$$\theta_1 = \tan^{-1}(1/S_1)$$

$$FS_{static} = \frac{\tan\phi \cdot (S_1 \cdot H/2 \cdot \cos^2\theta_1 + L + S_2 \cdot H/2)}{\cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}$$

$$k_y = \frac{(FS_{static} - 1) \cdot \cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}{H \cdot (S_1 + S_2)/2 + L}$$

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

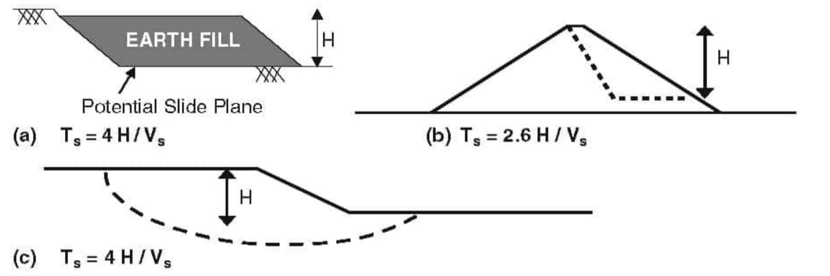


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT - DOWNSTREAM SLOPE
OBE NHSM Ky=0.21g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes
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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.210		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.5		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	0.878	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

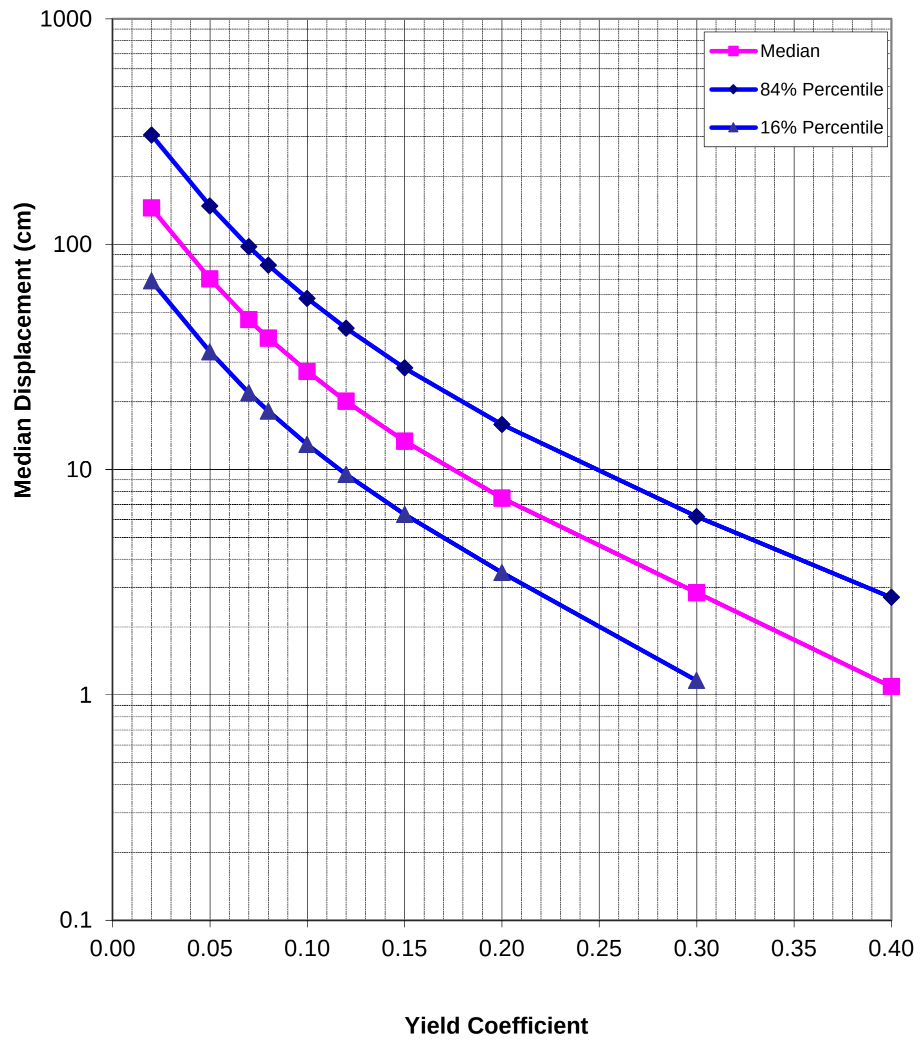
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	6.81	cm	eq. 3a or 3b	1.917685
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.01**		eq. 2
D1	**3.1**	cm	calc. using eq. 1
D2	**6.7**	cm	calc. using eq. 1
D3	**14.3**	cm	calc. using eq. 1
P(D>d_threshold)	**0.02**		eq. 1

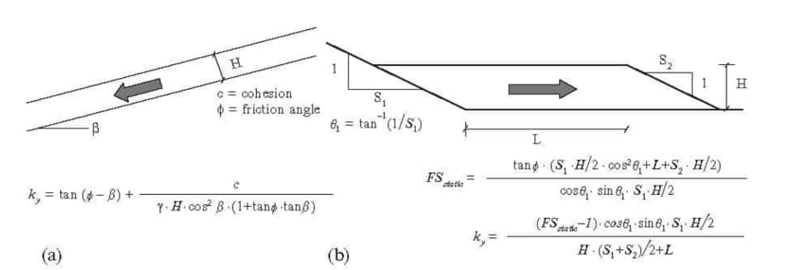
Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	144.8	144.8	305.3	68.7
0.05	0.000	70.1	70.1	147.7	33.2
0.07	0.000	46.2	46.2	97.5	21.9
0.08	0.000	38.3	38.3	80.8	18.2
0.1	0.000	27.3	27.3	57.5	12.9
0.12	0.001	20.1	20.1	42.3	9.5
0.15	0.002	13.4	13.4	28.2	6.3
0.2	0.009	7.5	7.5	15.8	3.5
0.3	0.068	3.0	2.8	6.2	1.2
0.4	0.244	1.5	1.1	2.7	<1



Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)



$$k_y = \tan(\phi - \beta) + \frac{c}{\gamma \cdot H \cdot \cos^2\beta \cdot (1+\tan\phi \cdot \tan\beta)}$$

$$FS_{static} = \frac{\tan\phi \cdot (S_1 \cdot H/2 \cdot \cos^2\theta_1 + L + S_2 \cdot H/2)}{\cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}$$

$$k_y = \frac{(FS_{static} - 1) \cdot \cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}{H \cdot (S_1 + S_2)/2 + L}$$

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

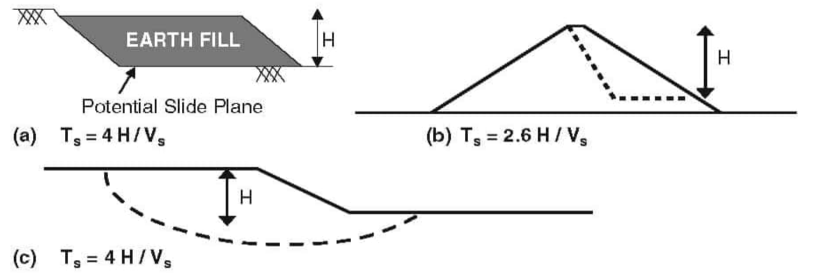


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT - UPSTREAM SLOPE - CYCLIC SOFTENING (UNDRAINED)
SEE NHSM Ky=0.25g

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by Jorge Macedo, Jonathan D. Bray, and Chenying Liu
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If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.250		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.9		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	3.134	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

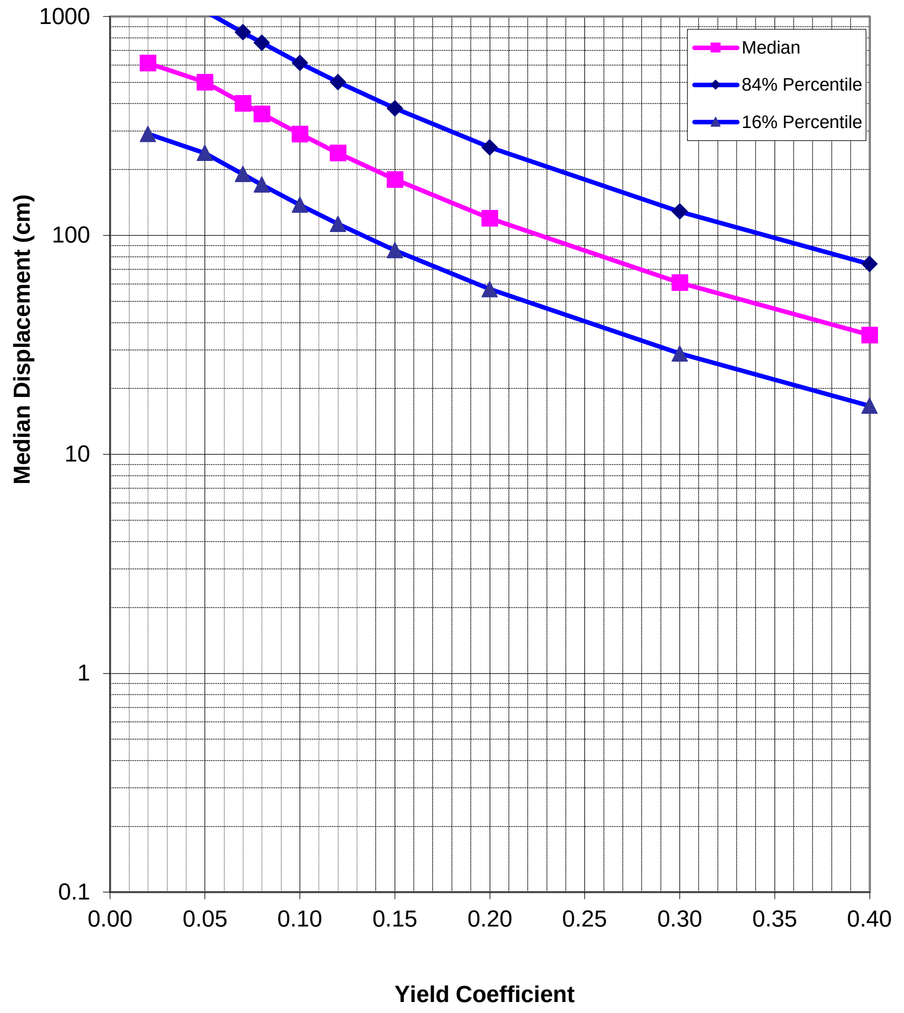
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	83.79	cm	eq. 3a or 3b	4.4283528
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**39.7**	cm	calc. using eq. 1
D2	**83.8**	cm	calc. using eq. 1
D3	**176.7**	cm	calc. using eq. 1
P(D>d_threshold)	**0.91**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	612.9	612.9	1292.2	290.7
0.05	0.000	502.0	502.0	1058.3	238.1
0.07	0.000	401.8	401.8	847.1	190.6
0.08	0.000	359.7	359.7	758.4	170.6
0.1	0.000	290.7	290.7	613.0	137.9
0.12	0.000	238.0	238.0	501.8	112.9
0.15	0.000	180.4	180.4	380.4	85.6
0.2	0.000	119.8	119.8	252.6	56.8
0.3	0.000	60.9	60.9	128.5	28.9
0.4	0.000	35.1	35.1	74.1	16.7



Notes
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2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
(e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

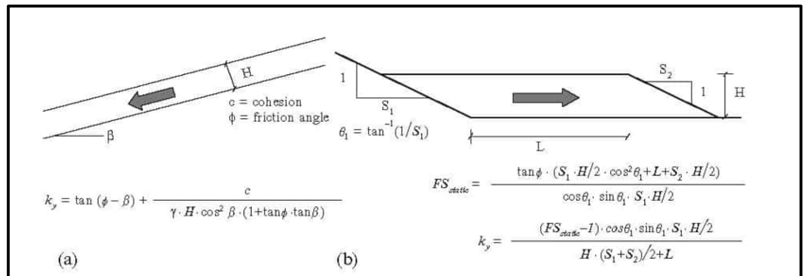


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

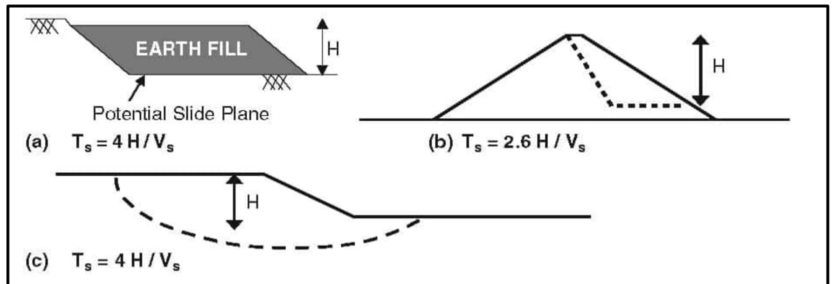


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT - DOWNSTREAM SLOPE - CYCLIC SOFTENING (UNDRAINED)
SEE NHSM Ky=0.1g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes
by Jorge Macedo, Jonathan D. Bray, and Chenying Liu
Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET
If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.100		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.9		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	3.134	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

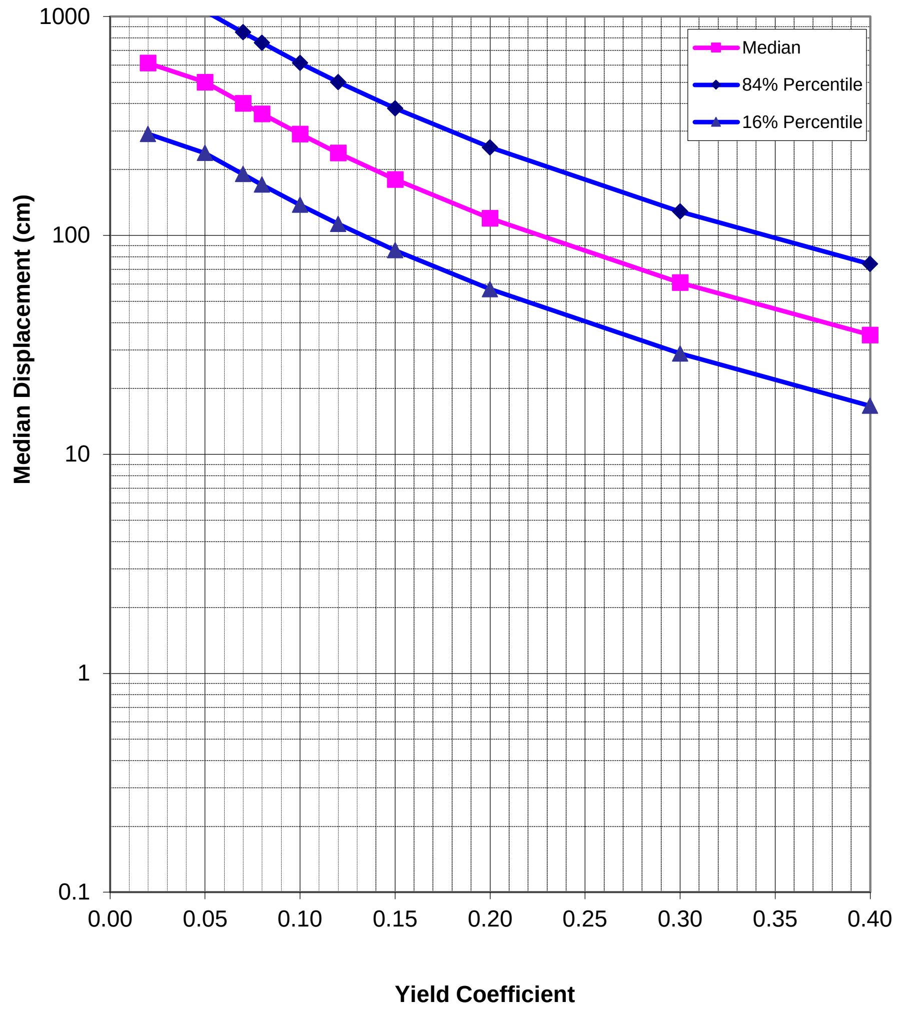
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	290.74	cm	eq. 3a or 3b	5.6724442
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**137.9**	cm	calc. using eq. 1
D2	**290.7**	cm	calc. using eq. 1
D3	**613.0**	cm	calc. using eq. 1
P(D>d_threshold)	**1.00**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	612.9	612.9	1292.2	290.7
0.05	0.000	502.0	502.0	1058.3	238.1
0.07	0.000	401.8	401.8	847.1	190.6
0.08	0.000	359.7	359.7	758.4	170.6
0.1	0.000	290.7	290.7	613.0	137.9
0.12	0.000	238.0	238.0	501.8	112.9
0.15	0.000	180.4	180.4	380.4	85.6
0.2	0.000	119.8	119.8	252.6	56.8
0.3	0.000	60.9	60.9	128.5	28.9
0.4	0.000	35.1	35.1	74.1	16.7



Notes
1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
(e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

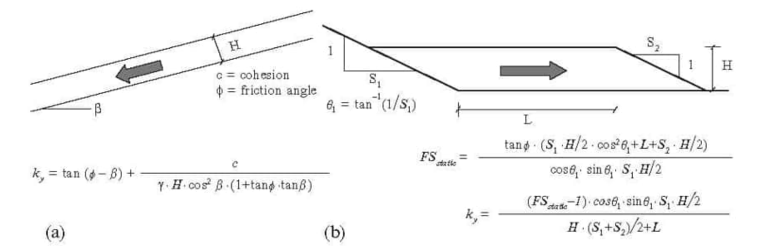


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

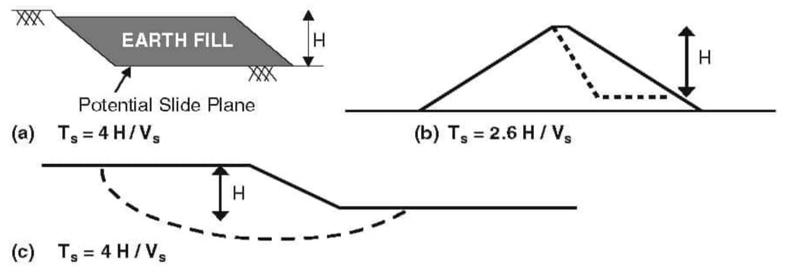


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

EASTERN DAM EMBANKMENT WITH BERM - DOWNSTREAM SLOPE - CYCLIC SOFTENING
SEE NHSM Ky=0.2g

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes

by Jorge Macedo, Jonathan D. Bray, and Chenying Liu

Seismic Slope Displacement Procedure for Interface and Intraslab Subduction Zone Earthquakes, ASCE JGGE, 2023, in review

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

If using the scalar model (eq. 3a), enter all required inputs and -1 for PGV; If using the vector model (eq. 3b), enter all required inputs and the PGV estimate

Input Parameters			
Yield Coefficient (ky)	0.200		**Based on pseudostatic analysis**
Initial Fundamental Period (Ts)	0.32	seconds	**1D: Ts=4H/Vs 2D: Ts=2.6H/Vs**
Degraded Period (1.3Ts)	0.42	seconds	
Moment Magnitude (Mw)	7.9		
PGV	-1.0	cm/s	**Use -1 if using the scalar model**
Spectral Acceleration (Sa(1.3Ts))	3.134	g	**Input Spectral Acceleration at base of sliding mass assuming there is no material above it.**
Mechanism	0		**0 for interface 1 for intraslab**

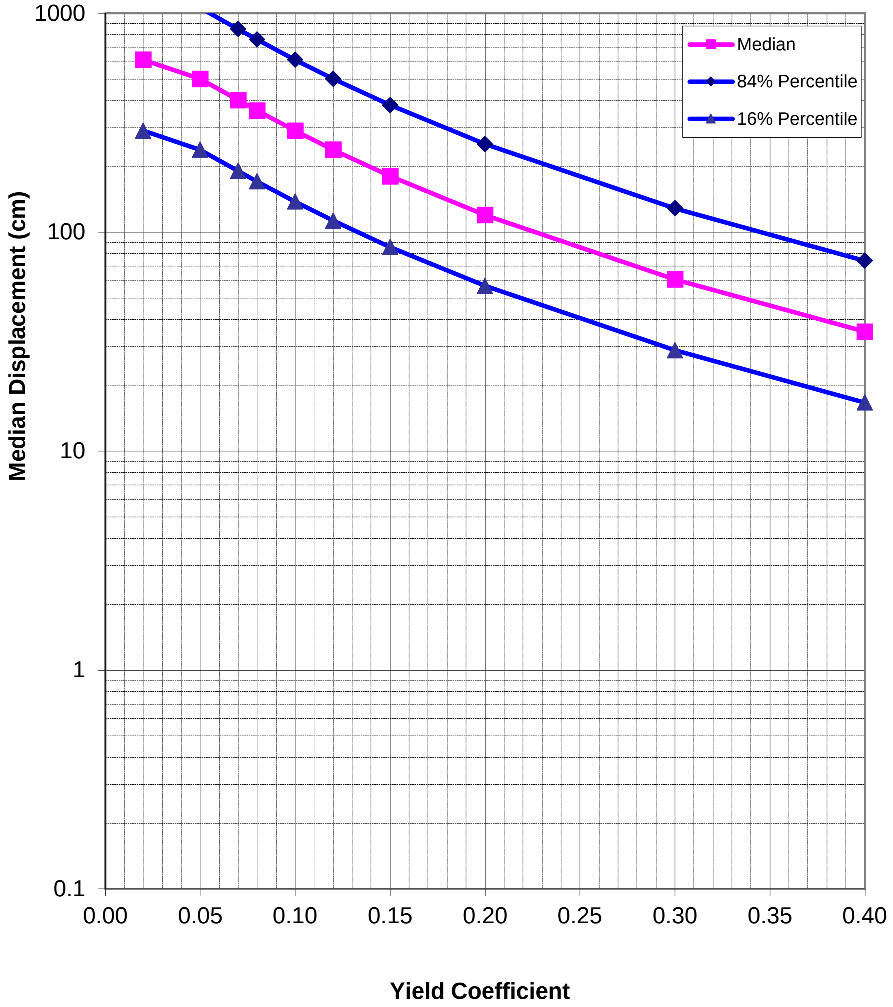
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	119.83	cm	eq. 3a or 3b	4.7860942
Standard Deviation of Non-Zero Seismic D	0.75			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. 2
D1	**56.8**	cm	calc. using eq. 1
D2	**119.8**	cm	calc. using eq. 1
D3	**252.6**	cm	calc. using eq. 1
P(D>d_threshold)	**0.97**		eq. 1

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.000	612.9	612.9	1292.2	290.7
0.05	0.000	502.0	502.0	1058.3	238.1
0.07	0.000	401.8	401.8	847.1	190.6
0.08	0.000	359.7	359.7	758.4	170.6
0.1	0.000	290.7	290.7	613.0	137.9
0.12	0.000	238.0	238.0	501.8	112.9
0.15	0.000	180.4	180.4	380.4	85.6
0.2	0.000	119.8	119.8	252.6	56.8
0.3	0.000	60.9	60.9	128.5	28.9
0.4	0.000	35.1	35.1	74.1	16.7



Notes

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2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below in Fig. 14.1 provide approximate values.
10. Examples of how Ts is estimated are shown below in Fig. 14.4 and Fig. 3.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

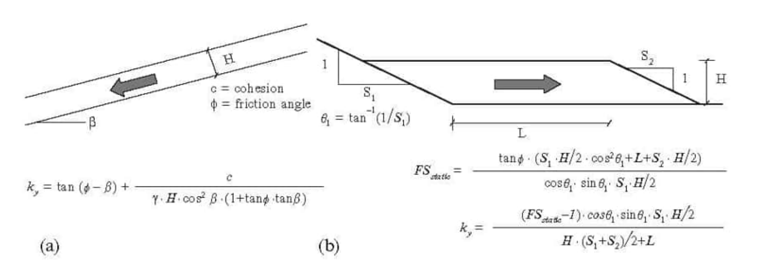


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

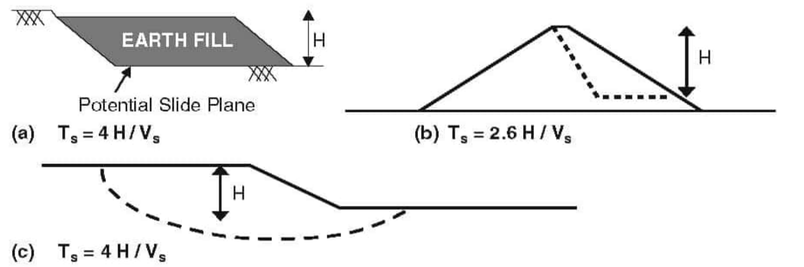


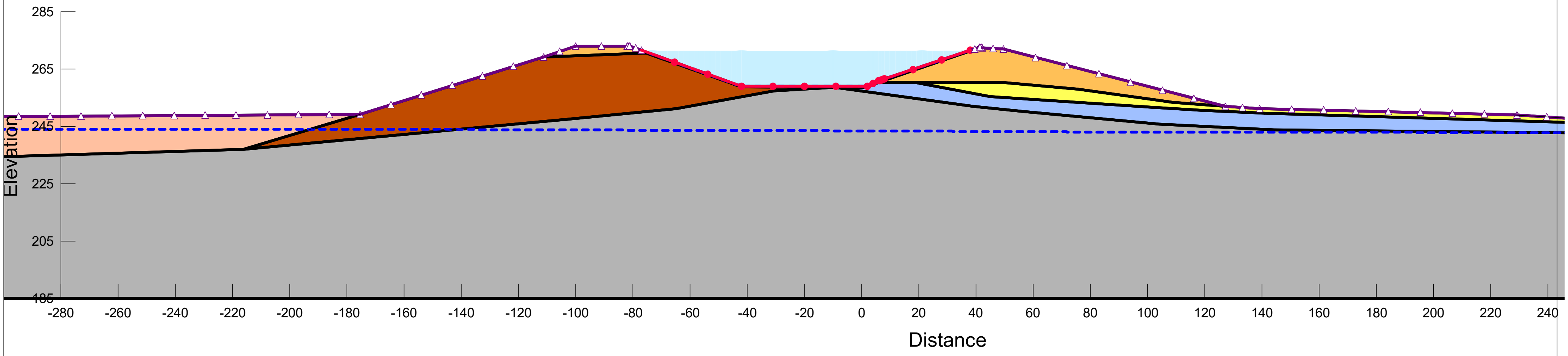
Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, V. 6, Pitilakis, Kyriazis D., Ed., Springer, 327-353.

Western Reservoir Rim Seepage Analyses
Analysed By: CHEV
Checked By: ABL

Color	Name	Category	Kind	Parameters
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Full Supply Level	Hydraulic	Water Total Head	271.5 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Clay Liner (Compacted Loess)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	HDPE Liner		HDPE	HDPE	1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1



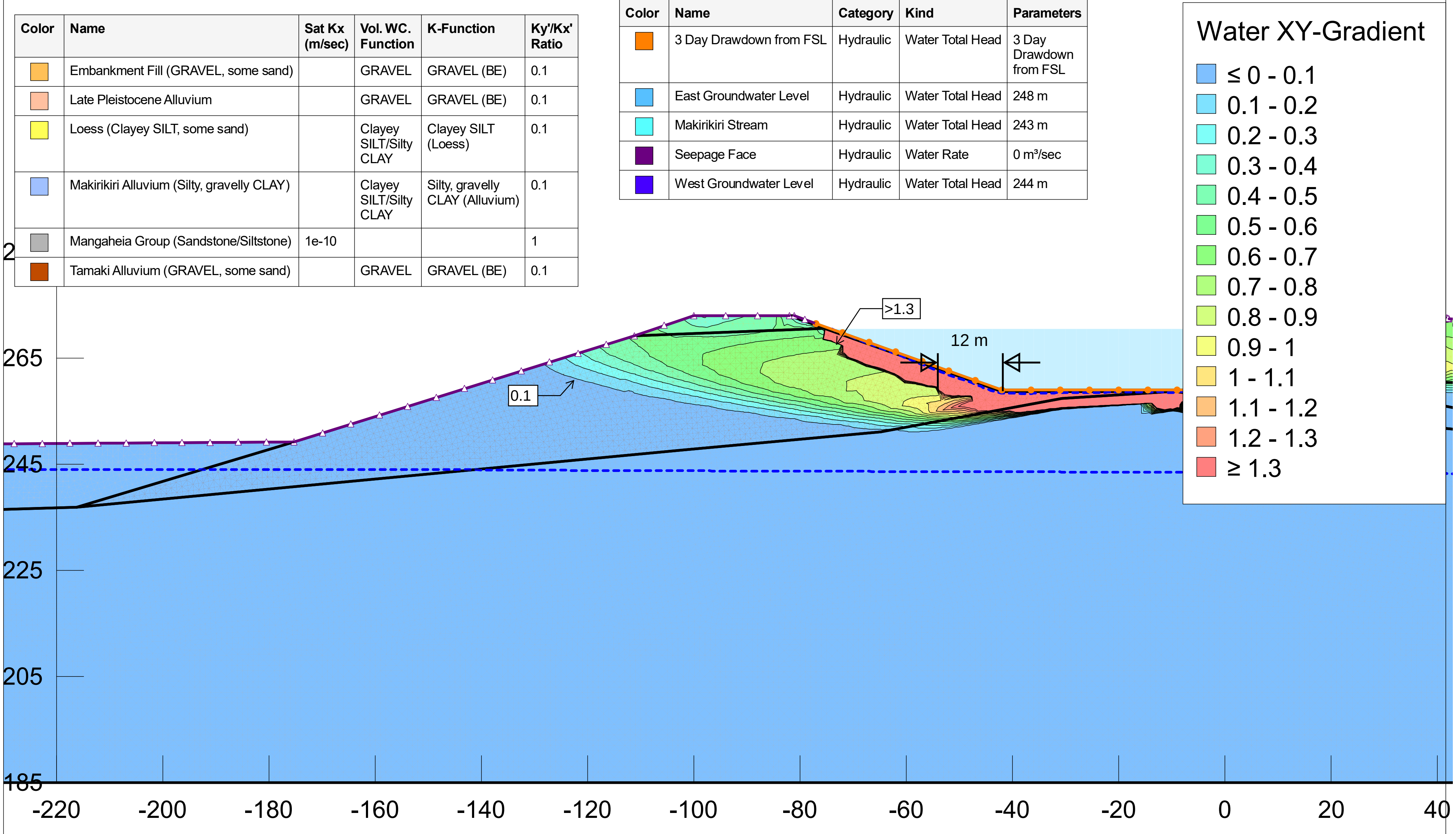
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Steady-State Seepage (FSL) - HDPE and clay liner		Analysed by: CHEV
Comments:	Scale: 1:2,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

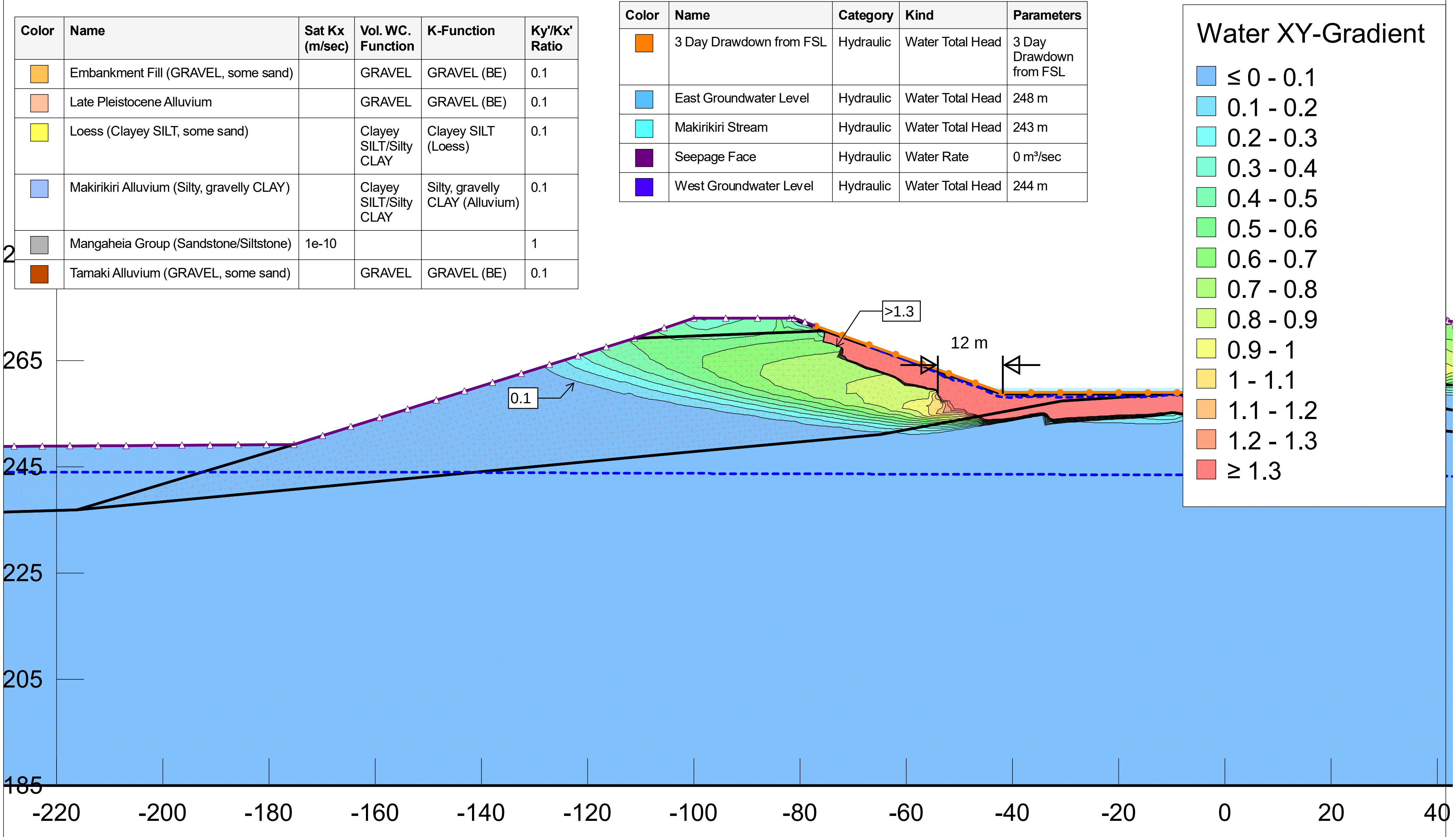


Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
	Analysis: Transient Seepage (Best Estimate Perm) - 6 hours		Analysed by: CHEV
	Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 2.75 days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

Water Total Head

- 242 - 244 m
- 244 - 246 m
- 246 - 248 m
- 248 - 250 m
- 250 - 252 m
- 252 - 254 m
- 254 - 256 m
- 256 - 258 m
- 258 - 260 m
- 260 - 262 m
- 262 - 264 m
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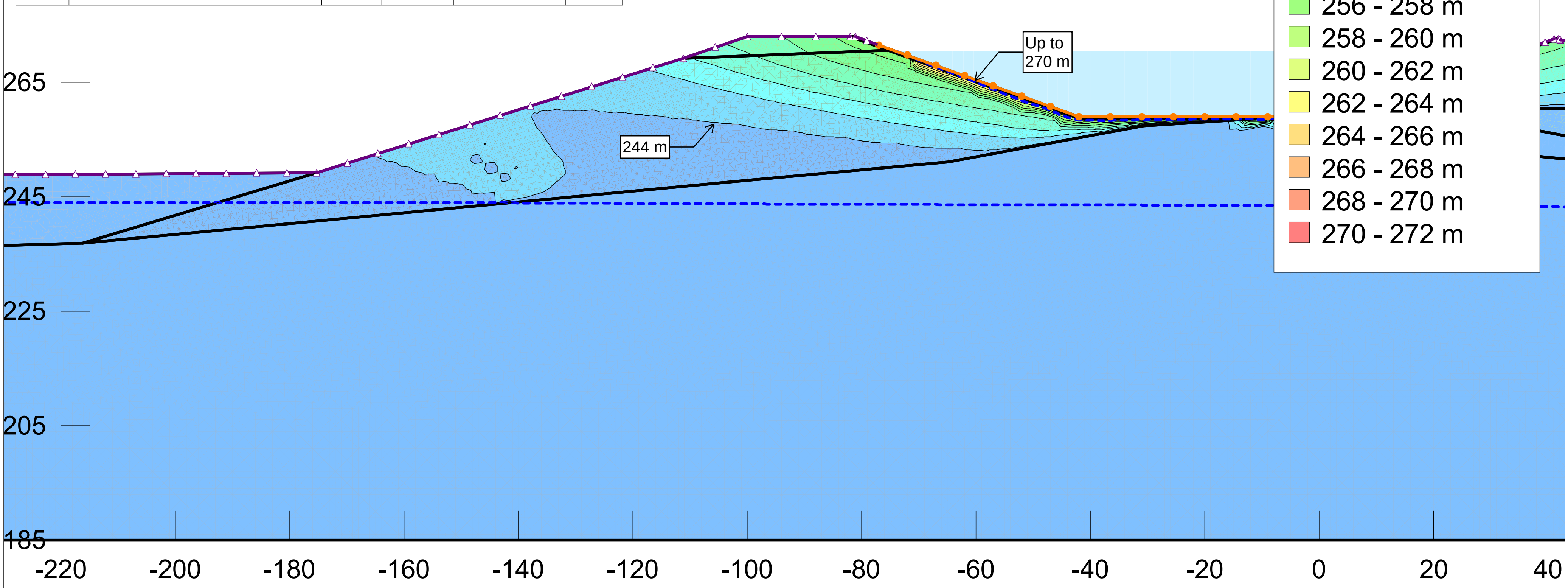
- 266 - 268 m
- 268 - 270 m
- 270 - 272 m



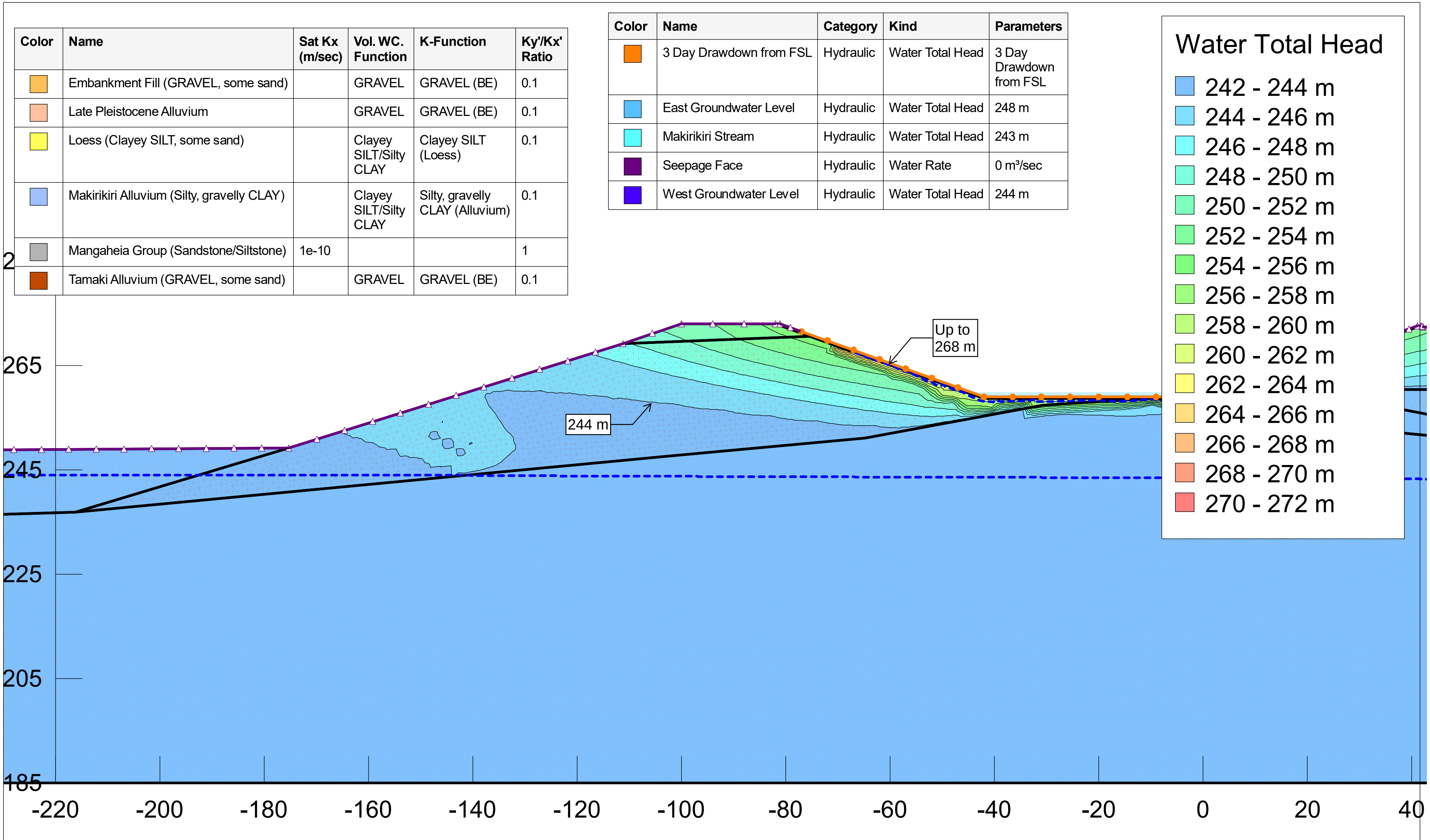
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz	Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 6 hours	Analysed by: CHEV
Comments: Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



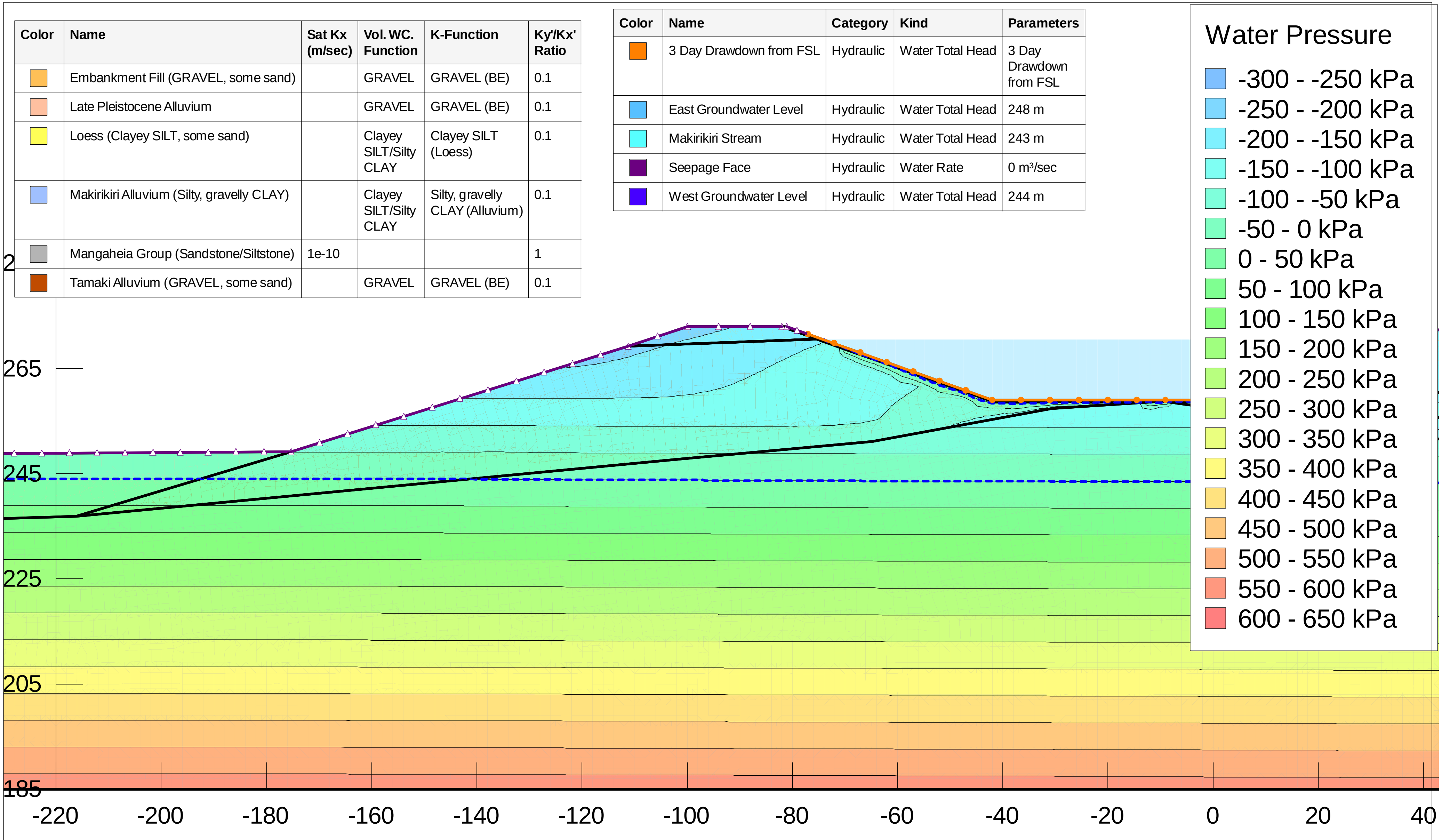
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 2.75 days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

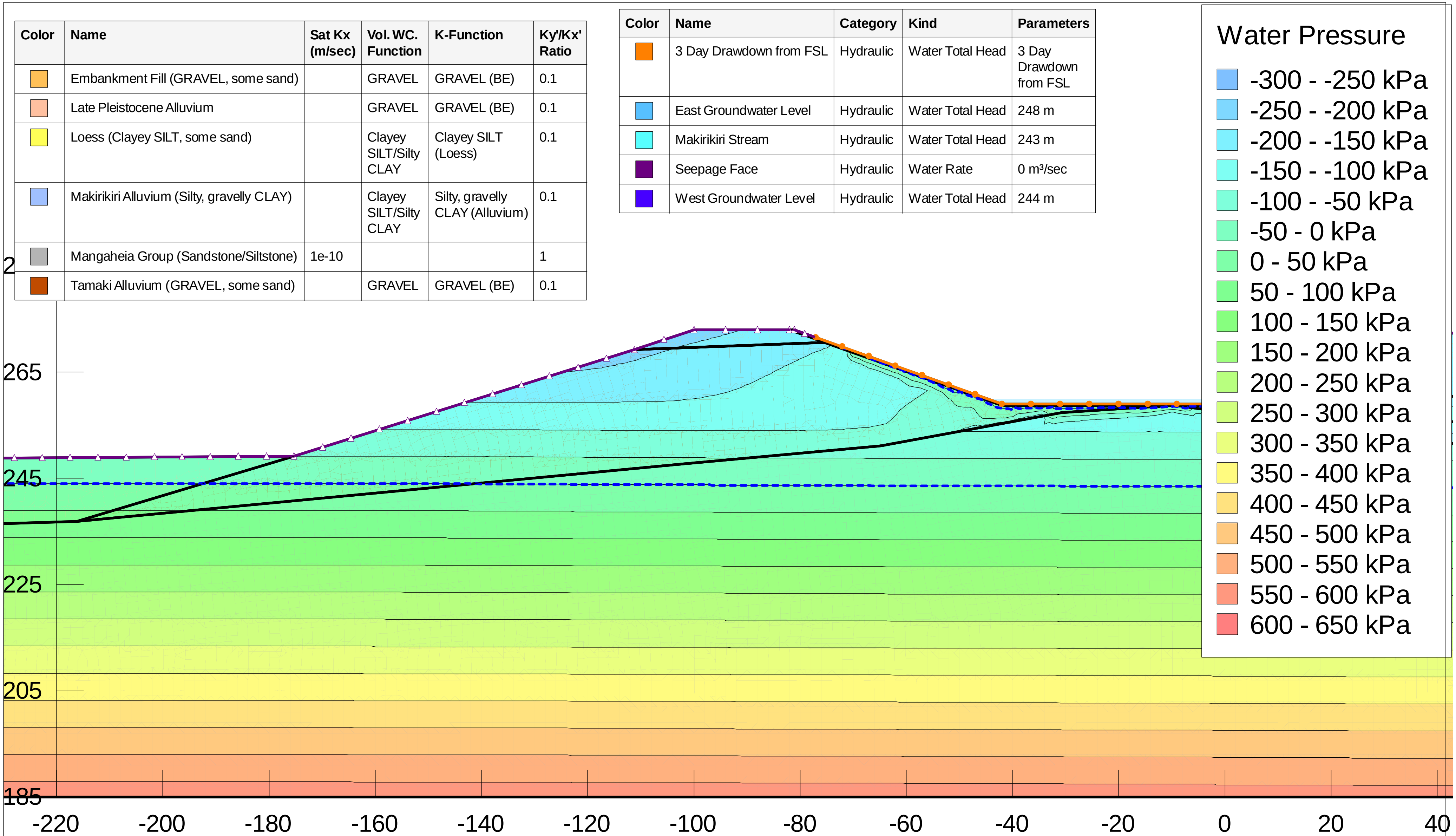
Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 6hrs		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\



Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

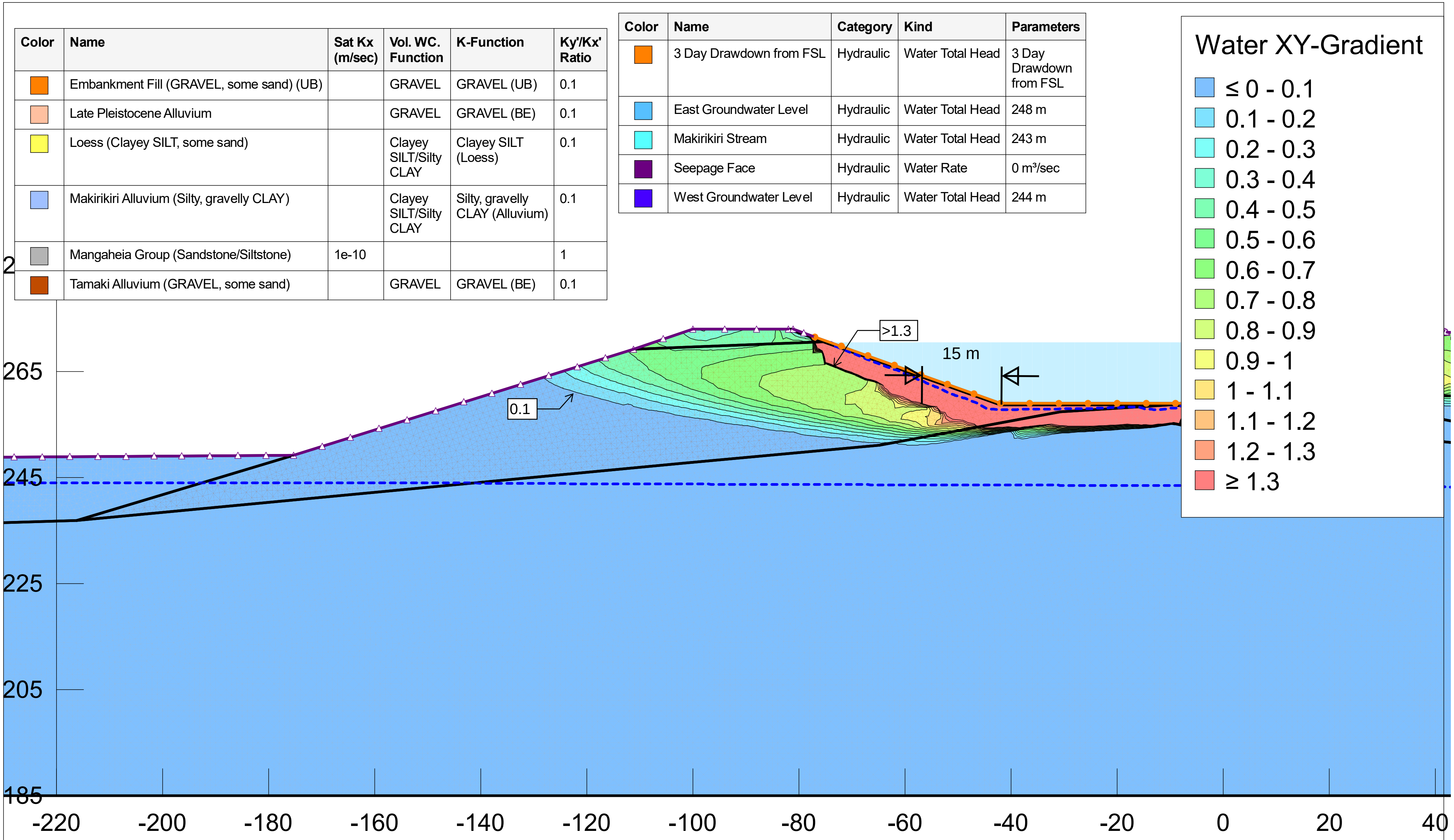
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 2.75days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



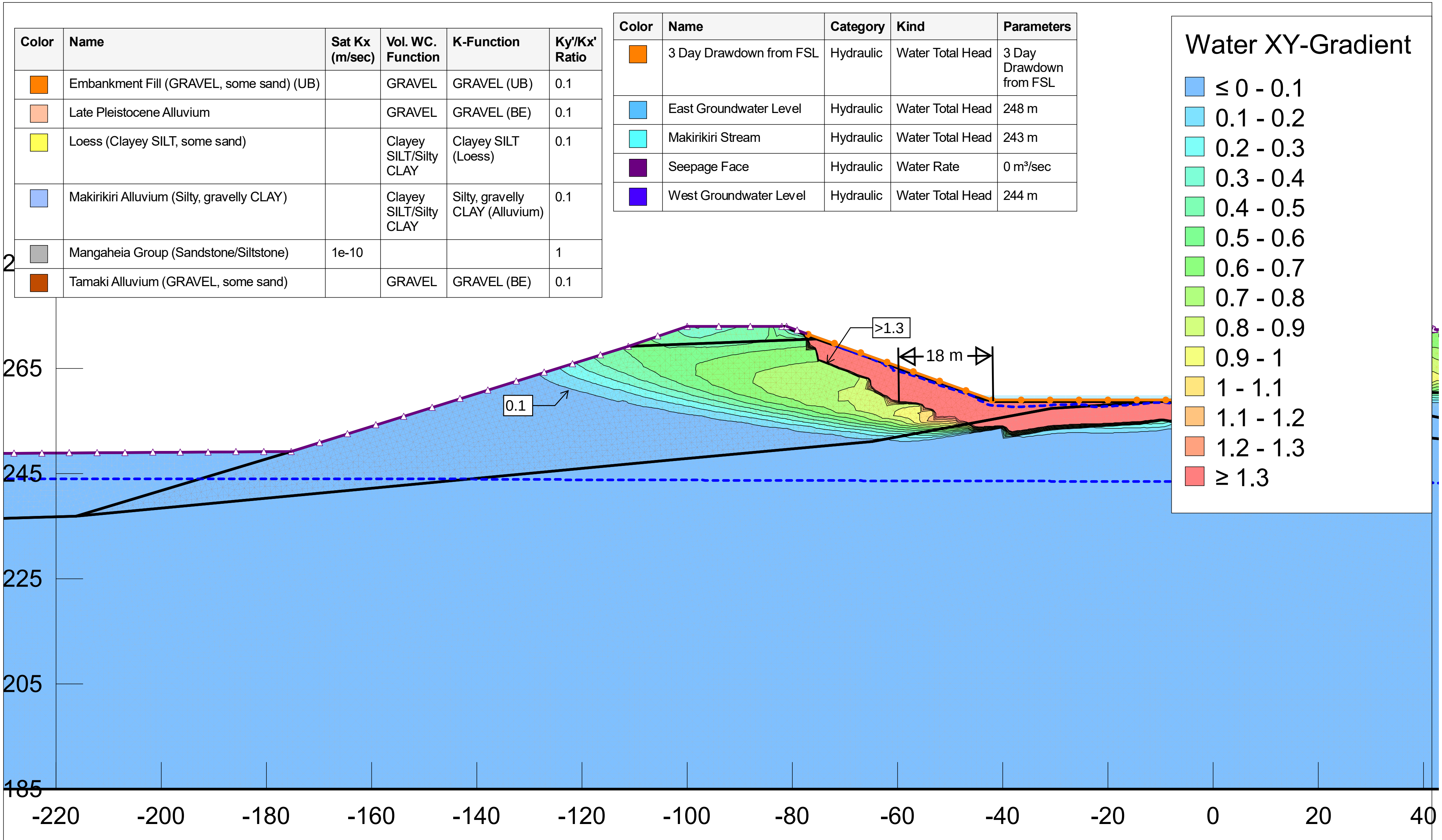
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Higher Perm) - 6 hours		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

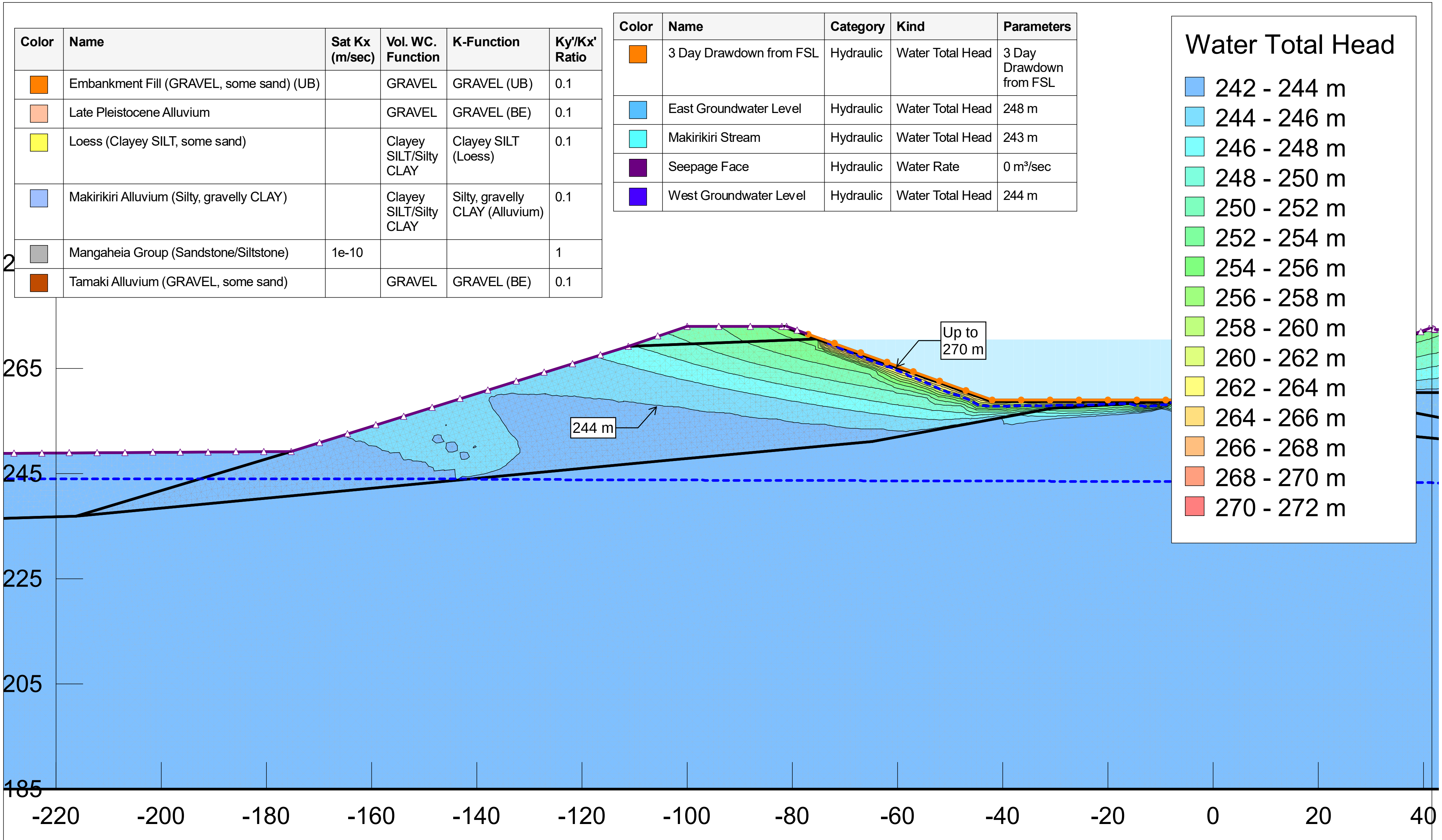


Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz	Job Number: 1020688.4000
	Analysis: Transient Seepage (Higher Perm) - 2.75 days	Analysed by: CHEV
	Comments: Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

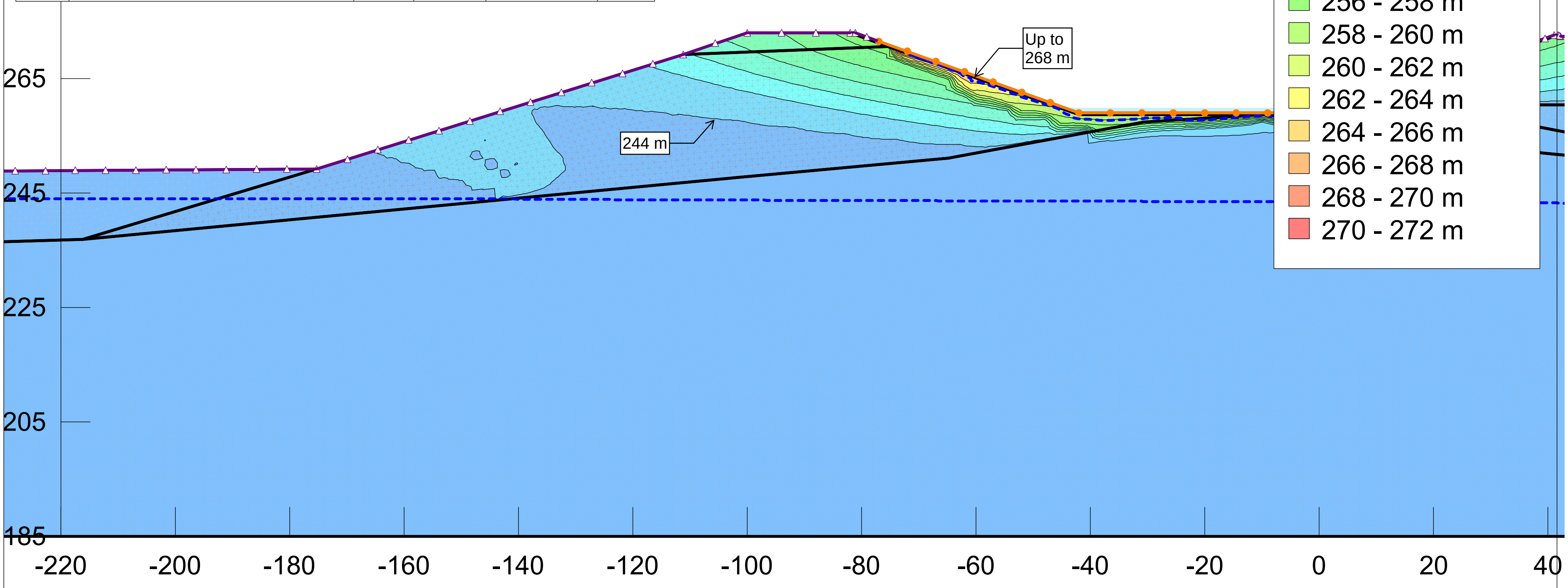


Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz	Job Number: 1020688.4000
	Analysis: Transient Seepage (Higher Perm) - 6 hours	Analysed by: CHEV
	Comments: / Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



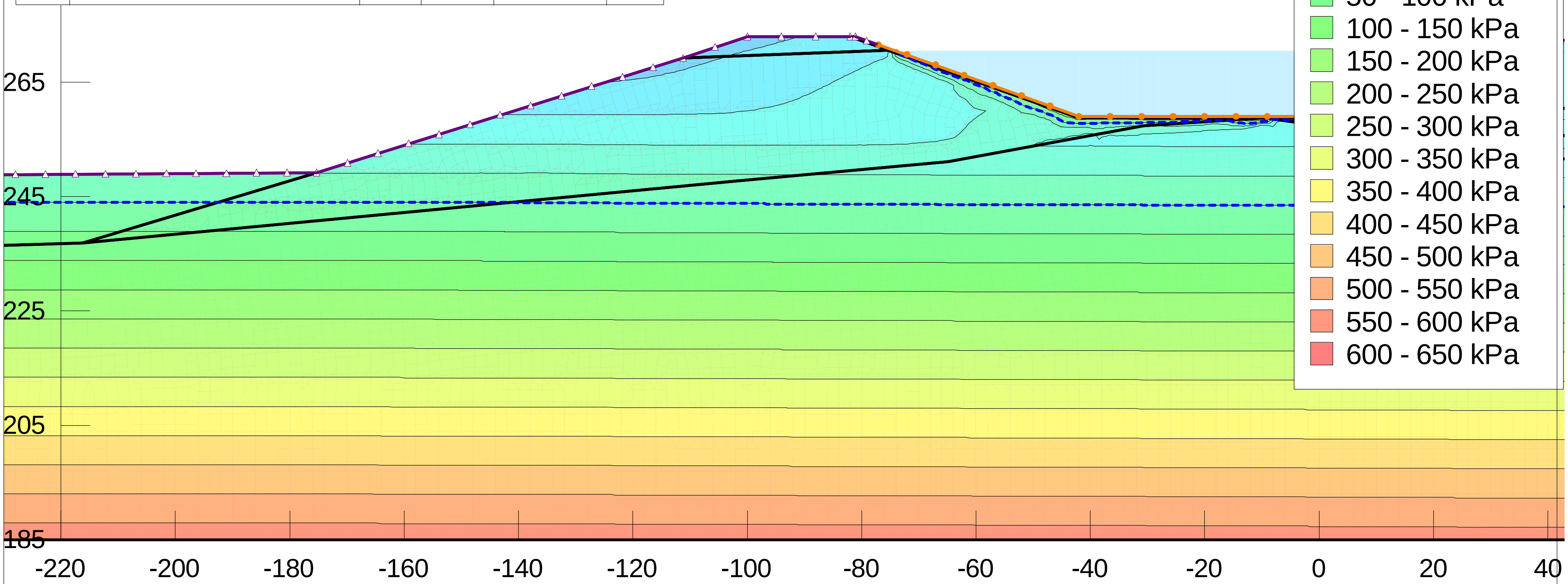
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Higher Perm) - 2.75 days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Water Pressure

- -300 - -250 kPa
- -250 - -200 kPa
- -200 - -150 kPa
- -150 - -100 kPa
- -100 - -50 kPa
- -50 - 0 kPa
- 0 - 50 kPa
- 50 - 100 kPa
- 100 - 150 kPa
- 150 - 200 kPa
- 200 - 250 kPa
- 250 - 300 kPa
- 300 - 350 kPa
- 350 - 400 kPa
- 400 - 450 kPa
- 450 - 500 kPa
- 500 - 550 kPa
- 550 - 600 kPa
- 600 - 650 kPa

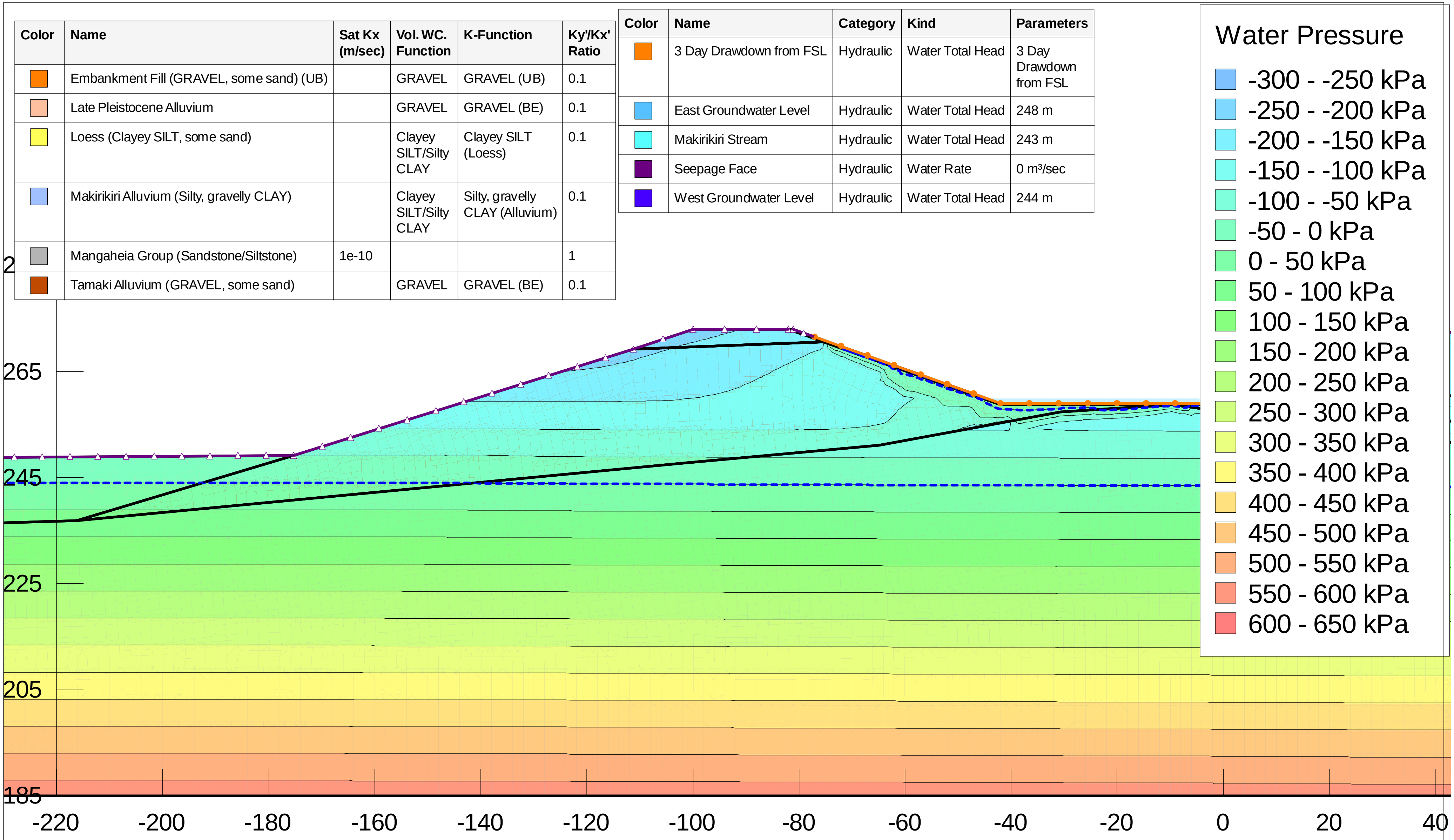
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Higher Perm) - 6 hours		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Tonkin+Taylor

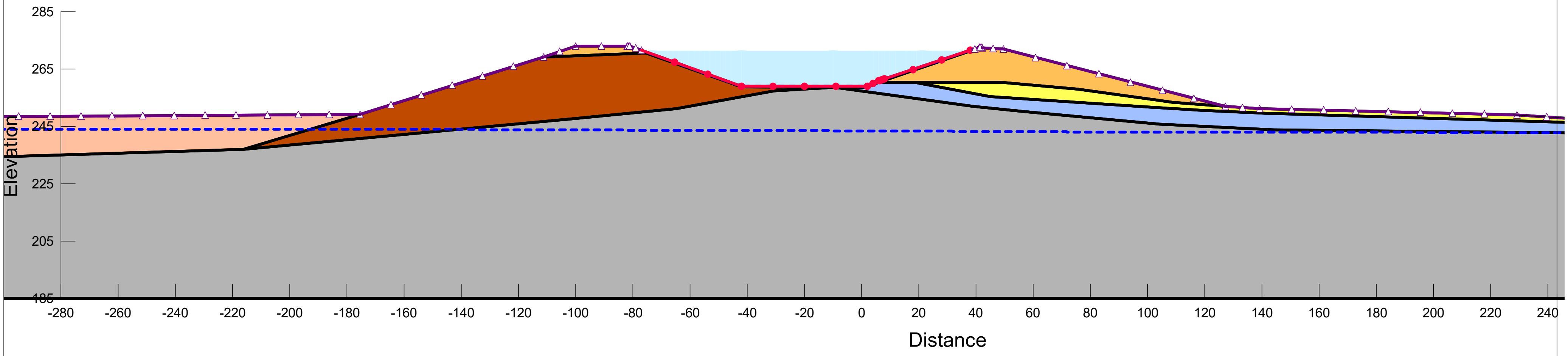
Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Higher Perm) - 2.75 days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Eastern Embankment Seepage Analyses
Analysed By: CHEV
Checked By: ABL

Color	Name	Category	Kind	Parameters
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Full Supply Level	Hydraulic	Water Total Head	271.5 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Clay Liner (Compacted Loess)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	HDPE Liner		HDPE	HDPE	1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

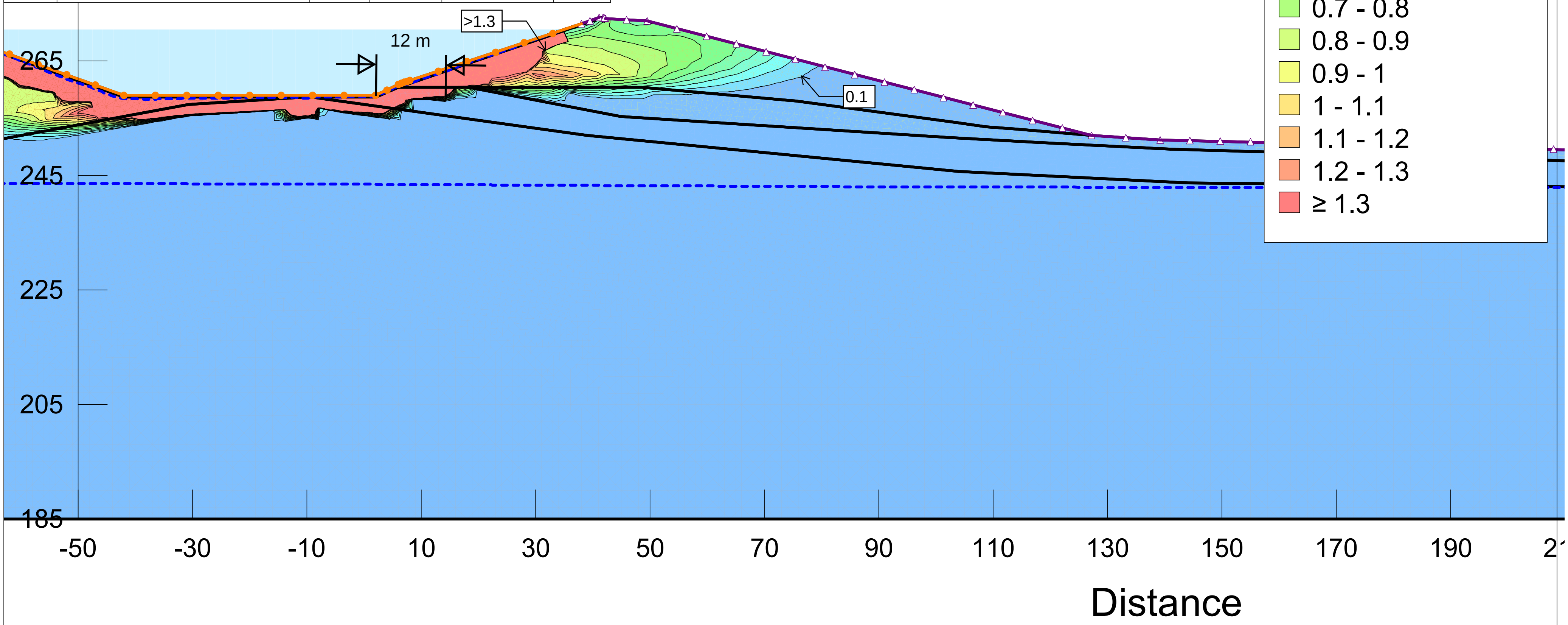


Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz	Job Number: 1020688.4000
	Analysis: Steady-State Seepage (FSL) - HDPE and clay liner	Analysed by: CHEV
	Comments: Scale: 1:2,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

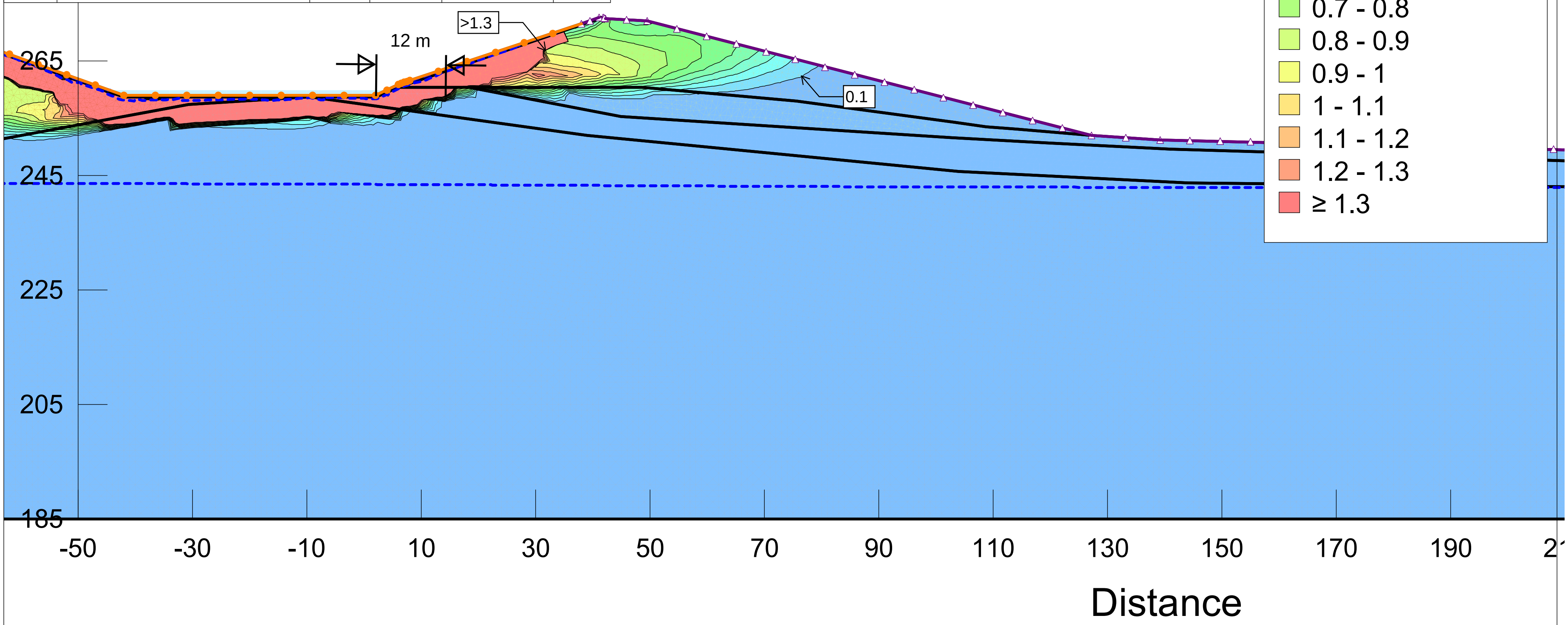


Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz	Job Number: 1020688.4000
	Analysis: Transient Seepage (Best Estimate Perm) - 6 hours	Analysed by: CHEV
	Comments: Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

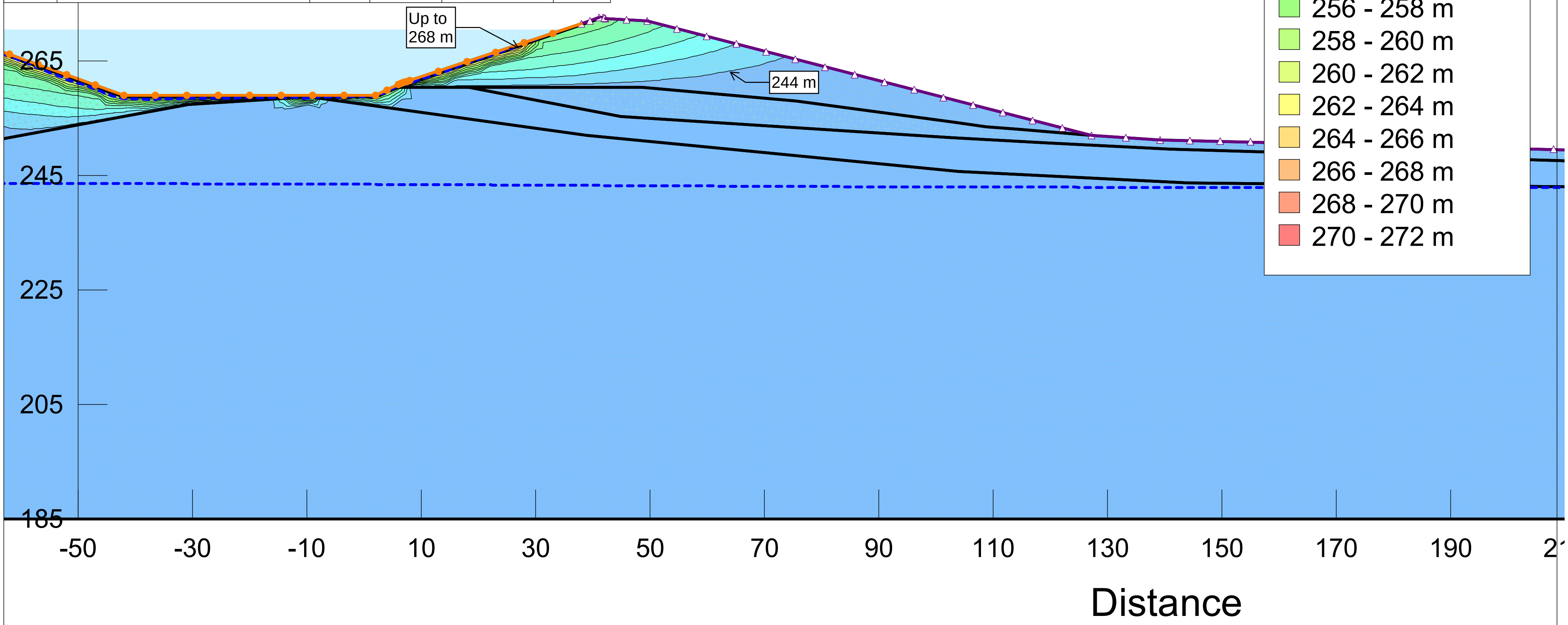


Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
	Analysis: Transient Seepage (Best Estimate Perm) - 2.75 days		Analysed by: CHEV
	Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

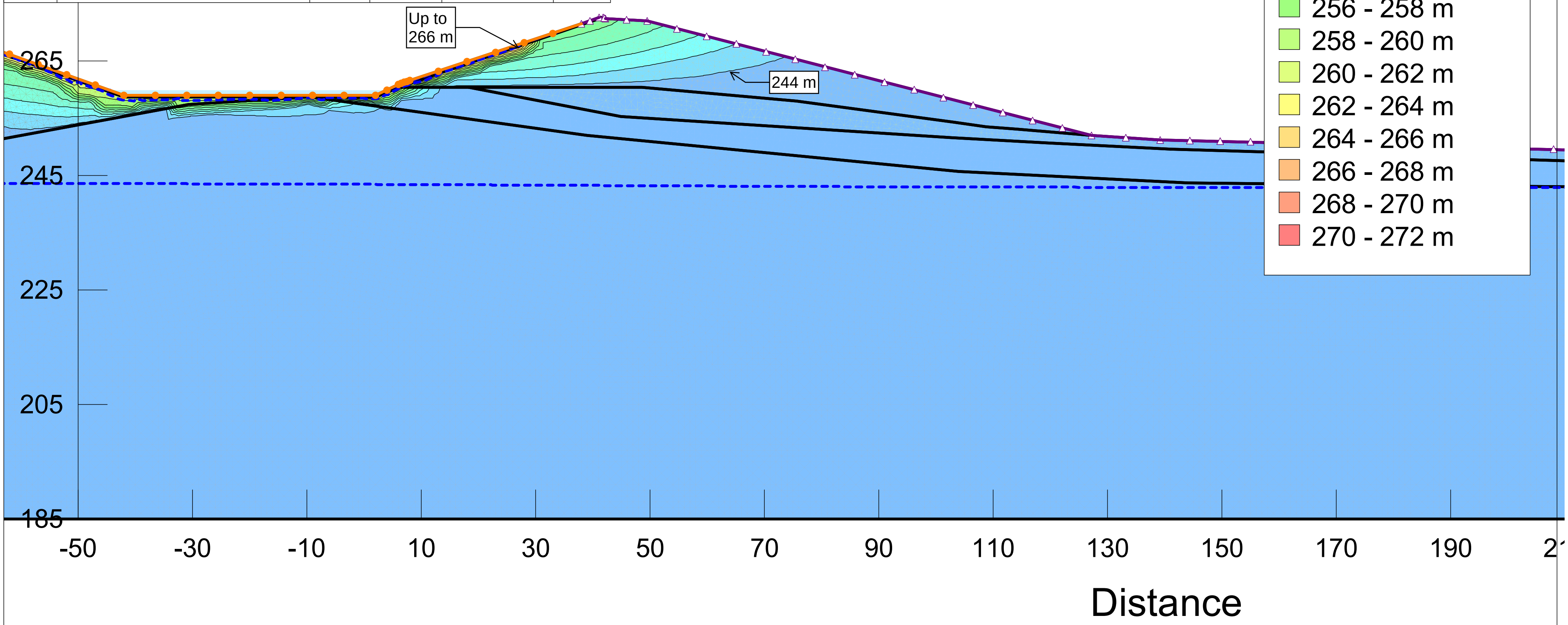


Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
	Analysis: Transient Seepage (Best Estimate Perm) - 6 hours		Analysed by: CHEV
	Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



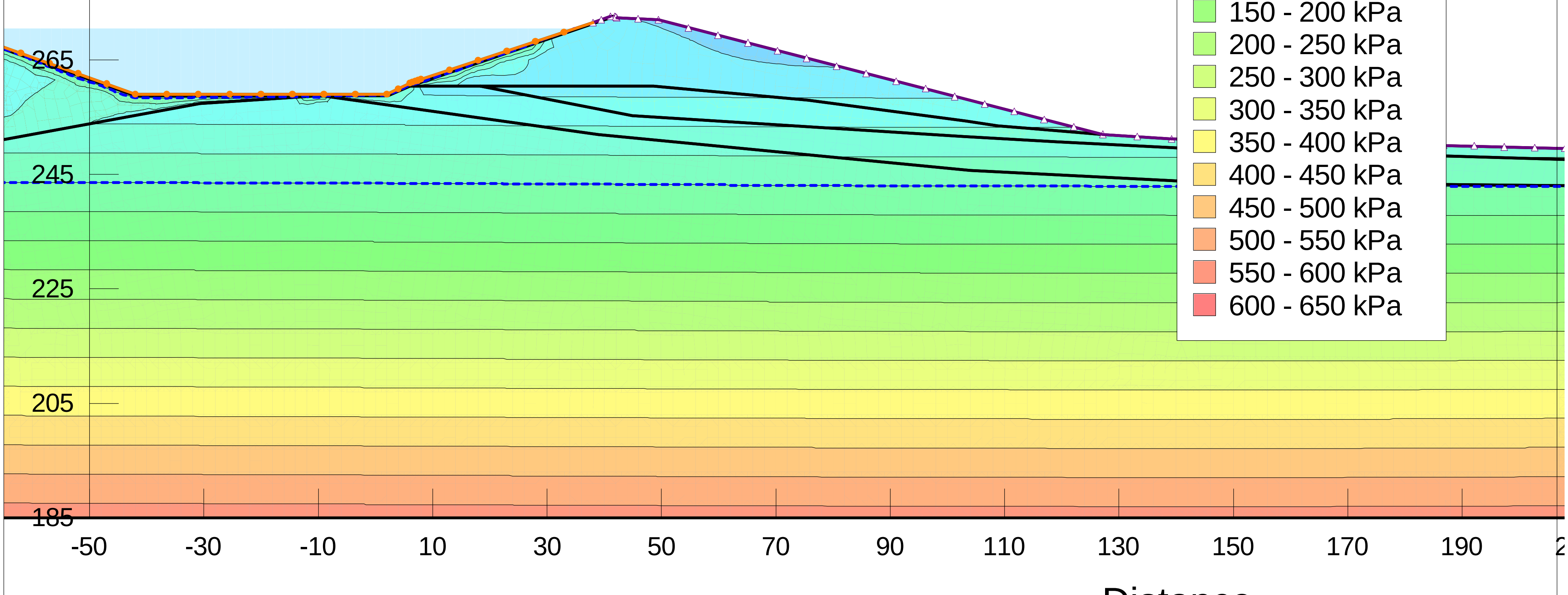
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 2.75 days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



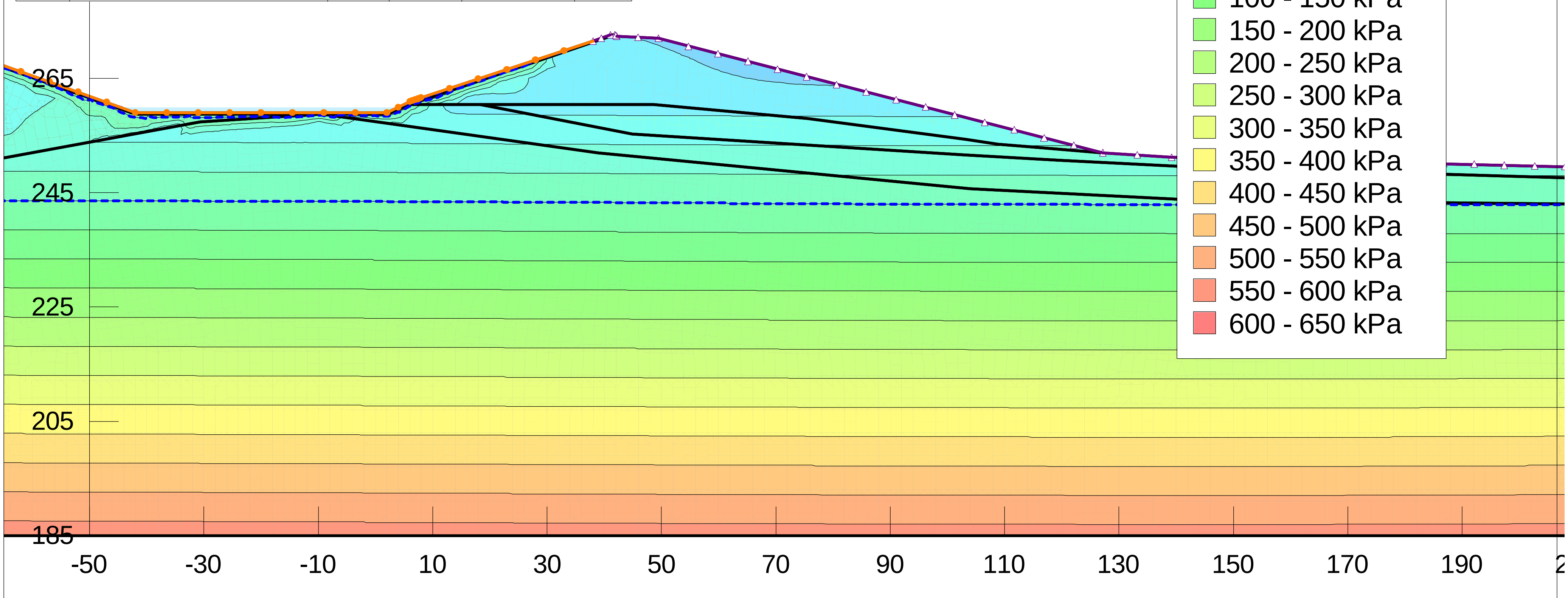
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz	Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 6 hours	Analysed by: CHEV
Comments: Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



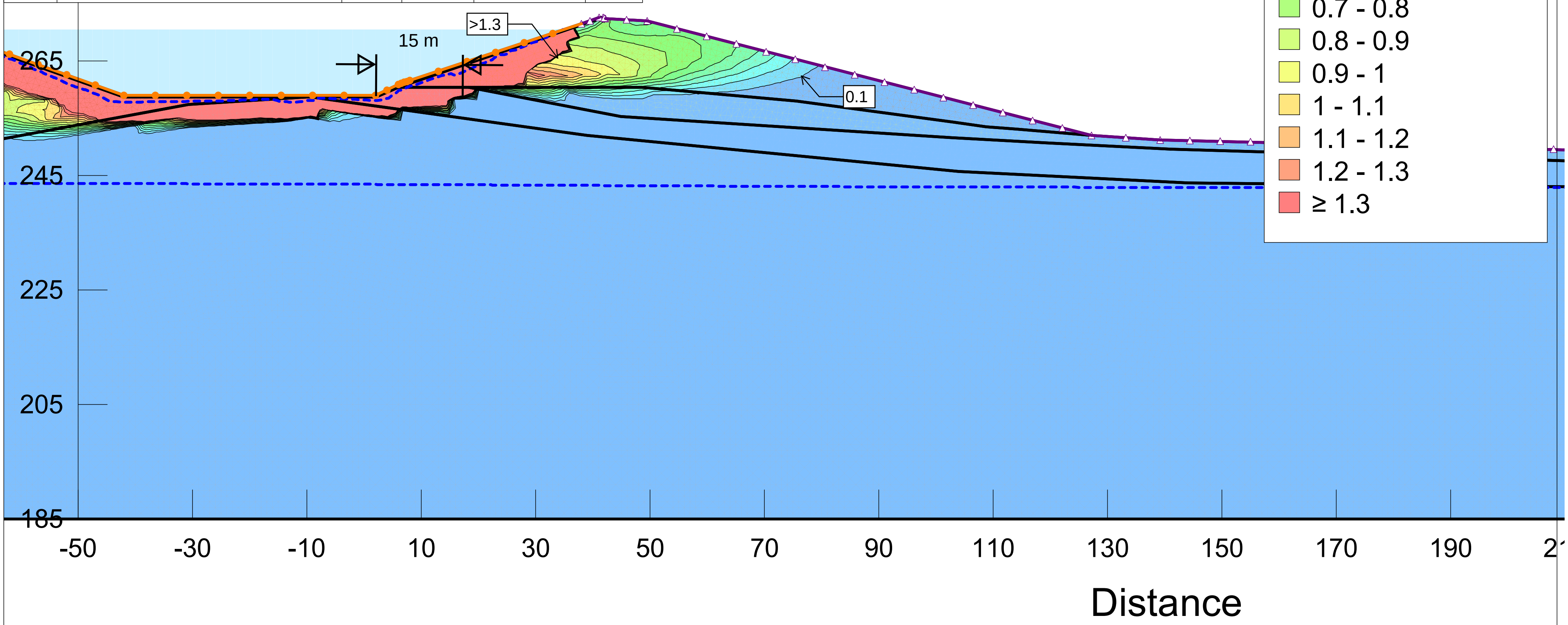
Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Best Estimate Perm) - 2.75days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz	Job Number: 1020688.4000
Analysis: Transient Seepage (Higher Perm) - 6 hours	Analysed by: CHEV
Comments: Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
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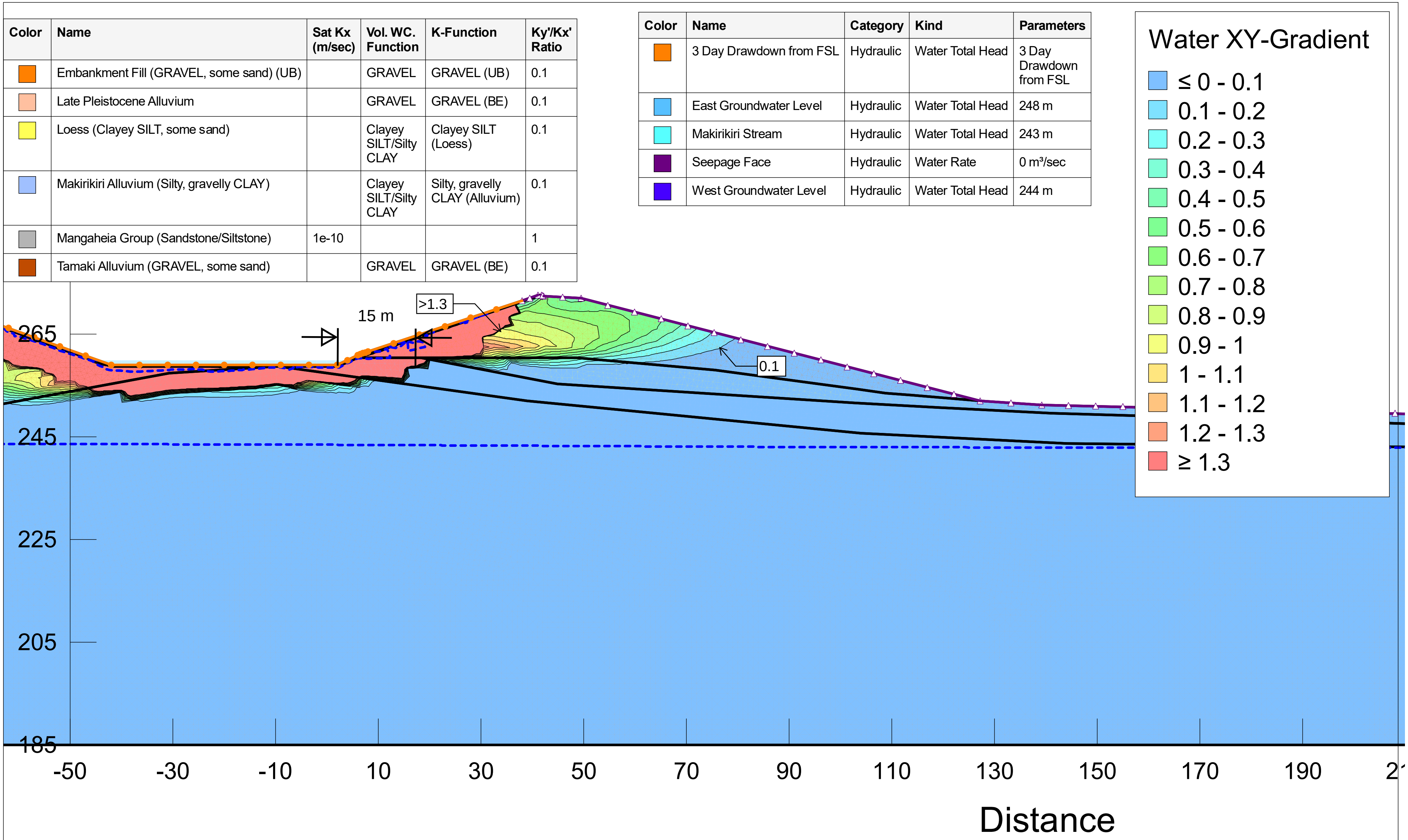
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Tonkin+Taylor	Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
	Analysis: Transient Seepage (Higher Perm) - 2.75 days		Analysed by: CHEV
	Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
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	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
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Color	Name	Category	Kind	Parameters
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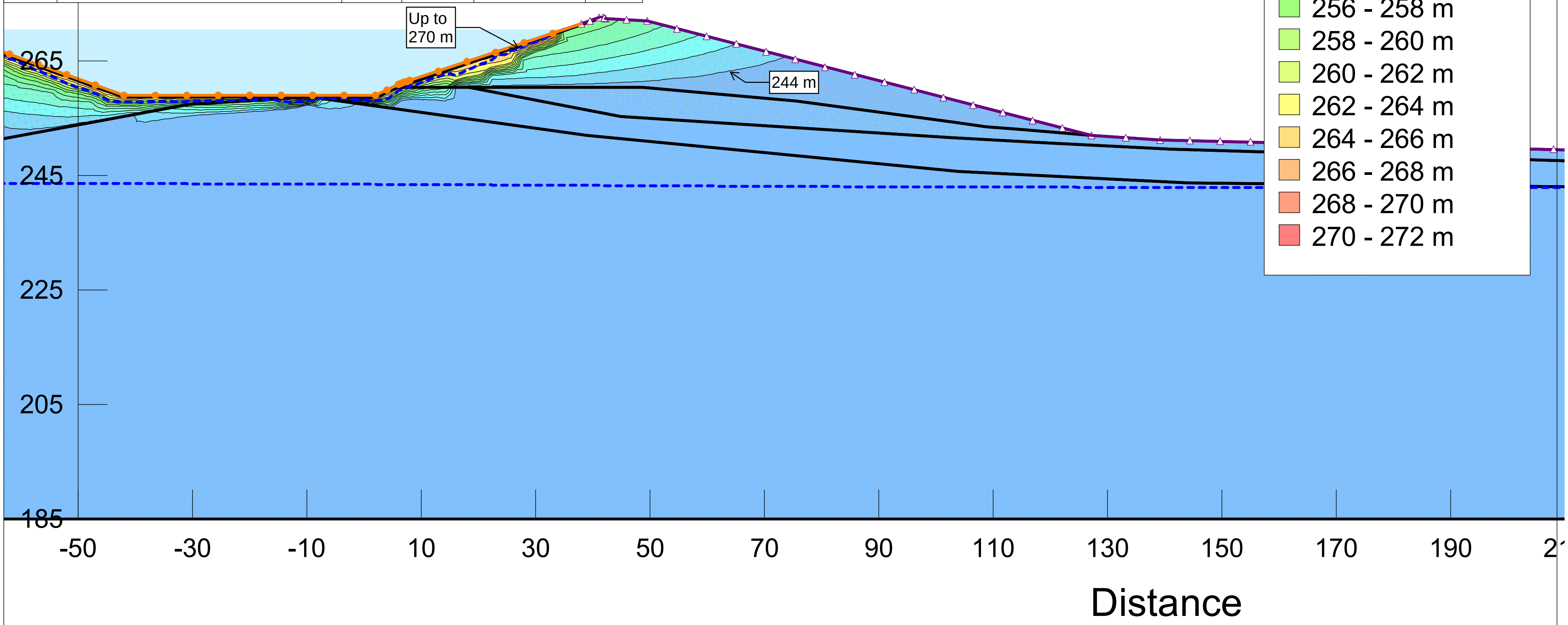
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

Water Total Head

- 242 - 244 m
- 244 - 246 m
- 246 - 248 m
- 248 - 250 m
- 250 - 252 m
- 252 - 254 m
- 254 - 256 m
- 256 - 258 m
- 258 - 260 m
- 260 - 262 m
- 262 - 264 m
- 264 - 266 m
- 266 - 268 m
- 268 - 270 m
- 270 - 272 m



Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz

Job Number: 1020688.4000

Analysis: Transient Seepage (Higher Perm) - 6 hours

Analysed by: CHEV

Comments:

Scale: 1:1,000 @ A4

Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
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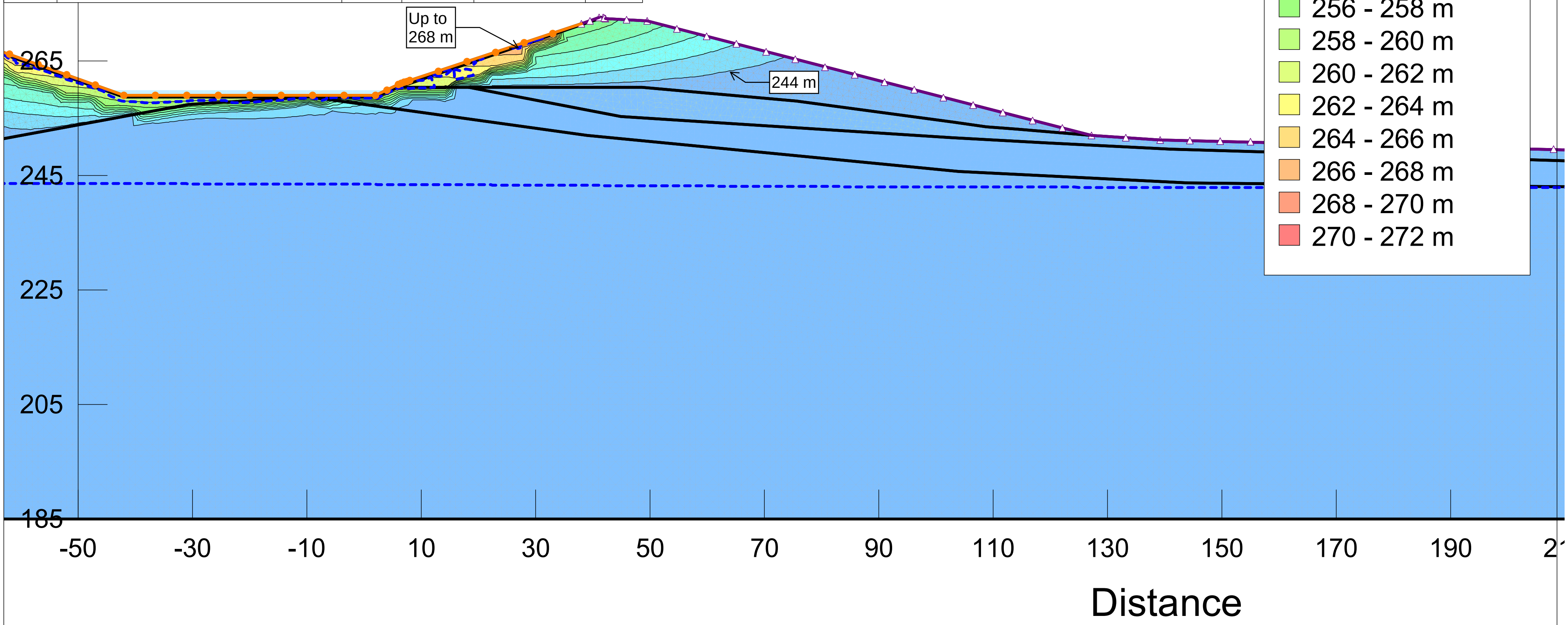
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Higher Perm) - 2.75 days		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

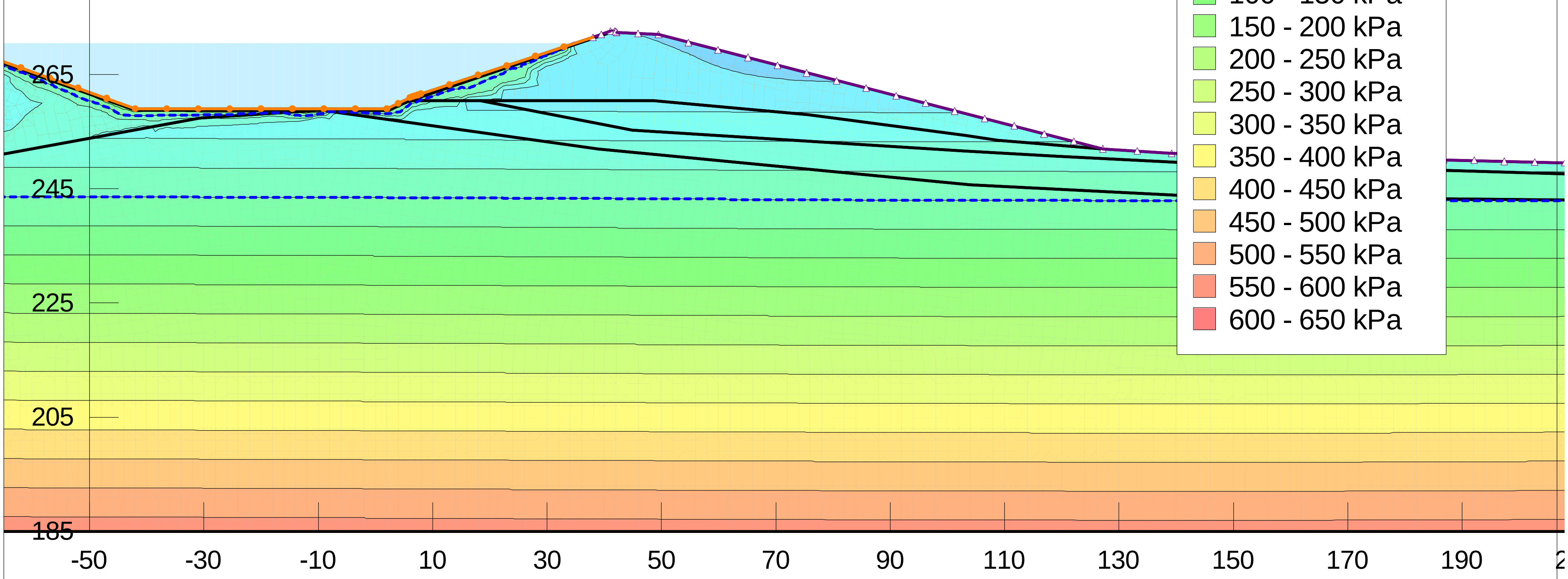
Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
	East Groundwater Level	Hydraulic	Water Total Head	248 m
	Makirikiri Stream	Hydraulic	Water Total Head	243 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m

Water Pressure

- -300 - -250 kPa
- -250 - -200 kPa
- -200 - -150 kPa
- -150 - -100 kPa
- -100 - -50 kPa
- -50 - 0 kPa
- 0 - 50 kPa
- 50 - 100 kPa
- 100 - 150 kPa
- 150 - 200 kPa
- 200 - 250 kPa
- 250 - 300 kPa
- 300 - 350 kPa
- 350 - 400 kPa
- 400 - 450 kPa
- 450 - 500 kPa
- 500 - 550 kPa
- 550 - 600 kPa
- 600 - 650 kPa



Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz		Job Number: 1020688.4000
Analysis: Transient Seepage (Higher Perm) - 6 hours		Analysed by: CHEV
Comments:	Scale: 1:1,000 @ A4	Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	K-Function	Ky'/Kx' Ratio
	Embankment Fill (GRAVEL, some sand) (UB)		GRAVEL	GRAVEL (UB)	0.1
	Late Pleistocene Alluvium		GRAVEL	GRAVEL (BE)	0.1
	Loess (Clayey SILT, some sand)		Clayey SILT/Silty CLAY	Clayey SILT (Loess)	0.1
	Makirikiri Alluvium (Silty, gravelly CLAY)		Clayey SILT/Silty CLAY	Silty, gravelly CLAY (Alluvium)	0.1
	Mangaheia Group (Sandstone/Siltstone)	1e-10			1
	Tamaki Alluvium (GRAVEL, some sand)		GRAVEL	GRAVEL (BE)	0.1

Color	Name	Category	Kind	Parameters
	3 Day Drawdown from FSL	Hydraulic	Water Total Head	3 Day Drawdown from FSL
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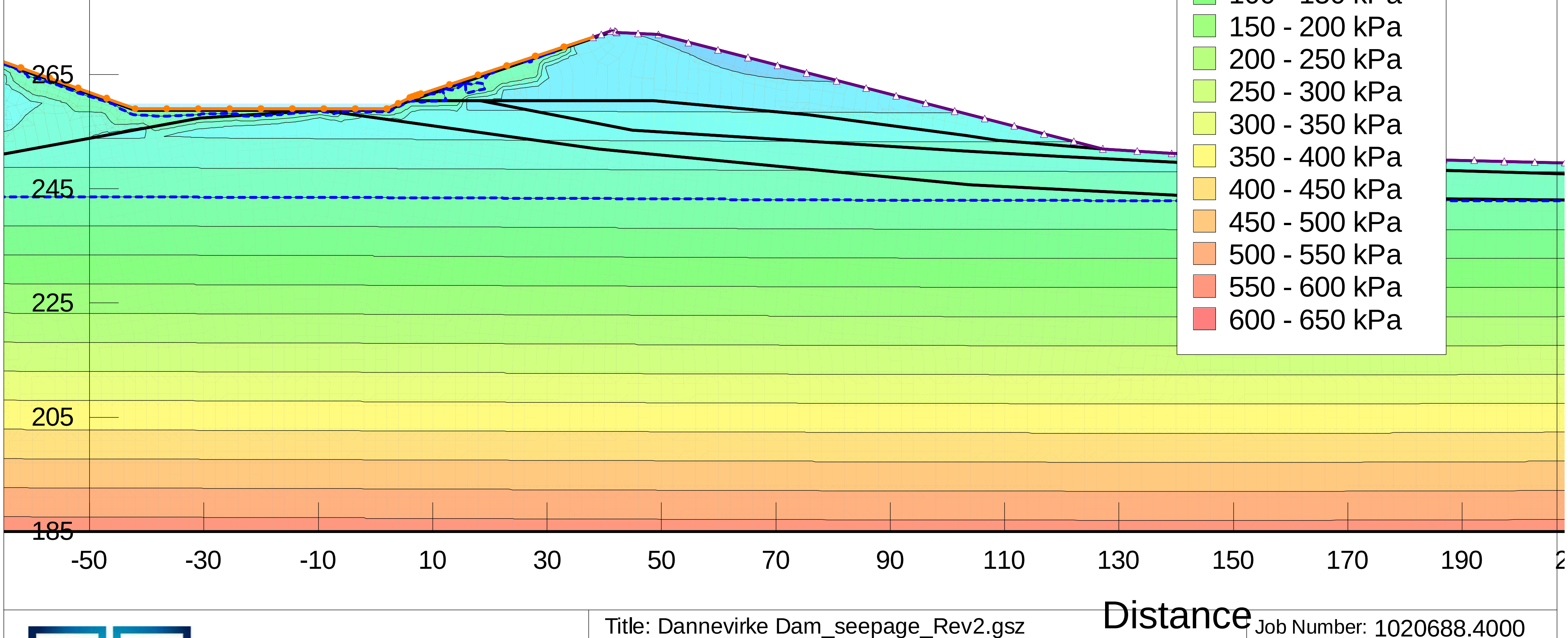
	Seepage Face	Hydraulic	Water Rate	0 m³/sec
	West Groundwater Level	Hydraulic	Water Total Head	244 m



Tonkin+Taylor

Title: Dannevirke Dam_seepage_Rev2.gsz

Job Number: 1020688.4000

Analysis: Transient Seepage (Higher Perm) - 2.75 days

Analysed by: CHEV

Comments:

Scale: 1:1,000 @ A4

Checked by: ABL

Directory: \\ttgroup.local\corporate\Wellington\TT Projects\1020688\1020688.4000\WorkingMaterial\05 Geotechnical\04 Slope Stability and Seepage\

Appendix D Internal erosion assessment

DESIGN CALCULATIONS

17 July 2024

Job Name:	Dannevirke Raw Water Reservoir	Revision No:	1
Job Location:	Dannevirke	Job No:	1020688.6000
Design Case:	Concentrated Leak Erosion	Designer:	JONE

Revision History

Ref	Scope	Reviewed by	Date checked	PD Review	Comments
1	Concentrated leak erosion assessment for Stage 2 Geotechnical Interpretation Report 1020688.6000 V1	DMK	17/7/24		

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Concentrated Leak Erosion - Conduit

Overview

With reference to the design shown on drawing 1020688.4000-5031 Rev 1, concentrated Leak erosion is considered:
1. Alongside the existing 10 m section of concrete backfill / encasement around the subsoil outlet pipe (to be retained).
2. Along the existing 200 mm diameter subsoil outlet pipe between the concrete backfill and 2050 mm diameter manhole (to be retained).
3. Alongside the 450 mm diameter weld jointed PE pipe and the stream outlet (to be retained).

Initiation assessment

Crack diameter

Following Fell et al (2015) Section 8.3.3.2 for collapse settlement of poorly compacted soil:

	Subsoil drain	Outlet pipe		
Pipe diameter	200	450	mm	
Poorly compacted soil thickness	100	225	mm	Assumed = 0.5 x diameter
Collapse strain	0.05	0.05	-	

Fell et al (2015) Table 8.8 assuming no formal compaction beneath pipe haunches:

Estimated collapse settlement	5	11.25	mm

Hydraulic shear stress

Following Fell et al (2015) Section 8.3.4.2 for cylindrical pipes

(a) Cylindrical pipe:

$$\tau = \rho_w \frac{gH_f d}{4L} \qquad (8.1)$$

(b) Vertical transverse crack

$$\tau = \frac{\rho_w \, gH_f^2 W}{2(H_f + W)L} \qquad (8.2)$$

where τ = Hydraulic shear stress in N/m^2
ρ_w = Density of water in kg/m^3
g = Acceleration due to gravity = 9.8 m/s^2
H_f = Head loss in pipe or crack due to friction in metres
L = Length of pipe or crack base in metres
D = Diameter of the pipe in metres
W = Width of crack in metres

ρw	1000	kg/m3	
g	9.8	m/s2	

Maximum Water Level	271.5	mRL	Levels from DWG
Subsoil pipe 4 invert level	260.9	mRL	1020688.4000-5033 Rev 1
Subsoil pipe 6 invert level	257.6	mRL	
Outlet pipe invert level	250.9	mRL	From As-Built DWG at Ø 900 mm MH

Design condition	1	2	3	4	
Hf	6.7	10.0	13.6	20.6	m

Design conditions detailed in calculation pack document. 1. Elevated leakage rates 2. Elevated leakage rates and blocked Subsoil Pipe 5 and 6. 3. HDPE liner damaged and drainage completely blocked. 4. HDPE and clay liner completely compromised, and drainage completely blocked.

L	125	125	125	125	m

Distance along subsoil pipe from section encased in concrete backfill to Ø 900 mm MH is approximately 125 m from Tararua As-Built drawings.

i=Hf/L	0.05	0.08	0.11	0.16	
d	0.01	0.01	0.01	0.01	m

Following collapse settlement assessment above

τ	1.3	2.0	2.7	4.0	N/m2

Critical hydraulic shear stress

The As-Built details (Sheet No. 11) indicate the outlet pipe is surrounded by clay backfill. It is assumed this is derived from, and so has the same grading as the Loess.

Material	Classifications	Best Estimate Erosion Rate Index (IHET)		Critical Shear Stress (τc) Pa	
		Fell (2015) Table 8.11		Fell (2015) Table 8.12	
		Best estimate	Likely Range	Best estimate	Likely Range
Loess derived clay backfill	USC: CL Dispersivity: non-dispersive	3 to 4	3 to 4.5	5	2 to 20

Table 8.11 Representative erosion rate index ($\tilde{I}_{HET}$) versus soil classification for non dispersive soils based on Wan and Fell (2002).

Unified Soil Classification	Erosion Rate Index ($\tilde{I}_{HET}$) *Likely Minimum*	*Best Estimate*	*Likely Maximum*
SM with <30% fines	1	<2	2.5
SM with >30% fines	<2	2 to 3	3.5
SC with <30% fines	<2	2 to 3	3.5
SC with >30% fines	2	3	4
ML	2	2 to 3	3
CL-ML	2	3	4
CL	3	3 to 4	4.5
CL-CH	3	4	5
MH	3	3 to 4	4.5
CH with Liquid Limit <65%	3	4	5
CH with Liquid Limit >65%	4	5	6

Notes. (1) Use best estimate value for best estimate probabilities. Check sensitivity if the outcome is strongly dependent on the results.
(2) For important decisions carry out Hole Erosion Tests, rather than relying on this table which is approximate.

Table 8.12 Approximate estimates and likely range of initial shear stress (τ_{C_0}) versus Hole Erosion Index (I_{HET}) (Fell *et al.*, 2008).

Hole Erosion Index (I_{HET})	Critical Shear Stress (τ_c) Pa — Non Dispersive Soil Behaviour: Best Estimate	Non Dispersive Soil Behaviour: Likely Range	Dispersive Soil Behaviour: Best Estimate	Dispersive Soil Behaviour: Likely Range
<2	2	1 to 5	1	0.5 to 2
2 to 3	2	1 to 5	1	0.5 to 2
3.5	5	2 to 20	2	1 to 5
4	25	10 to 50	5	2 to 10
5	60	25 to 100	5	2 to 10
6	100	60 to 140	5	2 to 10

Note. To be used with caution. For important decisions carry out Hole Erosion Tests to determine the critical shear stress (τ_c).

Continuation and Progression

Should internal erosion initiate, the 25% design does not include any filters around the conduit to prevent to the continuation of erosion.

Following Fell et al (2015) Table 8.16, assuming the pipe backfill to predominantly have a USC Soil Classification of CL, it is very likely the material can "support a roof". There is also no clear mechanism by which "crack filling" action could stop the erosion process.

The 10m section of concrete backfill section is expected to restrict the progression of a developing pipe, albeit there is also potential for cracks associated with the backfilled concrete trench (see trench cracking mechanism assessed separately).

Table 8.16 Likelihood of a soil being able to support a roof to an erosion pipe (Fell *et al.*, 2008).

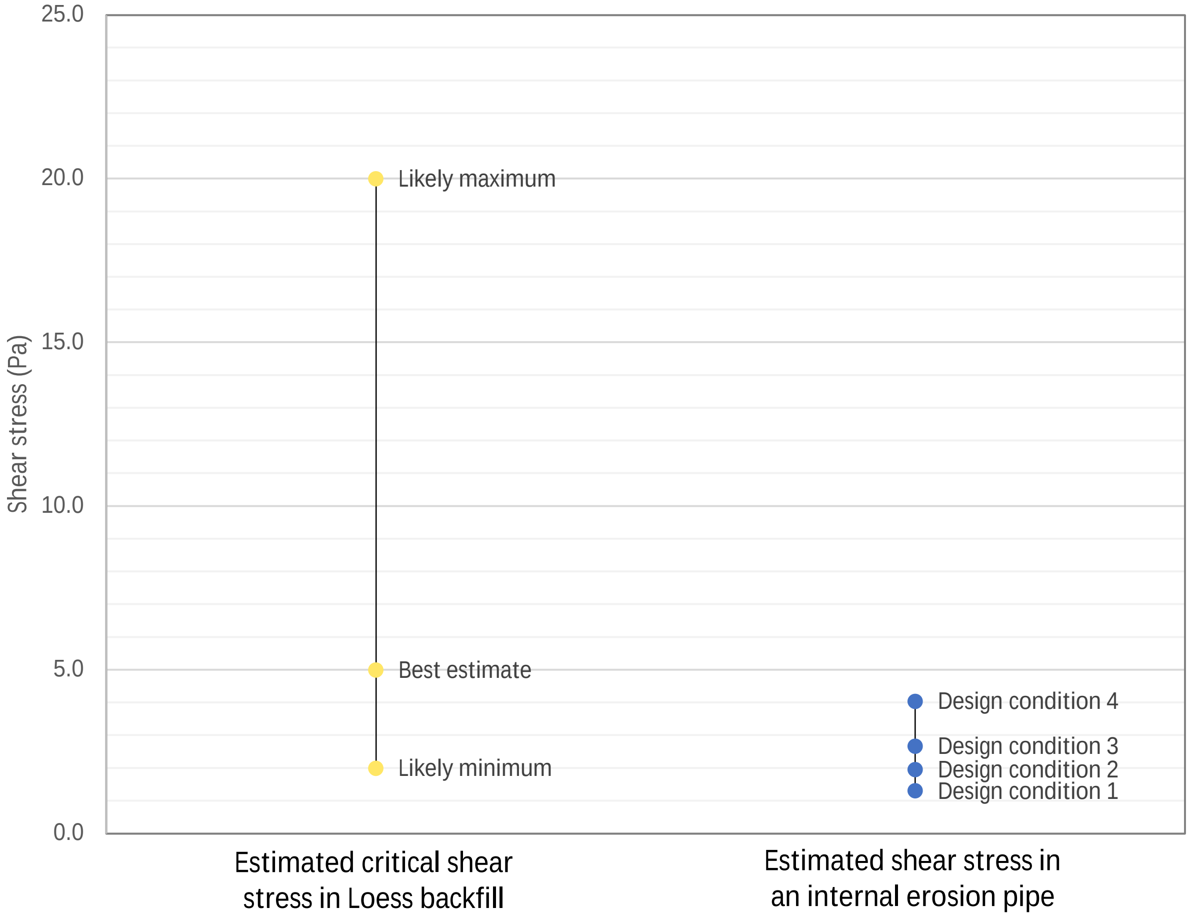
Soil Classification	*Percentage Fines*	*Plasticity of Fines*	*Moisture Condition*	*Likelihood of Supporting a Roof*	
Clays, sandy clays (CL, CH, CL-CH)	>50%	Plastic	Moist or saturated	1.0	Loess
ML or MH	>50%	Plastic or non-plastic	Moist or saturated	1.0	
Sandy clays, Gravely clays, (SC, GC)	15%–50%	Plastic	Moist or Saturated	1.0	
Silty sands, Silty gravels, Silty sandy gravel (SM, GM)	>15%	Non plastic	Moist Saturated	0.7 to 1.0 0.5 to 1.0	
Granular soils with some cohesive fines (SC-SP, SC-SW, GC-GP, GC-GW)	5% to 15%	Plastic	Moist Saturated	0.5 to 1.0 0.2 to 0.5	
Granular soils with some non plastic fines (SM-SP, SM-SW, GM-GP, GM-GW)	5% to 15%	Non plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05	
Granular soils, (SP, SW, GP, GW)	<5%	Non plastic Plastic	Moist and saturated Moist and saturated	0.0001 0.001 to 0.01	

Notes: (1) Lower range of probabilities is for poorly compacted materials (i.e. not rolled), and upper bound for well compacted materials.
(2) Cemented materials give higher probabilities than indicated in the table. If soils are cemented, use the category that best describes the particular situation.

Summary

Design condition	1	2	3	4
Hydraulic shear stress, τ (Pa)	1.3	2.0	2.7	4.0

	Likely minimum	Best estimate	Likely maximum
Critical Shear Stress, τc (Pa)	2	5	20



Concentrated Leak Erosion - Crack

Failure mechanism

Fell et al (2015) Section 8.3.2.9 reports that a concentrated leak may occur due to cracking in soil or extremely weathered rock in the sides of a trench.

Cracking may also occur due to differential settlement and stress concentrations caused by benching in the conduit excavation. (See photograph of conduit excavation below).

To assess these mechanisms, a crack at the depth of the conduit in the native soil (Makirikiri Alluvium and Loess) or Embankment Fill has been assessed (see Stage 2 Interpretative Report Geological Cross Section B-B' Rev 1 and C-C' Rev 1). A failure mechanism could feasibly pass through a combination of these materials. For completeness, Tamaki Alluvium has also been considered, albeit this in not present at the location of the conduit under the Eastern Embankment, but does form the western embankment foundation.

Figure 3.1: Deep trenches cut through eastern fill embankment during construction (photo dated 23 August 2012, taken from Google Earth, overlaid on as-built drawing sheet 2)

Initiation assessment

Crack width

The mechanisms under consideration, are taken to most closely relate to mechanism IM12 in Fell (2015); cracking of the lower part of the embankment due to differential settlement due to small scale irregularities in the foundation profile.

Table A2.20:
Persistence: The aerial photograph above indicates steep-sided excavations associated with the outlet pipe are primarily located under the upstream shoulder. LF=2 is adopted.
Profile: The maximum thickness of placed material above the outlet is approximately 15m. It is considered credible that benching of up to 10% of this height was used for outlet construction. LF=4 is adopted.
Core geometry: The embankment is considered to be homogenous, and so a wide core is considered. LF=1 is adopted.

Table A2.28:
ΣRFxLF=3x2+2x4+1x1=15, giving a maximum likely crack width of 1-2mm. It can be seen a upper bound of crack width = 10mm is given for IM12.

Table A2.20 Factors influencing the likelihood of cracking or hydraulic fracturing in the middle and lower parts of embankment dams due to small scale irregularities in the foundation profile under the core (IM12).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)* Less Likely (1)	Neutral (2)	More Likely (3)	Much More Likely (4)
Persistence of the irregularity across the core	(3)	Persistent across less than 50% of the core[b]	Persistent 50% to 75% across the core width	Persistent 75% to 90% across the core	Persistent 90% to 100% across the core
Small scale irregularities in abutment profile	(2)	Uniform abutment profile, or irregularities treated by slope modification	Steps, benches, depressions in rock foundations less than 3% of the embankment height	Steps, benches, depressions in rock foundations 3% to 5% of the embankment height	Steps, benches, depressions in rock foundation 5% to 10% of the embankment height[a]
Core geometry Width (W)/ Height (H)	(1)	Wide core, W/H > 1.5	0.5 < W/H < 1.5	Narrow core, 0.25 < W/H < 0.5	Very narrow core, W/H < 0.25

Notes: (a) Larger irregularities are covered in Section A2.3.2.
(b) An irregularity with less than 50% persistence across the core is assumed to have a negligible contribution to the probability of a transverse crack.

Table A2.28 Maximum likely width of cracking in the dam versus ∑ (Relative importance factor) x (Likelihood factor) for cracking in the middle and lower parts of the dam.

Crack formation mechanism	*Maximum likely crack width in millimeters relative to (relative importance factor) x (likelihood factor)* 6 to 9	9 to 11	11 to 13	13 to 18	18 to 24
Cross valley differential settlement Table A2.16	0	0	1 to 2	2 to 10	10 to 20
Differential settlement causing arching of the core onto the shoulders Table A2.18	0	0	0 to 1	1 to 2	2 to 10
Differential settlement over small scale irregularities in the foundation Table A2.20	0	0	0 to 1	1 to 2	2 to 10
Differential settlements due to settlements in the foundation Table A2.9	0	0	0 to 2	2 to 10	10 to 20
Differential settlement causing arching of the core in the cut off trench Table A9.3 of Piping Toolbox	0	0	0 to 1	1 to 2	2 to 10

Hydraulic shear stress

Following Fell et al (2015) Section 8.3.4.2 for a Vertical transverse crack

(a) Cylindrical pipe:

$$\tau = \rho_w \frac{gH_f d}{4L} \tag{8.1}$$

(b) Vertical transverse crack

$$\tau = \frac{\rho_w gH_f^2 W}{2(H_f + W)L} \tag{8.2}$$

where τ = Hydraulic shear stress in N/m^2
ρ_w = Density of water in kg/m^3
g = Acceleration due to gravity = 9.8 m/s^2
H_f = Head loss in pipe or crack due to friction in metres
L = Length of pipe or crack base in metres
D = Diameter of the pipe in metres
W = Width of crack in metres

ρw	1000	kg/m3
g	9.8	m/s2

Design condition	1	2	3	4	
Hf	6.7	10.0	13.6	20.6	m
See conduit concentrated leak assessment					
L	125	125	125	125	m
Distance from toe of upstream shoulder to toe of downstream shoulder					
i=Hf/L	0.05	0.08	0.11	0.16	
W (best guess maximum)	0.002	0.002	0.002	0.002	m
τ (for best guess crack width)	0.5	0.8	1.1	1.6	N/m2
W (sensitivity)	0.01	0.01	0.01	0.01	m
τ (for upper estimate crack width)	2.6	3.9	5.3	8.1	N/m2

Critical hydraulic shear stress

Material	Classifications	Best Estimate Erosion Rate Index (IHET)		Critical Shear Stress (τc) Pa	
		Fell (2015) Table 8.11		Fell (2015) Table 8.12	
		Best estimate	Likely Range	Best estimate	Likely Range
Embankment Fill	GC/GM (say SM with <30% fines). Non-dispersive.	<2	1 to 2.5	2	1 to 5
Loess	CL. Non-dispersive.	3 to 4	3 to 4.5	5	2 to 20
Makirikiri Alluvium	GC/SC/CL (say SC with >30% fines). Dispersive.	3	2 to 4	1	0.5 to 2
Tamaki Alluvium (included for completeness)	GW (say SC with <30% fines) Assumed dispersive.	2 to 3	<2 to 3.5	1	0.5 to 2

Table 8.11 Representative erosion rate index ($\tilde{I}_{HET}$) versus soil classification for non dispersive soils based on Wan and Fell (2002).

	Erosion Rate Index ($\tilde{I}_{HET}$)		
Unified Soil Classification	*Likely Minimum*	*Best Estimate*	*Likely Maximum*
SM with <30% fines	1	<2	2.5
SM with >30% fines	<2	2 to 3	3.5
SC with <30% fines	<2	2 to 3	3.5
SC with >30% fines	2	3	4
ML	2	2 to 3	3
CL-ML	2	3	4
CL	3	3 to 4	4.5
CL-CH	3	4	5
MH	3	3 to 4	4.5
CH with Liquid Limit <65%	3	4	5
CH with Liquid Limit >65%	4	5	6

Notes. (1) Use best estimate value for best estimate probabilities. Check sensitivity if the outcome is strongly dependent on the results.
(2) For important decisions carry out Hole Erosion Tests, rather than relying on this table which is approximate.

Table 8.12 Approximate estimates and likely range of initial shear stress (τ_{Co}) versus Hole Erosion Index (I_{HET}) (Fell *et al.*, 2008).

Hole Erosion Index (I_{HET})	*Critical Shear Stress (τ_c) Pa*			
	Non Dispersive Soil Behaviour		*Dispersive Soil Behaviour*	
	Best Estimate	*Likely Range*	*Best Estimate*	*Likely Range*
<2	2	1 to 5	1	0.5 to 2
2 to 3	2	1 to 5	1	0.5 to 2
3.5	5	2 to 20	2	1 to 5
4	25	10 to 50	5	2 to 10
5	60	25 to 100	5	2 to 10
6	100	60 to 140	5	2 to 10

Note. To be used with caution. For important decisions carry out Hole Erosion Tests to determine the critical shear stress (τ_c).

Continuation and Progression

Should internal erosion initiate, the 25% design does not include any filters around the conduit to prevent to the continuation of erosion.
For the materials present, from the table below it is reasonable to consider that they could support a roof.

Table 8.16 Likelihood of a soil being able to support a roof to an erosion pipe (Fell *et al.*, 2008).

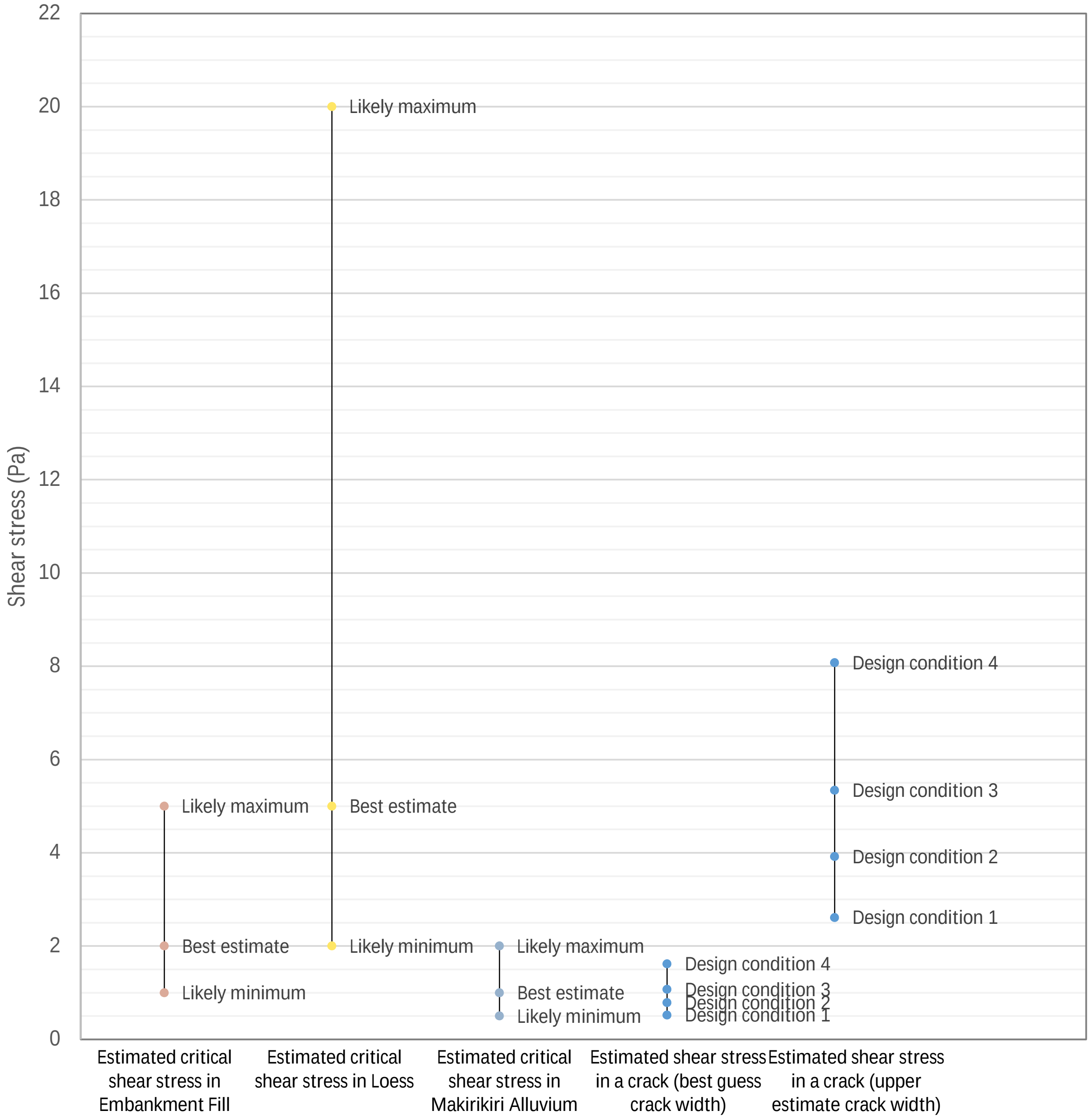
Soil Classification	*Percentage Fines*	*Plasticity of Fines*	*Moisture Condition*	*Likelihood of Supporting a Roof*
Clays, sandy clays (CL, CH, CL-CH)	>50%	Plastic	Moist or saturated	1.0 Loess
ML or MH	>50%	Plastic or non-plastic	Moist or saturated	1.0
Sandy clays, Gravely clays, (SC, GC)	15%–50%	Plastic	Moist or Saturated	Embankment Fill 1.0 Makirikiri Alluvium
Silty sands, Silty gravels, Silty sandy gravel (SM, GM)	>15%	Non plastic	Moist Saturated	0.7 to 1.0 0.5 to 1.0
Granular soils with some cohesive fines (SC-SP, SC-SW, GC-GP, GC-GW)	5% to 15%	Plastic	Moist Saturated	0.5 to 1.0 0.2 to 0.5 Tamaki Alluvium
Granular soils with some non plastic fines (SM-SP, SM-SW, GM-GP, GM-GW)	5% to 15%	Non plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05
Granular soils, (SP, SW, GP, GW)	<5%	Non plastic Plastic	Moist and saturated Moist and saturated	0.0001 0.001 to 0.01

Notes: (1) Lower range of probabilities is for poorly compacted materials (i.e. not rolled), and upper bound for well compacted materials.
(2) Cemented materials give higher probabilities than indicated in the table. If soils are cemented, use the category that best describes the particular situation.

Summary

Design condition	1	2	3	4
Hydraulic shear stress, τ (Pa) for best guess crack width	0.5	0.8	1.1	1.6
Hydraulic shear stress, τ (Pa) for upper estimate crack width	2.6	3.9	5.3	8.1

Critical Shear Stress, τc (Pa)	Likely minimum	Best estimate	Likely maximum
Embankment Fill	1	2	5
Loess	2	5	20
Makirikiri Alluvium	0.5	1	2



Appendix E Hole Erosion Test (HET) Interpretation

E1 Introduction

The risk of internal erosion to Dannevirke Raw Water Reservoir in its existing situation, and following proposed remedial works in accordance with the 25% Detailed Design, is discussed in Section 5 of this report. It is identified that for the concentrated leak erosion mechanism of internal erosion there is:

- A "possible risk" to the reservoir in the existing situation, and following remediation a "low risk during normal operation", relating to the potential failure mode shown in Figure 0.1 (refer Section 5.1 and Table 5-6 therein).
- A "possible" risk of internal erosion through cracks induced by the Safety Evaluation Earthquake (SEE), which is a separate potential failure mode (refer Section 5.2).

For concentrated leak erosion to initiate, hydraulic shear stresses in a crack in the embankment must exceed the critical shear stress at which the in-situ soil's resistance to erosion is exceeded.

For the concentrated leak erosion assessment in Appendix D, the assumed resistance of the soil is largely based on empirical correlations. Four Hole Erosion Tests (HET) have been conducted on the dam Embankment Fill and Loess Borrow Material, to provide a direct measure of the soils' resistance to erosion, and reduce uncertainty in the concentrated leak erosion assessment.

E2 Test summary

A Standard and Critical Head HET was carried out on each of the bulk samples of Embankment Fill and Loess Borrow Material. For the HETs, the samples were compacted at the respective optimum moisture content, to 98% of the maximum dry density.

The HET test results and supporting Maximum Dry Density test results are presented at the end of this Appendix and are summarised in Table E-1.

Table E-1: HET test summary

Hole	Sample depth (m)	Geology	Sample description	Maximum dry density (t/m^3)	Optimum moisture content (%)	HET type	Initial density ratio (%)	Initial moisture content (%)	Test differential head (mm)	Sample length (mm) Note 2	Sample diameter (mm) Note 3	Tester comments Note 4	Critical Shear Stress, τ_c, or Initial Shear Stress, τ_0, (N/m^2)	Coefficient of Soil Erosion, C_e (s/m)
BH01	8.6 - 10.1	Embankment fill	Gravelly SILT with sand; brown	1.91 Note 1	14.0 Note 1	Critical Head	98.0	14.0	Initial: 30 Final: 600	Start: 115.2 Final: 84	Start: 6.0 Final: 6.1	There may be initiation of erosion around 300mm to 500 mm [differential] head.	τ_0 = 50 to 85 Note 5	Not determined for Critical Head test.
BH01	8.6 - 10.1	Embankment fill	Gravelly SILT with sand; brown			Standard	98.0	14.0	500	Start: 115.2 Final: 85	Start: 6.0 Final: 6.0	Some erosion.	τ_c not clearly defined as very little erosion. τ_0 > 85 Note 6	Not clearly defined. Note 6
TP05	1.0	Loess	Sandy SILT; brown	1.75	18.0	Critical Head	98.0	18.0	Initial: 30 Final: 900	Start: 115.6 Final: 95	Start: 6.0 Final: 6.0	Very little erosion up to 900mm maximum [differential] head height.	τ_0 > 135 Note 7	Not determined for Critical Head test.
TP05	1.0	Loess	Sandy SILT; brown			Standard	98.0	18.0	900	Start: 115.2 Final: 96	Start: 6.0 Final: 6.0	Very little erosion observed.	τ_c not clearly defined as very little erosion. τ_0 > 135 Note 7	Not clearly defined as very little erosion. Note 7

Note:

1 Maximum dry density and optimum moisture content test results taken from test on sample from TP01.

2 Final sample length taken as the length of the hole excluding bevelling due to slaking at the upstream and downstream face of the sample.

3 Final diameter taken as the average diameter of the sample over the length of the sample, excluding the bevelling at the upstream and downstream face of the sample.

4 Received from David Brooke (GHD) on 11/4/2024.

5 Considering initiation of erosion between 300mm to 500 mm differential head.

6 Whilst some erosion was reported by the Tester, the results attached at the end of this appendix indicate that erosion was principally through slaking at the upstream and downstream face of the sample, and that there was no discernible change in the hole diameter. Consequently, the Coefficient of Soil Erosion, C_e, and Critical Shear Stress, τ_c, are not clearly defined. It is assumed that the Initial Shear Stress has not been reached.

7 There is no clear indication that the Initial Shear Stress has been reached.

E3 Test interpretation

The test results have been interpreted following the methodology described in Wan and Fell (2004)[4], such that the hydraulic shear stress at time, t, is defined as:

$$\tau_t = \rho_w g s_t \frac{\emptyset_t}{4} \qquad \text{Equation 1}$$

Where:

ρ_w is the density of the eroding fluid (998 kg/m^3).

g is the acceleration due to gravity (9.8 m/s^2).

s_t is the hydraulic gradient at time, t, calculated to reflect the change in length of the sample during the test due to slaking at the upstream and downstream faces.

$\emptyset_t$ is the diameter of the hole at time, t.

The change in length of the sample has been assumed to occur linearly with time between the initial and final values.

The results from the test result interpretation are presented in Table E-1. The results are presented in terms of the Critical Shear Stress and Initial Shear Stress, where the Critical Shear Stress is determined from Standard type HETs by extrapolation of the interpreted erosion rate back to a zero rate, whereas the Initial Shear Stress is generally determined from Critical Head type HETs from observation of when erosion initiates. (A lower estimate of the Initial Shear Stress has been determined from the Standard type HETs where erosion was not observed to initiate during the test).

E4 Conclusion

The Critical Shear Stress values estimated following the empirical correlations in Fell et al. (2015)[5], which were used for the assessment of concentrated leak erosion in Appendix D, are presented in Table E-2. The results are compared with the Initial Shear Stress values interpreted from the HET results as presented in Table E-1.

Table E-2: Comparison of the previously estimated Critical Shear Stress with the Initial Shear Stress determined from the HET test results

Geology	Estimated Critical Shear Stress following empirical correlations in Fell et al. (2015) per Appendix D		Initial Shear Stress from HET test results (N/m^2)
	Best estimate (N/m^2)	Likely range (N/m^2)	
Embankment fill	2	1 to 5	>50
Loess	5	2 to 20	>135

[4] Wan, C. F. and Fell, R., 2004, "Laboratory Tests on the Rate of Piping Erosion of Soils in Embankment Dams," *Geotechnical Testing Journal*, Vol. 27, No. 3, pp. 1-9.
[5] Fell, R., MacGregor, D., Stapledon, D., Bell, G., Foster, M., 2015, "*Geotechnical Engineering of Dams, 2nd edition*," Taylor & Francis Group.

The Initial Shear Stresses determined from the HETs are significantly higher for both the Embankment Fill and Loess than was previously assumed in the assessment of concentrated leak erosion presented in Appendix D. In that assessment, the upper estimate of driving shear stresses was <10 N/m^2; significantly less that the resistances presented in Table E-1. Consequently, if the HET results are representative of the in-situ Embankment fill, Loess, and Loess derived clay backfill, they indicate that the likelihood of initiation of concentrated leak erosion in both the existing situation, and following remediation is lower (better) than indicated in Appendix D.

However, this conclusion must be considered alongside the following residual uncertainties:

- Only two samples have been tested. Natural variability of the materials may mean material of lesser erosive resistance to that measured is present in-situ.
- Erosive resistance can be significantly impacted by the degree of compaction and saturation rate, both of which may vary significantly in-situ from the tested samples.
- HET tests have not been conducted in the Makirikiri Alluvium. Consequently, a higher degree of uncertainty remains for concentrated leaks occurring through this foundation material. Albeit that for the failure modes considered in this report, it is considered unlikely that a concentrated leak failure mechanism could develop exclusively through this material, due to its depth beneath the downstream shoulder.
- The risk of concentrated leak erosion through cracks caused by a SEE have not been assessed quantitatively in the concentrated leak erosion assessments conducted to date.

In summary, the HET results indicate a lower likelihood of concentrated leak erosion than that calculated in Appendix D but key conclusions remain unchanged due to variability of natural materials and other areas of uncertainty i.e., relating to size and nature of in situ cracks or internal erosion pipes.

E5 Laboratory test results

GEOTECHNICS
Level 4
2 Hunter Street
Wellington 6011
New Zealand
p. +64 4 381 8584

Geotechnics Project ID	1092032.0000.2.0
Customer Project ID	1020688.42
Customer Project Name	Dannevirke

DETERMINATION OF THE DRY DENSITY / WATER CONTENT RELATIONSHIP
NZS 4402:1986 Test 4.1.1 (Standard Compaction)

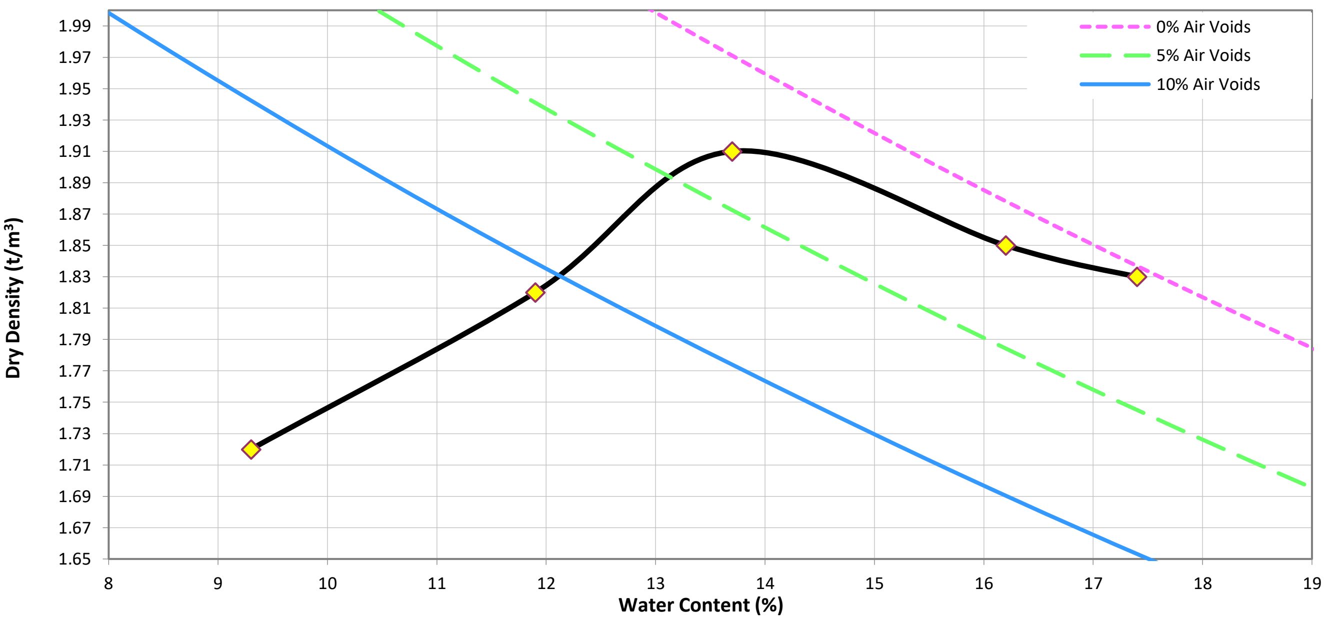
VANE SHEAR STRENGTH OF COHESIVE SOIL - NZGS GUIDELINE FOR HAND HELD SHEAR VANE TEST - 2001

TEST DETAILS

LOCATION	**ID**	TP01		
	Description	Dannevirke		
	Data	N/A		
SAMPLE	**Geotechnics ID**	GWL-202309130	**Date Received**	Unknown
	Reference	0.3m	**Depth**	0.30m
	Description	Gravelly SILT, with some sand; brown. Moist.		
SPECIMEN	**Reference**	1	**Depth**	N/A
	Description	N/A		

TEST RESULTS

Dry Density / Water Content Relationship



Maximum Dry Density	Optimum Water Content	Solid Density*	Whole Sample NWC
1.91 t/m³	14%	2.70 t/m³	13.5%

Natural Water Content (NWC)			✓					
Water Content (%)	9.3	11.9	13.7	16.2	17.4			
Dry Density (t/m³)	1.722	1.817	1.913	1.852	1.831			
Undrained Shear Strength (kPa)	UTP	UTP	137	19	0			

TEST REMARKS

• The material used for testing was natural, whole soil. • *Use of assumed values in calculations is at the customers discretion and risk. • The amount of material retained on a 19mm sieve was 22% by wet mass.

This test result is IANZ accredited.

Approved by KTP	JMG	**Date**	5/10/2023

HOLE EROSION TEST (HET)

F9.1.64

Test Procedure : 1. Wan,C.F & Fell,R (2002)UNCIV Report No R-412 ISBN:85841 379 5 2.Wan,C.F & Fell,R (2004a) ASCE Vol 130 No 4 pp 373-380
3.Wan,C.F & Fell,R (2004b) ASTM Vol 27, No 3 pp 295-303

Client:	**Tonkin Taylor**	**BH No:**	**BH01**	**Job No:**	**12626698**	**Report No SYD2302889.1**
Project:	**Dannevirke Dam**	**Depth (m):**	**8.6-10.1**	**GHD Lab Sample No:**	**SYD23-0521-01**	
Location:	**Dannevirke, NZ**	**HET Type :**	**Critical Head**	**Client Sample ID:**	**BH01**	

Test No. :	**HET01**
Test Date:	**13-Nov-23**
Test by:	**JV**
Bulk sample SMDD (t/m³) :	n/a
Bulk Sample OMC (%):	n/a
Targeted Density Ratio (%):	98.0%
Targeted Water Content (%):	OMC
Conditioning moisture content %:	**14.0%**
Moisture Content from test :	14.0%
Upstream water head at start of test(mm):	n/a
Fluid for conditioning soil :	Syd Tap Water
Eroding fluid :	Syd Tap Water
Eroding fluid mean temperature (°C) :	21.0
Diameter of hole (mm):	6.0

% Moist & Density Before	
Soil + mould	4412
wt mould	2273
vol mould	1003.0
wt wet soil	2139
wet density	2.133
cont No	14A
wet + cont (g)	313.6
dry + cont (g)	294.1
cont (g)	154.6
moisture %	14.0%
DryDensity(t/m³):	**1.871**
Density ratio :	**98.0%**
Moisture ratio:	**100.0%**

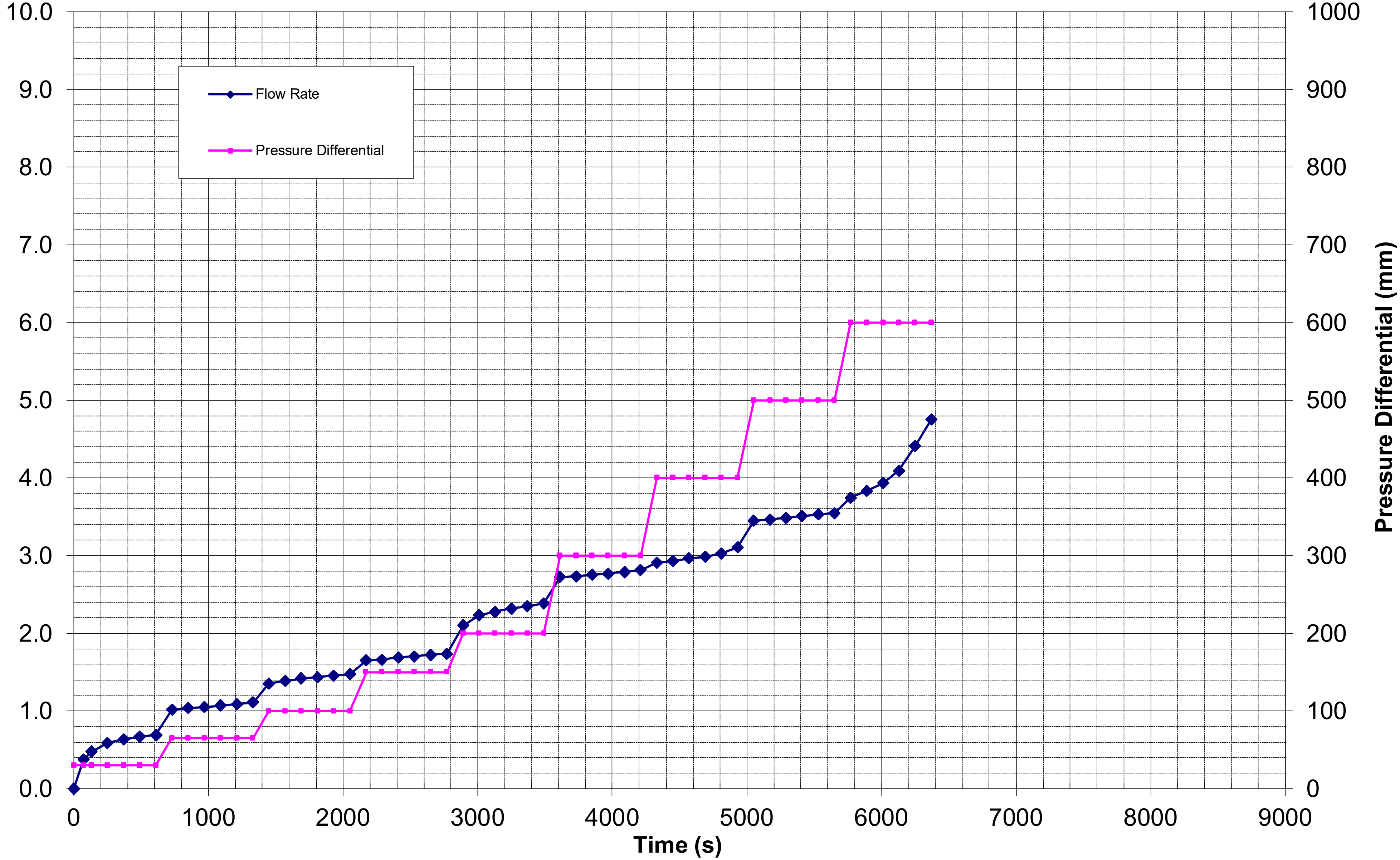
Initial remoulded Density	
Target Wet Dens.(t/m³)	
Total Wt Req'd (g)	
Wt per Layer (g)	
Ø (cm) =	10.53
Ht (cm) =	11.52
HET Mould	*A*
MDD	1.91
OMC	14.0%
POST MC	HET01
cont No	E49
wet + cont (g)	528.7
dry + cont (g)	469.4
cont (g)	73
moisture %	15.0%

Sample Description: Gravelly SILT with sand; brown

(1) Time start (mins)	(2) Time (1) * 60 (s)	Outflow mls	(3) Flow rate (L/min)	(4) Head in U/S Tube (mm)	(5) Head in D/S Tube (mm)	(6) Head Diff. (4) - (5) (mm)	(7) Remarks
0.0	0	0	0.00	355	325	30	
1.0	70	125	0.38	355	325	30	eroding particles clear flow
2.0	130	159	0.48	355	325	30	particles flowing into DS
4.0	250	196	0.59	355	325	30	minor ds face erosion
6.0	370	212	0.64	355	325	30	
8.0	490	222	0.67	355	325	30	
10.0	610	230	0.69	355	325	30	
12.0	**730**	**339**	**1.02**	**390**	**325**	**65**	
14.0	850	345	1.04	390	325	65	eroding particles clear flow
16.0	970	350	1.05	390	325	65	minor ds face erosion
18.0	1090	357	1.07	390	325	65	particles flowing into DS
20.0	1210	363	1.09	390	325	65	
22.0	1330	371	1.11	390	325	65	
24.0	**1450**	**451**	**1.35**	**425**	**325**	**100**	
26.0	1570	462	1.39	425	325	100	eroding particles clear flow
28.0	1690	473	1.42	425	325	100	particles flowing into DS
30.0	1810	479	1.44	425	325	100	minor ds face erosion
32.0	1930	486	1.46	424	324	100	
34.0	2050	491	1.47	425	325	100	
36.0	**2170**	**550**	**1.65**	**475**	**325**	**150**	
38.0	2290	554	1.66	475	325	150	
40.0	2410	563	1.69	475	325	150	eroding particles clear flow
42.0	2530	568	1.70	475	325	150	particles flowing into DS
44.0	2650	574	1.72	475	325	150	
46.0	2770	579	1.74	475	325	150	
48.0	**2890**	**701**	**2.10**	**525**	**325**	**200**	
50.0	3010	744	2.23	525	325	200	
52.0	3130	759	2.28	525	325	200	eroding particles clear flow
54.0	3250	772	2.32	525	325	200	particles flowing into DS
56.0	3370	783	2.35	525	325	200	
58.0	3490	795	2.39	525	325	200	

60.0	**3610**	**908**	**2.72**	**625**	**325**	**300**	
62.0	3730	911	2.73	625	325	300	
64.0	3850	917	2.75	625	325	300	eroding particles clear flow
66.0	3970	923	2.77	625	325	300	particles flowing into DS
68.0	4090	930	2.79	625	325	300	minor ds face erosion
70.0	4210	937	2.81	625	325	300	
72.0	**4330**	**971**	**2.91**	**725**	**325**	**400**	
74.0	4450	976	2.93	725	325	400	eroding particles clear flow
76.0	4570	988	2.96	725	325	400	particles flowing into DS
78.0	4690	994	2.98	725	325	400	minor ds face erosion
80.0	4810	1009	3.03	725	325	400	
82.0	4930	1035	3.11	725	325	400	
84.0	**5050**	**1149**	3.45	**825**	**325**	**500**	
86.0	5170	1155	3.47	825	325	500	eroding particles clear flow
88.0	5290	1162	3.49	825	325	500	particles flowing into DS
90.0	5410	1170	3.51	825	325	500	minor ds face erosion
92.0	5530	1177	3.53	825	325	500	
94.0	5650	1182	3.55	825	325	500	
96.0	**5770**	**1248**	3.74	**925**	**325**	**600**	
98.0	5890	1278	3.83	925	325	600	eroding particles clear flow
100.0	6010	1311	3.93	925	325	600	minor ds face erosion
102.0	6130	1365	4.10	925	325	600	particles flowing into DS
104.0	6250	1470	4.41	925	325	600	eroding particles clear flow
106.0	6370	1584	4.75	925	325	600	EOT

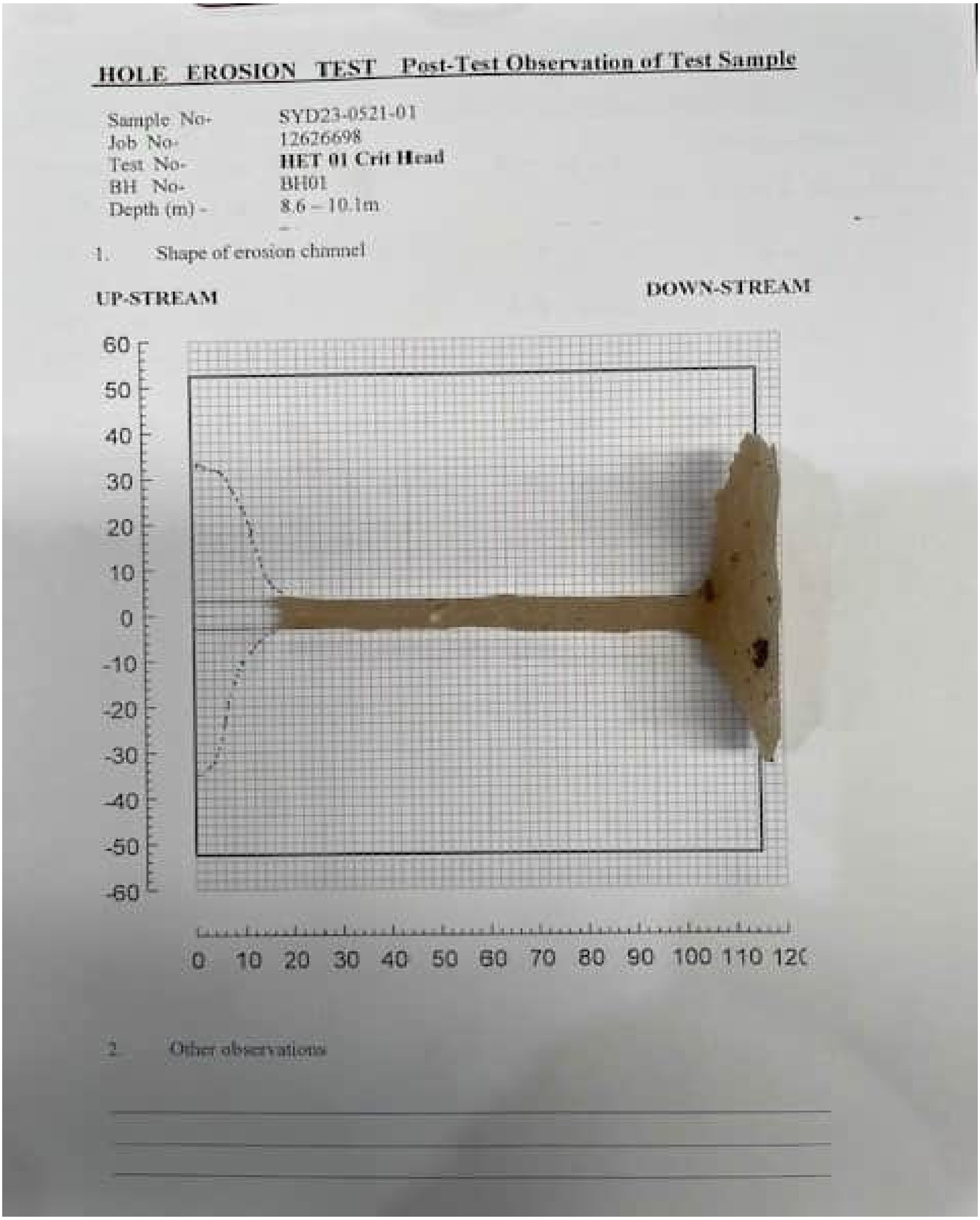
Flow Rate and Pressure Differential v Time
HET01, BH01 8.6-10.1m
Dannevirke Dam, NZ



Client: Tonkin Taylor
Project: Dannevirke Dam
Location: Dannevirke, NZ

BH No: BH01
Depth (m): 8.6-10.1
HET01

Job No: 12626698
GHD Sample No: SYD23-0521-01
Client Sample ID: BH01



HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No- SYD23-0521-01
Job No- 12626698
Test No- **HET 01 Crit Head**
BH No- BH01
Depth (m) - 8.6 – 10.1m

1. Shape of erosion channel

UP-STREAM

DOWN-STREAM

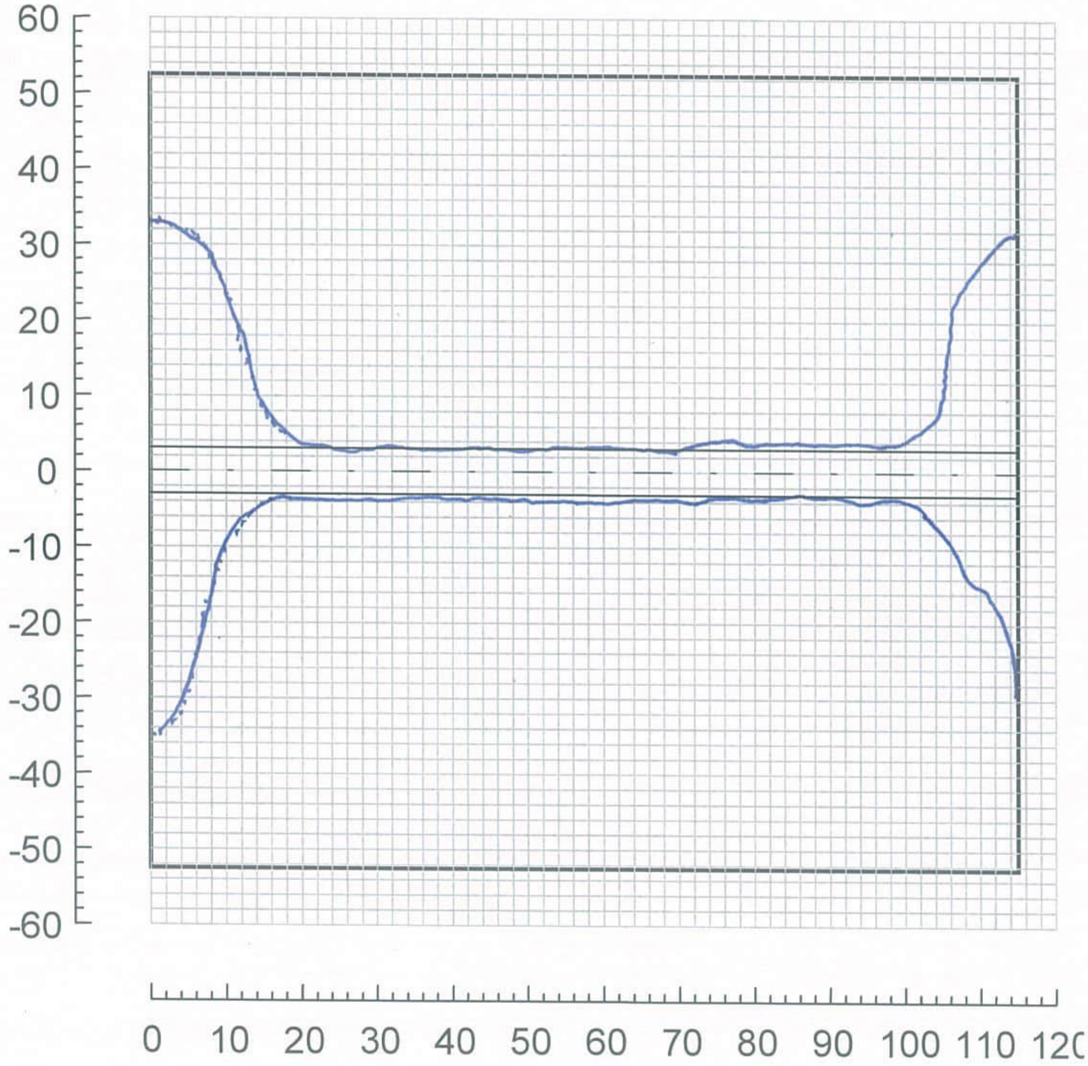
2. Other observations

HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No-	SYD23-0521-01
Job No-	12626698
Test No-	**HET 01 Crit Head**
BH No-	BH01
Depth (m) -	8.6 – 10.1m

1. Shape of erosion channel

UP-STREAM **DOWN-STREAM**



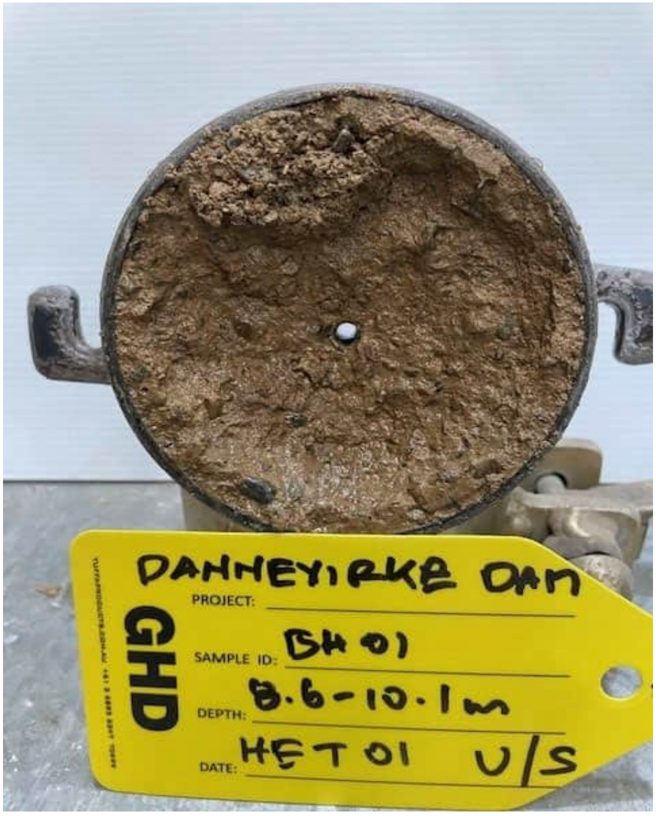
2. Other observations

______

______

______

Client:	Tonkin Taylor	BH No:	BH01	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	8.6-10.1	GHD Sample No:	SYD23-0521-01
Location:	Dannevirke, NZ		HET01	Client Sample ID:	BH01



Client:	Tonkin Taylor	BH No:	BH01	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	8.6-10.1	GHD Sample No:	SYD23-0521-01
Location:	Dannevirke, NZ		HET01	Client Sample ID:	BH01

HOLE EROSION TEST (HET)

Test Procedure : 1. Wan,C.F & Fell,R (2002)UNCIV Report No R-412 ISBN:85841 379 5 2.Wan,C.F & Fell,R (2004a) ASCE Vol 130 No 4 pp 373-380
3.Wan,C.F & Fell,R (2004b) ASTM Vol 27, No 3 pp 295-303

Client:	**Tonkin Taylor**	**BH No:**	**BH01**	**Job No:**	**12626698**
Project:	**Dannevirke Dam**	**Depth (m):**	**8.6-10.1**	**GHD Lab Sample No:**	**SYD23-0521-01**
Location:	**Dannevirke, NZ**	**HET Type :**	**Standard**	**Client Sample ID:**	**BH01**

Report No SYD2302889.2

	Test No. :	**HET02**
	Test Date:	**15-Nov-23**
	Test by:	**JV**
Bulk sample	SMDD (t/m³) :	n/a
Bulk Sample	OMC (%):	n/a
	Targeted Density Ratio (%):	98.0%
	Targeted Water Content (%):	OMC
	Conditioning moisture content %:	**14.0%**
	Moisture Content from test :	14.0%
	Upstream water head at start of test(mm):	500
	Fluid for conditioning soil :	Syd Tap Water
	Eroding fluid :	Syd Tap Water
	Eroding fluid mean temperature (°C) :	21.0
	Diameter of hole (mm):	6.0

% Moist & Density Before	
Soil + mould	4410
wt mould	2273
vol mould	1003.0
wt wet soil	2137
wet density	2.131
cont No	14A
wet + cont (g)	313.6
dry + cont (g)	294.1
cont (g)	154.6
moisture %	14.0%
DryDensity(t/m³):	**1.869**
Density ratio :	**98.0%**
Moisture ratio:	**100.0%**

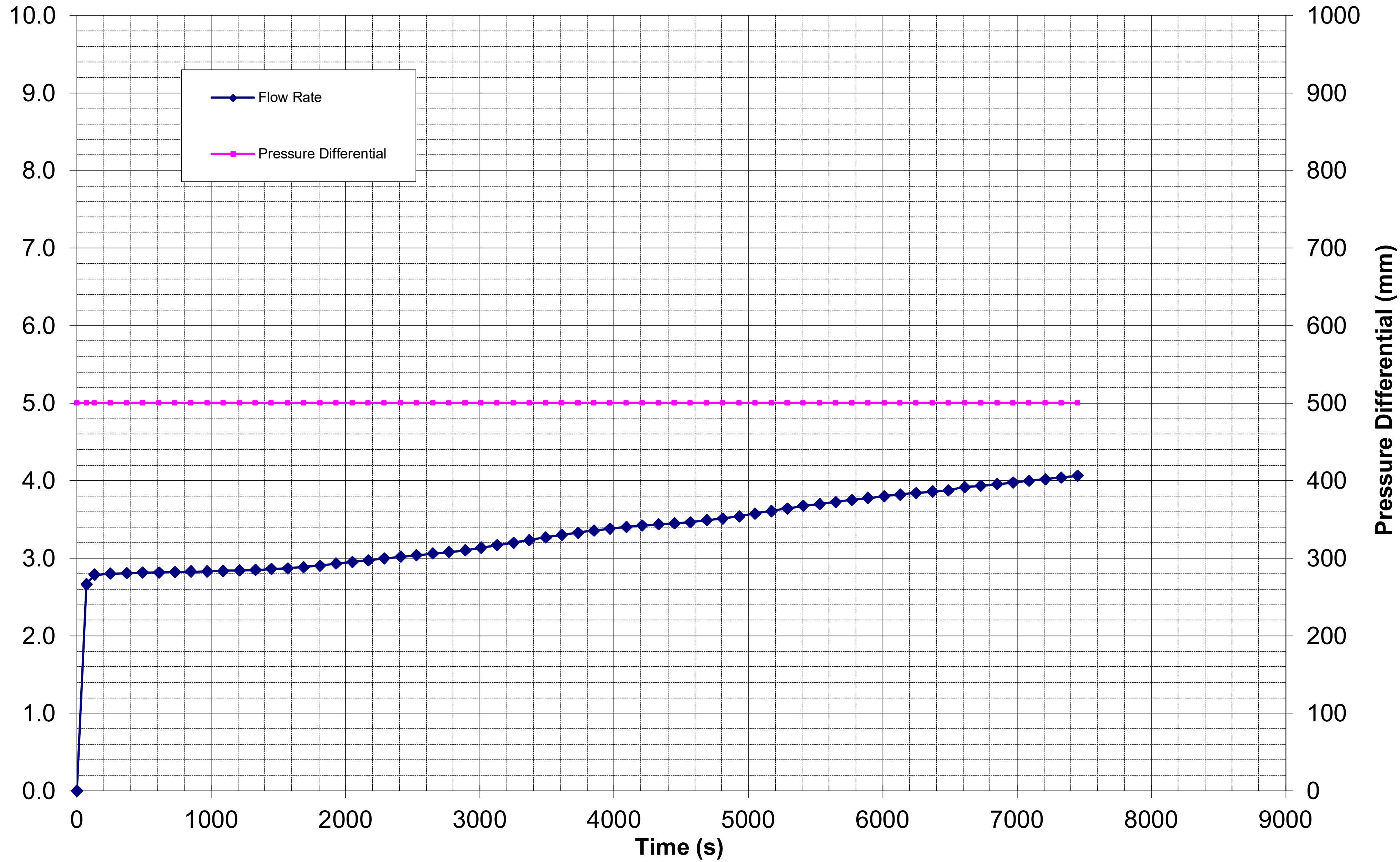
Initial remoulded Density	
Target Wet Dens.(t/m³)	
Total Wt Req'd (g)	
Wt per Layer (g)	
Ø (cm) =	10.53
Ht (cm) =	11.52
HET Mould	*A*
MDD	1.91
OMC	14.0%
POST MC	HET02
cont No	D10
wet + cont (g)	607.1
dry + cont (g)	538.3
cont (g)	73.1
moisture %	14.8%

Sample Description: Gravelly SILT with sand; brown

(1) Time start (mins)	(2) Time (1) * 60 (s)	Outflow mls	(3) Flow rate (L/min)	(4) Head in U/S Tube (mm)	(5) Head in D/S Tube (mm)	(6) Head Diff. (4) - (5) (mm)	(7) Remarks
0.0	0	0	0.00	825	325	500	
1.0	70	888	2.66	825	325	500	eroding particles clear flow
2.0	130	928	2.78	825	325	500	particles flowing into DS
4.0	250	933	2.80	825	325	500	minor ds face erosion
6.0	370	935	2.81	825	325	500	
8.0	490	937	2.81	825	325	500	
10.0	610	939	2.82	825	325	500	
12.0	730	941	2.82	825	325	500	
14.0	850	942	2.83	825	325	500	
16.0	970	943	2.83	825	325	500	
18.0	1090	945	2.84	825	325	500	
20.0	1210	947	2.84	825	325	500	
22.0	1330	949	2.85	825	325	500	
24.0	1450	953	2.86	825	325	500	
26.0	1570	957	2.87	825	325	500	eroding particles clear flow
28.0	1690	962	2.89	825	325	500	particles flowing into DS
30.0	1810	968	2.90	825	325	500	minor ds face erosion
32.0	1930	976	2.93	825	325	500	
34.0	2050	984	2.95	825	325	500	
36.0	2170	991	2.97	825	325	500	
38.0	2290	999	3.00	825	325	500	
40.0	2410	1006	3.02	825	325	500	
42.0	2530	1013	3.04	825	325	500	
44.0	2650	1020	3.06	825	325	500	
46.0	2770	1026	3.08	825	325	500	
48.0	2890	1033	3.10	825	325	500	
50.0	3010	1044	3.13	825	325	500	minor ds face erosion
52.0	3130	1056	3.17	825	325	500	eroding particles clear flow
54.0	3250	1066	3.20	825	325	500	particles flowing into DS
56.0	3370	1078	3.23	825	325	500	
58.0	3490	1089	3.27	825	325	500	

60.0	3610	1100	3.30	825	325	500	eroding particles clear flow
62.0	3730	1110	3.33	825	325	500	particles flowing into DS
64.0	3850	1119	3.36	825	325	500	
66.0	3970	1127	3.38	825	325	500	
68.0	4090	1134	3.40	825	325	500	
70.0	4210	1140	3.42	825	325	500	
72.0	4330	1145	3.44	825	325	500	
74.0	4450	1149	3.45	825	325	500	eroding particles clear flow
76.0	4570	1155	3.47	825	325	500	particles flowing into DS
78.0	4690	1163	3.49	825	325	500	minor ds face erosion
80.0	4810	1171	3.51	825	325	500	
82.0	4930	1180	3.54	825	325	500	
84.0	5050	1191	3.57	825	325	500	
86.0	5170	1203	3.61	825	325	500	
88.0	5290	1214	3.64	825	325	500	
90.0	5410	1224	3.67	825	325	500	
92.0	5530	1233	3.70	825	325	500	
94.0	5650	1242	3.73	825	325	500	
96.0	5770	1251	3.75	825	325	500	eroding particles clear flow
98.0	5890	1259	3.78	825	325	500	particles flowing into DS
100.0	6010	1267	3.80	825	325	500	minor ds face erosion
102.0	6130	1273	3.82	825	325	500	
104.0	6250	1280	3.84	825	325	500	
106.0	6370	1286	3.86	825	325	500	
108.0	6490	1292	3.88	825	325	500	
110.0	6610	1305	3.92	825	325	500	
112.0	6730	1311	3.93	825	325	500	
114.0	6850	1318	3.95	825	325	500	
116.0	6970	1325	3.98	825	325	500	
118.0	7090	1333	4.00	825	325	500	
120.0	7210	1340	4.02	825	325	500	
122.0	7330	1346	4.04	825	325	500	
124.0	7450	1354	4.06	825	325	500	EOT

Flow Rate and Pressure Differential v Time
HET02, BH01 8.6-10.1m
Dannevirke Dam, NZ

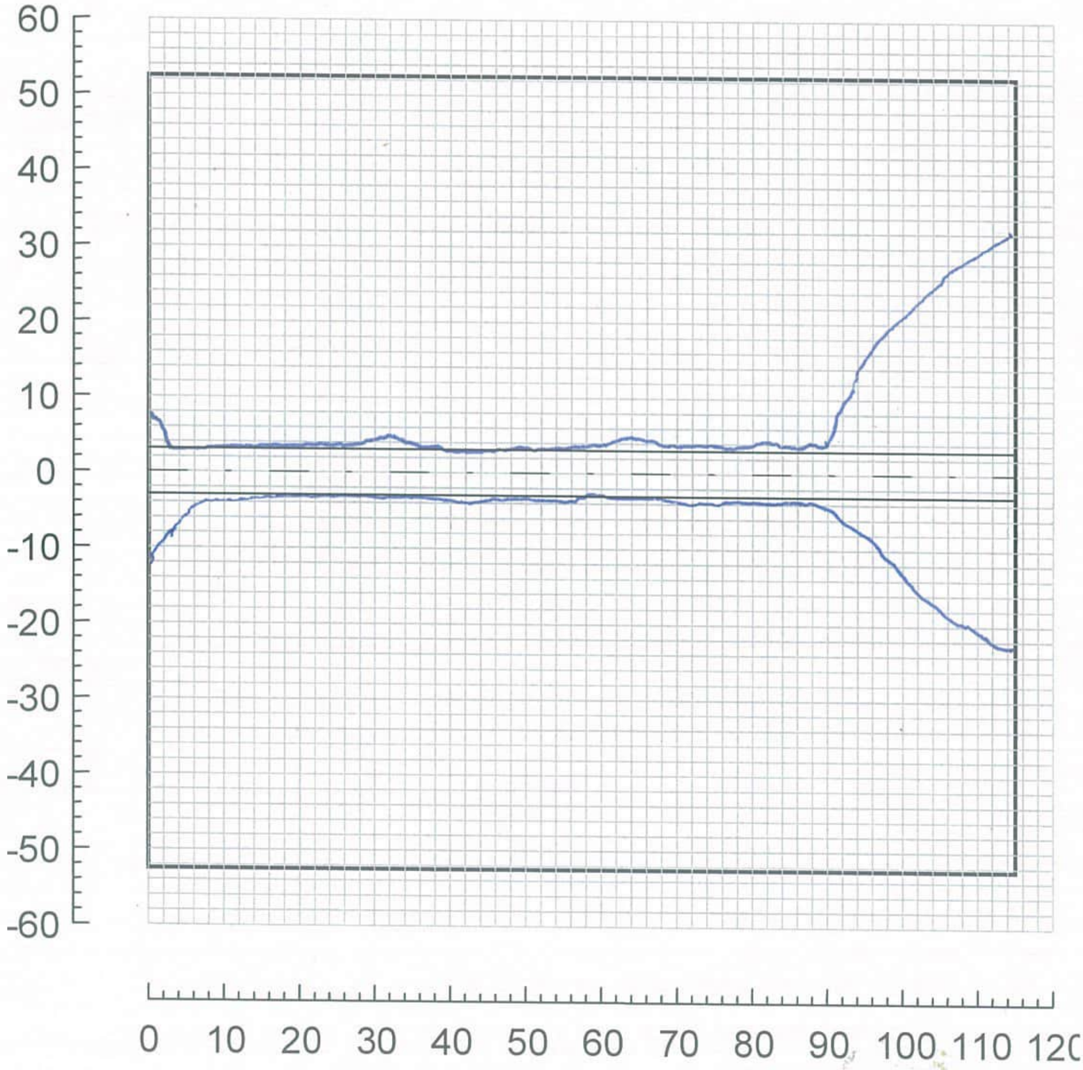


HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No-	SYD23-0521-01
Job No-	12626698
Test No-	**HET 02 Std**
BH No-	BH01
Depth (m) -	8.6 – 10.1m

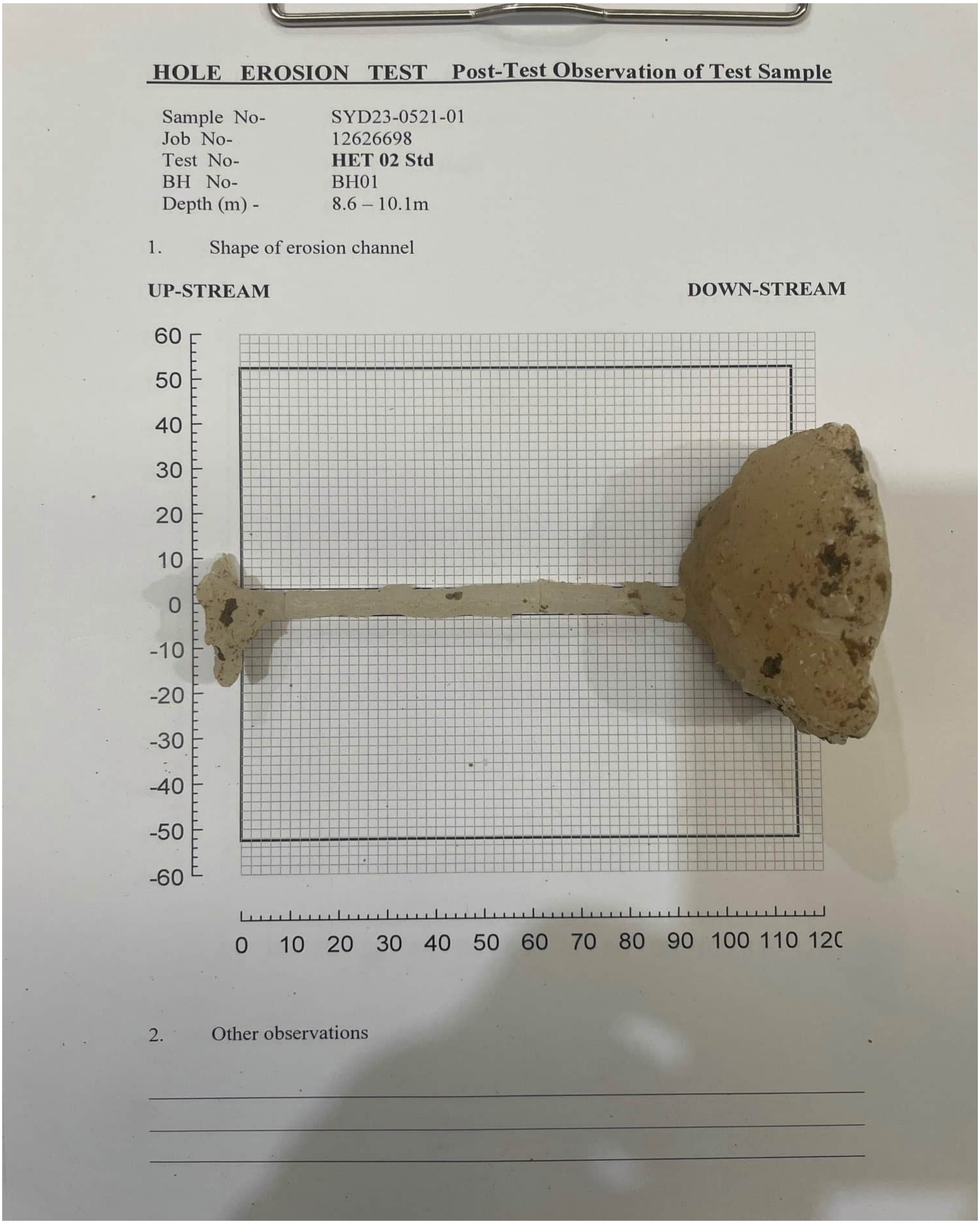
1. Shape of erosion channel

UP-STREAM **DOWN-STREAM**



2. Other observations

Client:	Tonkin Taylor	BH No:	BH01	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	8.6-10.1	GHD Sample No:	SYD23-0521-01
Location:	Dannevirke, NZ		HET02	Client Sample ID:	BH01



HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No- SYD23-0521-01
Job No- 12626698
Test No- **HET 02 Std**
BH No- BH01
Depth (m) - 8.6 – 10.1m

1. Shape of erosion channel

UP-STREAM

DOWN-STREAM

2. Other observations

Client:	Tonkin Taylor	BH No:	BH01	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	8.6-10.1	GHD Sample No:	SYD23-0521-01
Location:	Dannevirke, NZ		HET02	Client Sample ID:	BH01

Client:	Tonkin Taylor	BH No:	BH01	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	8.6-10.1	GHD Sample No:	SYD23-0521-01
Location:	Dannevirke, NZ		HET02	Client Sample ID:	BH01

Sydney Laboratory
Unit 5/43 Herbert St
Artarmon NSW 2064
email: artarmon@ghd.com.au
web: www.ghd.com.au/ghdgeotechnics
Tel: (02) 9462 4860
Fax:(02) 9462 4710

Maximum Dry Density Report

Report No: MDD:SYD23-0521-02
Issue No: 1

Client: Tonkin & Taylor Limited
161 Victoria St
Wellington NZ 6011

Project: 12626698

Accredited for compliance with ISO / IEC 17025 - Testing

NATA Accreditation No: 679

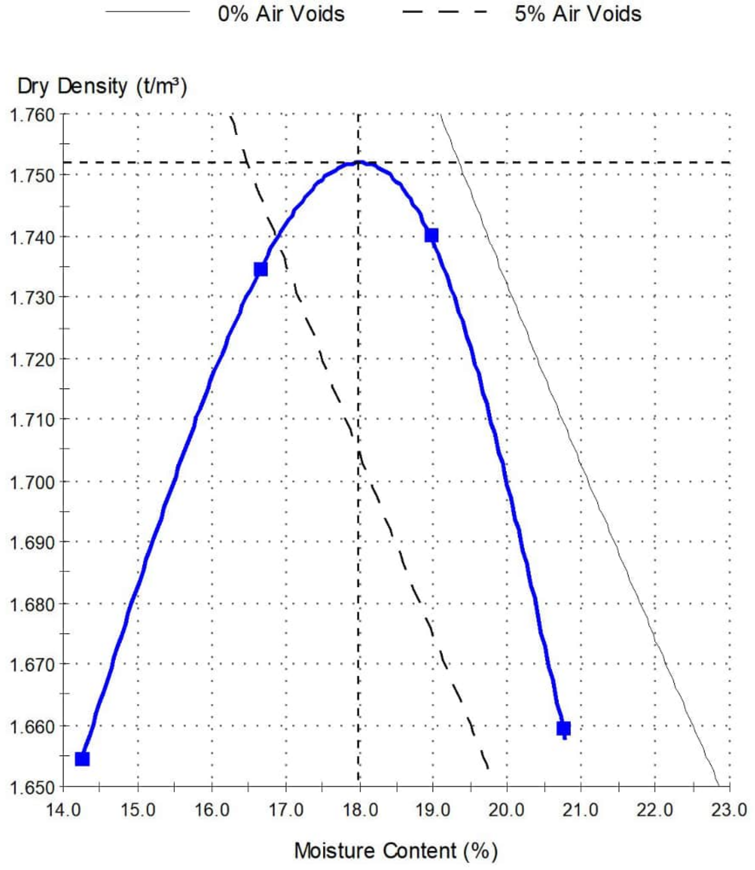
Approved Signatory: D.P Brooke

Date of Issue: 16/11/2023

Sample Details

Location:	Dannevirke Dam NZ		
Sample ID:	SYD23-0521-02	**Date Sampled:**	
Sampling Method:			
Source:		**Material:**	
Specification:		**Location:**	Sandy SILT
Date Tested:	10/11/2023	**Borehole No.:**	TP105
Depth (m):	1.0	**Sample Description:**	Sandy SILT

Dry Density - Moisture Content Relationship



Test Results

AS 1289.5.1.1 - 2017

Standard MDD (t/m³):	**1.75**
Standard OMC (%):	**18.0**
Retained Sieve (mm):	19.0
Oversize Material (%):	0
Curing Time (h):	28
LL Method:	Visual / Tactile

Comments

Zero Air voids line - 2.65 assumed

HOLE EROSION TEST (HET) F9.1.64

Test Procedure : 1. Wan,C.F & Fell,R (2002)UNCIV Report No R-412 ISBN:85841 379 5 2.Wan,C.F & Fell,R (2004a) ASCE Vol 130 No 4 pp 373-380
3.Wan,C.F & Fell,R (2004b) ASTM Vol 27, No 3 pp 295-303

Client:	**Tonkin Taylor**	**BH No:**	**TP105**	**Job No:**	**12626698**	**Report No SYD2302890.1**
Project:	**Dannevirke Dam**	**Depth (m):**	**1.0m**	**GHD Lab Sample No:**	**SYD23-0521-02**	
Location:	**Dannevirke, NZ**	**HET Type :**	**Critical Head**	**Client Sample ID:**	**TP105 1.0m**	

	Test No. :	**HET03**
	Test Date:	**14-Nov-23**
	Test by:	**JV**
Bulk sample	SMDD (t/m³) :	n/a
Bulk Sample	OMC (%):	n/a
	Targeted Density Ratio (%):	98.0%
	Targeted Water Content (%):	OMC
	Conditioning moisture content %:	**18.0%**
	Moisture Content from test :	18.0%
	Upstream water head at start of test(mm):	n/a
	Fluid for conditioning soil :	Syd Tap Water
	Eroding fluid :	Syd Tap Water
	Eroding fluid mean temperature (°C) :	21.0
	Diameter of hole (mm):	6.0

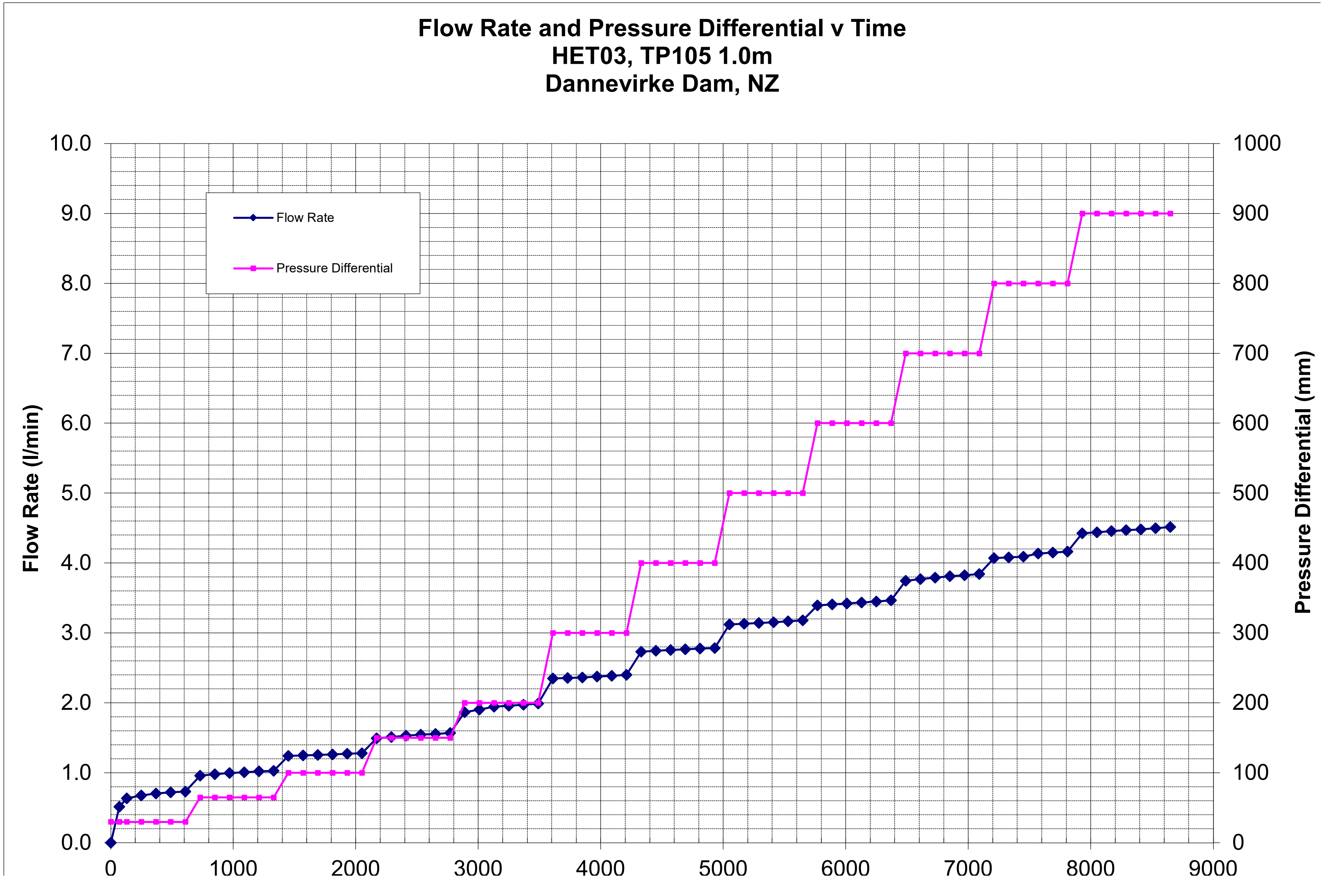
% Moist & Density Before	
Soil + mould	4311
wt mould	2272
vol mould	1005.0
wt wet soil	2039
wet density	2.029
cont No	F07
wet + cont (g)	253.1
dry + cont (g)	225.2
cont (g)	70.2
moisture %	18.0%
DryDensity(t/m³):	**1.719**
Density ratio :	**98.0%**
Moisture ratio:	**100.0%**

Initial remoulded Density	
Target Wet Dens.(t/m³)	
Total Wt Req'd (g)	
Wt per Layer (g)	
Ø (cm) =	10.52
Ht (cm) =	11.56
HET Mould	*B*
MDD	1.75
OMC	18.0%
POST MC	HET03
cont No	fix
wet + cont (g)	499.1
dry + cont (g)	431
cont (g)	74.1
moisture %	19.1%

Sample Description: Sandy SILT ; brown

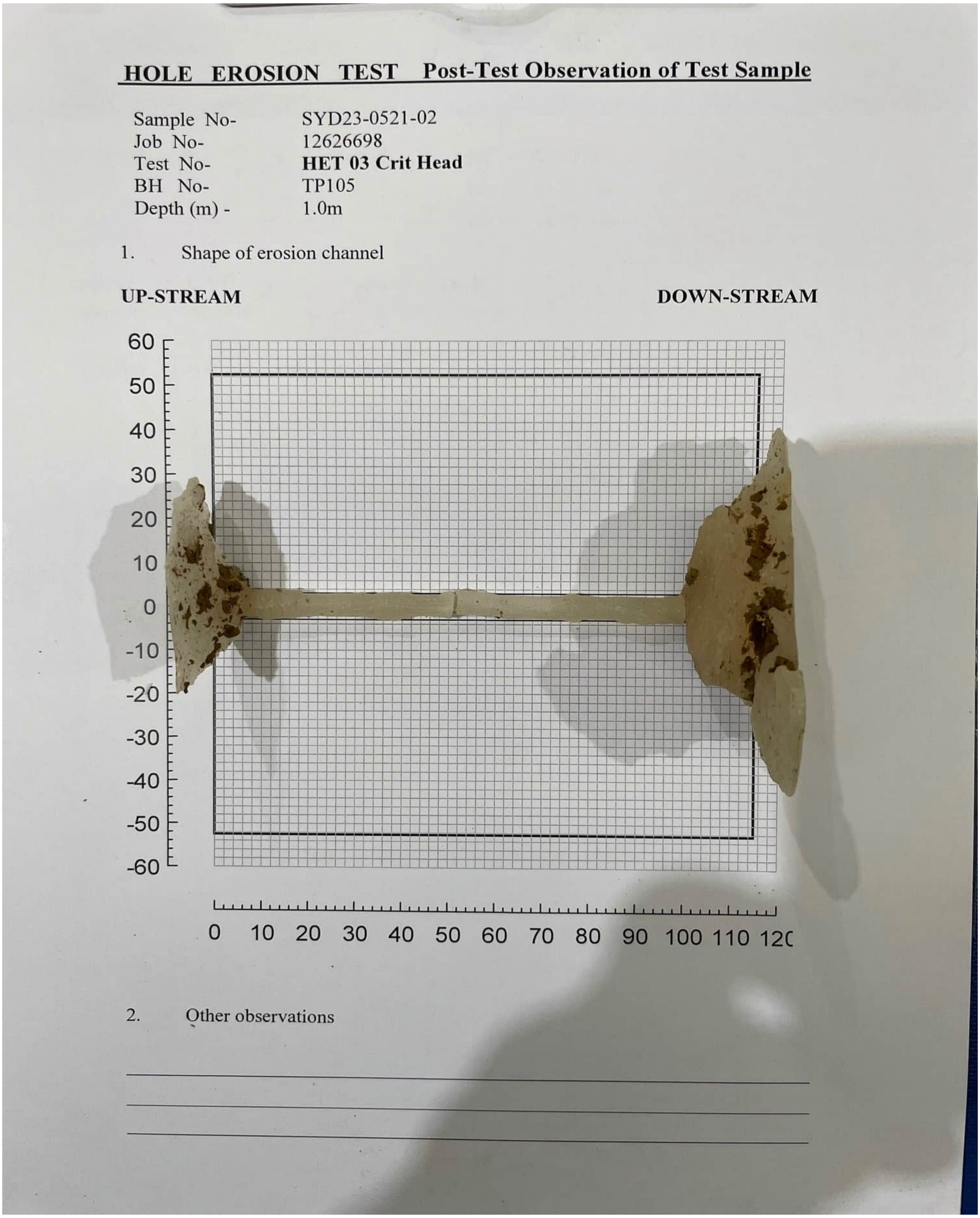
(1) Time start (mins)	(2) Time (1) * 60 (s)	Outflow mls	(3) Flow rate (L/min)	(4) Head in U/S Tube (mm)	(5) Head in D/S Tube (mm)	(6) Head Diff. (4) - (5) (mm)	(7) Remarks
0.0	0	0	0.00	355	325	30	
1.0	70	171	0.51	355	325	30	eroding particles clear flow
2.0	130	212	0.64	355	325	30	particles flowing into DS
4.0	250	225	0.68	355	325	30	
6.0	370	235	0.71	355	325	30	
8.0	490	240	0.72	355	325	30	
10.0	610	244	0.73	355	325	30	
12.0	**730**	**320**	**0.96**	**390**	**325**	**65**	
14.0	850	327	0.98	390	325	65	
16.0	970	332	1.00	390	325	65	eroding particles clear flow
18.0	1090	336	1.01	390	325	65	particles flowing into DS
20.0	1210	340	1.02	390	325	65	
22.0	1330	343	1.03	390	325	65	
24.0	**1450**	**414**	**1.24**	**425**	**325**	**100**	
26.0	1570	416	1.25	425	325	100	eroding particles clear flow
28.0	1690	418	1.25	425	325	100	particles flowing into DS
30.0	1810	421	1.26	425	325	100	minor ds face erosion
32.0	1930	424	1.27	424	324	100	
34.0	2050	427	1.28	425	325	100	
36.0	**2170**	**498**	**1.49**	**475**	**325**	**150**	
38.0	2290	504	1.51	475	325	150	
40.0	2410	510	1.53	475	325	150	eroding particles clear flow
42.0	2530	515	1.55	475	325	150	particles flowing into DS
44.0	2650	519	1.56	475	325	150	
46.0	2770	523	1.57	475	325	150	
48.0	**2890**	**622**	**1.87**	**525**	**325**	**200**	
50.0	3010	635	1.91	525	325	200	particles flowing into DS
52.0	3130	648	1.94	525	325	200	eroding particles clear flow
54.0	3250	653	1.96	525	325	200	
56.0	3370	658	1.97	525	325	200	
58.0	3490	663	1.99	525	325	200	

60.0	**3610**	**783**	**2.35**	**625**	**325**	**300**	
62.0	3730	785	2.36	625	325	300	particles flowing into DS
64.0	3850	788	2.36	625	325	300	eroding particles clear flow
66.0	3970	792	2.38	625	325	300	
68.0	4090	796	2.39	625	325	300	
70.0	4210	800	2.40	625	325	300	
72.0	**4330**	**911**	**2.73**	**725**	**325**	**400**	
74.0	4450	915	2.75	725	325	400	eroding particles clear flow
76.0	4570	919	2.76	725	325	400	particles flowing into DS
78.0	4690	922	2.77	725	325	400	
80.0	4810	925	2.78	725	325	400	
82.0	4930	928	2.78	725	325	400	
84.0	**5050**	**1041**	3.12	**825**	**325**	**500**	
86.0	5170	1044	3.13	825	325	500	eroding particles clear flow
88.0	5290	1047	3.14	825	325	500	particles flowing into DS
90.0	5410	1051	3.15	825	325	500	
92.0	5530	1055	3.17	825	325	500	
94.0	5650	1060	3.18	825	325	500	
96.0	**5770**	**1131**	3.39	**925**	**325**	**600**	
98.0	5890	1136	3.41	925	325	600	
100.0	6010	1141	3.42	925	325	600	eroding particles clear flow
102.0	6130	1145	3.44	925	325	600	particles flowing into DS
104.0	6250	1150	3.45	925	325	600	
106.0	6370	1155	3.47	925	325	600	
108.0	**6490**	**1249**	**3.75**	**1025**	**325**	**700**	
110.0	6610	1257	3.77	1025	325	700	particles flowing into DS
112.0	6730	1264	3.79	1025	325	700	eroding particles clear flow
114.0	6850	1270	3.81	1025	325	700	
116.0	6970	1275	3.83	1025	325	700	
118.0	7090	1281	3.84	1025	325	700	
120.0	**7210**	**1357**	**4.07**	**1125**	**325**	**800**	
122.0	7330	1360	4.08	1125	325	800	eroding particles clear flow
124.0	7450	1364	4.09	1125	325	800	particles flowing into DS
126.0	7570	1378	4.13	1125	325	800	
128.0	7690	1383	4.15	1125	325	800	
130.0	7810	1388	4.16	1125	325	800	
132.0	**7930**	**1475**	**4.43**	**1225**	**325**	**900**	
134.0	8050	1480	4.44	1225	325	900	particles flowing into DS
136.0	8170	1486	4.46	1225	325	900	eroding particles clear flow
138.0	8290	1490	4.47	1225	325	900	
140.0	8410	1494	4.48	1225	325	900	
142.0	8530	1499	4.50	1225	325	900	
144.0	8650	1505	4.52	1225	325	900	EOT



Flow Rate and Pressure Differential v Time
HET03, TP105 1.0m
Dannevirke Dam, NZ
Flow Rate
Pressure Differential
Flow Rate (l/min)
Pressure Differential (mm)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1000
900
800
700
600
500
400
300
200
100
0
0
1000
2000
3000
4000
5000
6000
7000
8000
9000

Client:	Tonkin Taylor	BH No:	TP105	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	1.0m	GHD Sample No:	SYD23-0521-02
Location:	Dannevirke, NZ		HET03	Client Sample ID:	TP105 1.0m



HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No-	SYD23-0521-02
Job No-	12626698
Test No-	**HET 03 Crit Head**
BH No-	TP105
Depth (m) -	1.0m

1. Shape of erosion channel

UP-STREAM **DOWN-STREAM**

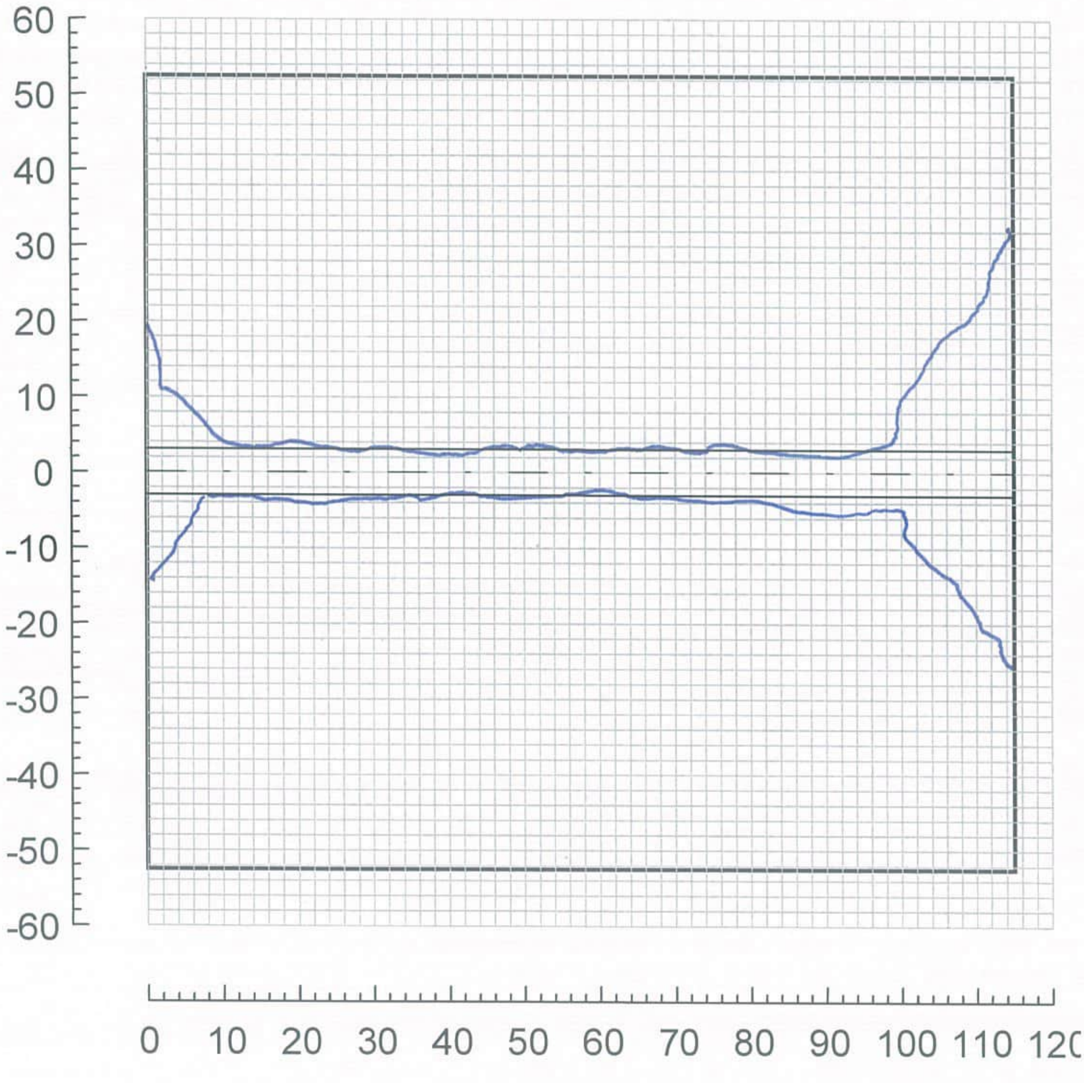
2. Other observations

HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No- SYD23-0521-02
Job No- 12626698
Test No- **HET 03 Crit Head**
BH No- TP105
Depth (m) - 1.0m

1. Shape of erosion channel

UP-STREAM **DOWN-STREAM**



2. Other observations

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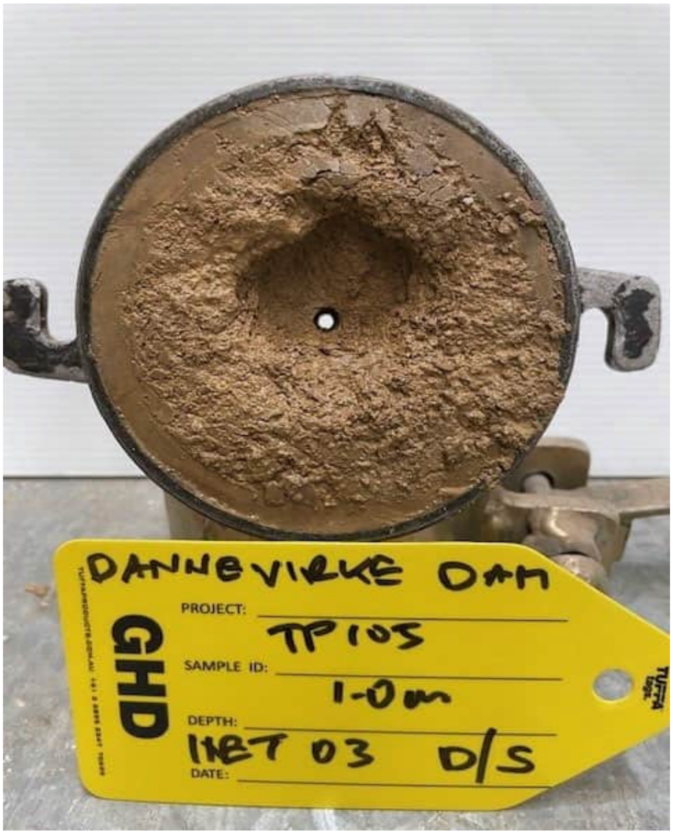
Client:	Tonkin Taylor	BH No:	TP105	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	1.0m	GHD Sample No:	SYD23-0521-02
Location:	Dannevirke, NZ		HET03	Client Sample ID:	TP105 1.0m



Client:	Tonkin Taylor	BH No:	TP105	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	1.0m	GHD Sample No:	SYD23-0521-02
Location:	Dannevirke, NZ		HET03	Client Sample ID:	TP105 1.0m

HOLE EROSION TEST (HET)

F9.1.64

Test Procedure : 1. Wan,C.F & Fell,R (2002)UNCIV Report No R-412 ISBN:85841 379 5 2.Wan,C.F & Fell,R (2004a) ASCE Vol 130 No 4 pp 373-380
3.Wan,C.F & Fell,R (2004b) ASTM Vol 27, No 3 pp 295-303

Client:	**Tonkin Taylor**	**BH No:**	**TP105**	**Job No:**	**12626698**
Project:	**Dannevirke Dam**	**Depth (m):**	**1.0m**	**GHD Lab Sample No:**	**SYD23-0521-02**
Location:	**Dannevirke, NZ**	**HET Type :**	**Critical Head**	**Client Sample ID:**	**TP105 1.0m**

Report No SYD2302890.2

Standard

Test No. :	**HET04**
Test Date:	**14-Nov-23**
Test by:	**JV**
Bulk sample SMDD (t/m³) :	n/a
Bulk Sample OMC (%):	n/a
Targeted Density Ratio (%):	98.0%
Targeted Water Content (%):	OMC
Conditioning moisture content %:	**18.0%**
Moisture Content from test :	18.0%
Upstream water head at start of test(mm):	900
Fluid for conditioning soil :	Syd Tap Water
Eroding fluid :	Syd Tap Water
Eroding fluid mean temperature (°C) :	21.0
Diameter of hole (mm):	6.0

% Moist & Density Before	
Soil + mould	4305
wt mould	2273
vol mould	1003.0
wt wet soil	2032
wet density	2.026
cont No	F07
wet + cont (g)	253.1
dry + cont (g)	225.2
cont (g)	70.2
moisture %	18.0%
DryDensity(t/m³):	**1.717**
Density ratio :	**98.0%**
Moisture ratio:	**100.0%**

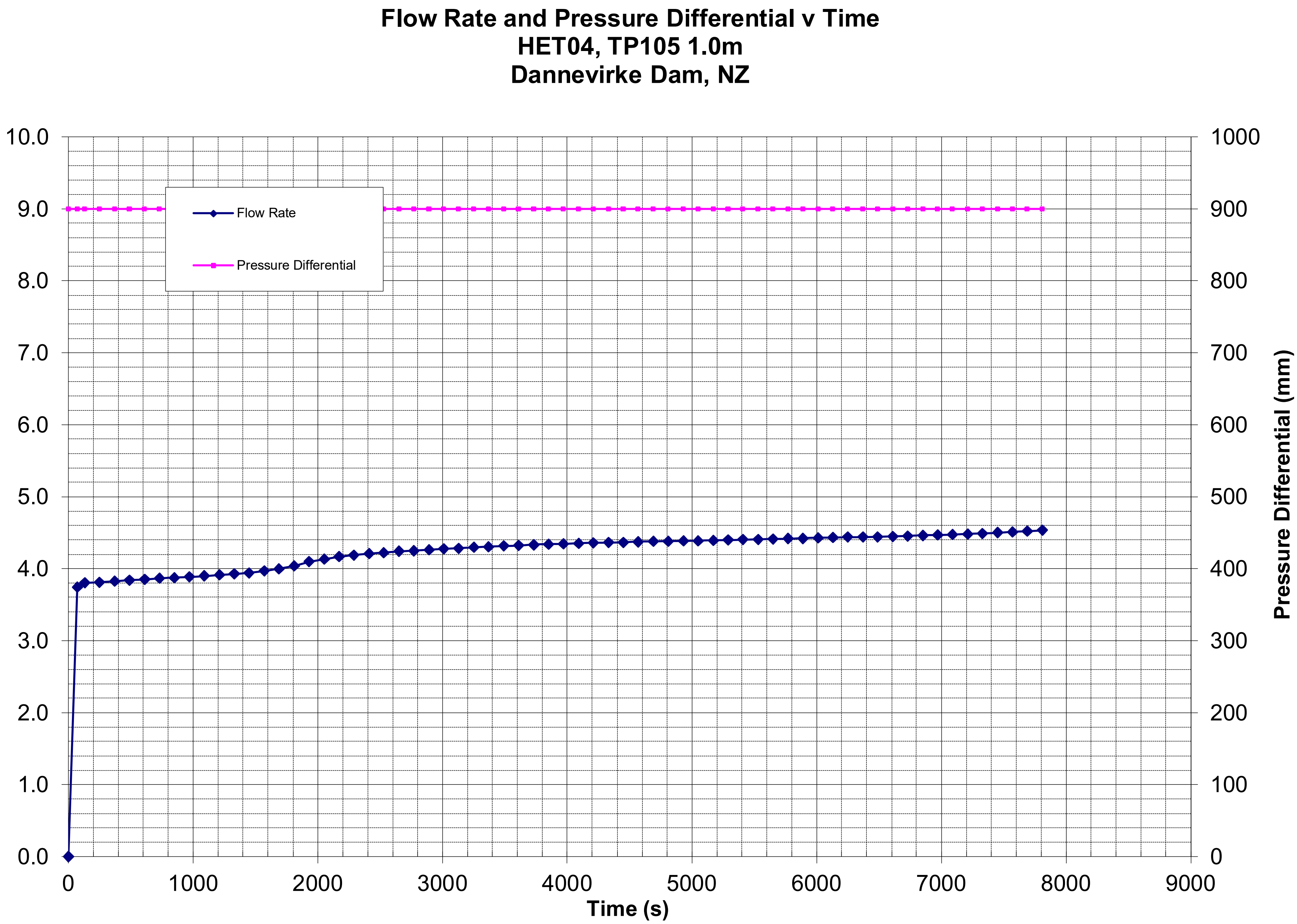
Initial remoulded Density	
Target Wet Dens.(t/m³)	
Total Wt Req'd (g)	
Wt per Layer (g)	
Ø (cm) =	10.53
Ht (cm) =	11.52

HET Mould	*A*
MDD	1.75
OMC	18.0%
POST MC	HET04
cont No	X62
wet + cont (g)	624.9
dry + cont (g)	540.1
cont (g)	85.5
moisture %	18.7%

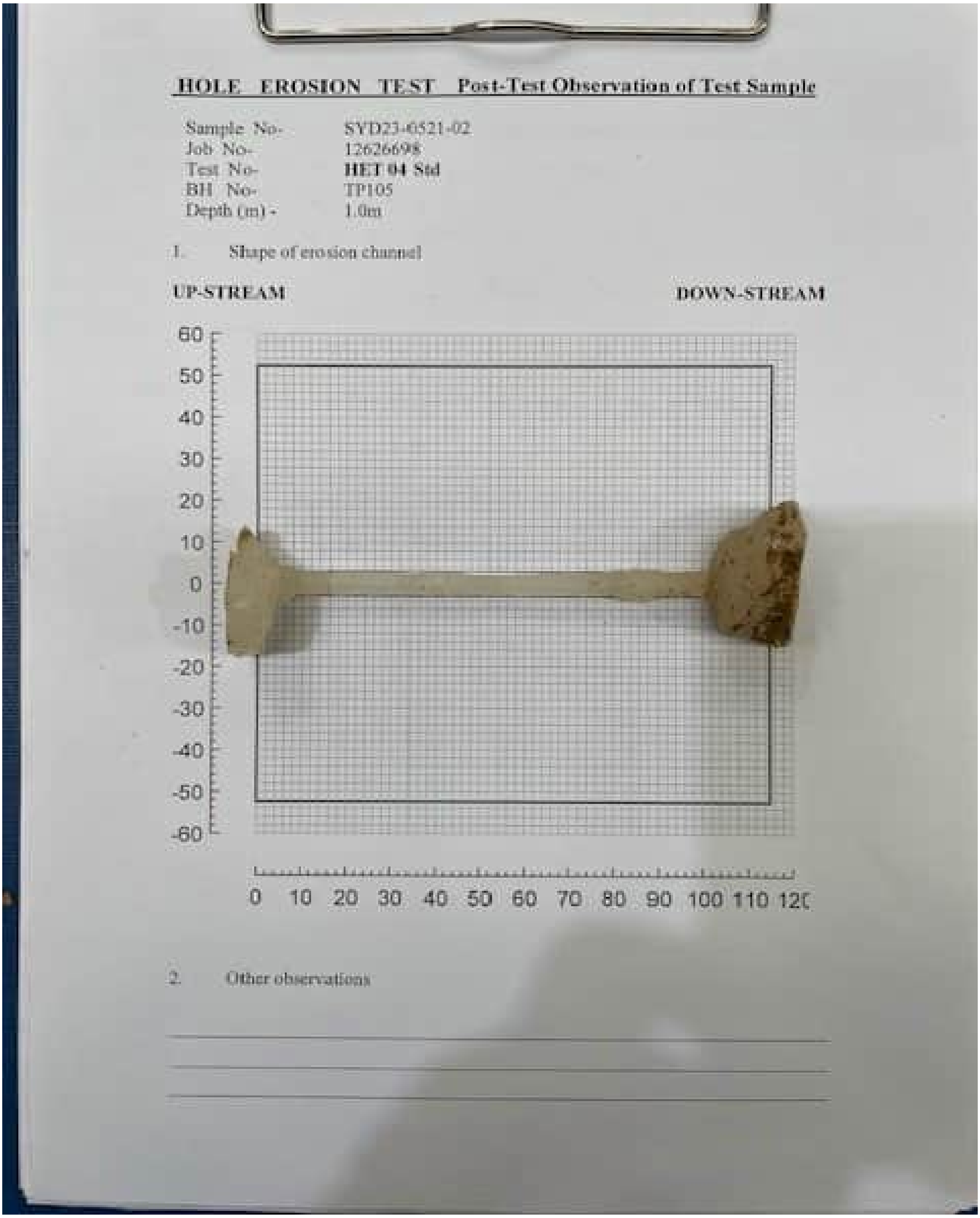
Sample Description: Sandy SILT ; brown

(1) Time start (mins)	(2) Time (1) * 60 (s)	Outflow mls	(3) Flow rate (L/min)	(4) Head in U/S Tube (mm)	(5) Head in D/S Tube (mm)	(6) Head Diff. (4) - (5) (mm)	(7) Remarks
0.0	0	0	0.00	1225	325	900	
1.0	70	1247	3.74	1225	325	900	eroding particles clear flow
2.0	130	1268	3.80	1225	325	900	particles flowing into DS
4.0	250	1271	3.81	1225	325	900	
6.0	370	1274	3.82	1225	325	900	
8.0	490	1280	3.84	1225	325	900	
10.0	610	1284	3.85	1225	325	900	
12.0	730	1288	3.86	1225	325	900	
14.0	850	1292	3.88	1225	325	900	particles flowing into DS
16.0	970	1295	3.89	1225	325	900	eroding particles clear flow
18.0	1090	1300	3.90	1225	325	900	
20.0	1210	1304	3.91	1225	325	900	
22.0	1330	1309	3.93	1225	325	900	
24.0	1450	1314	3.94	1225	325	900	
26.0	1570	1322	3.97	1225	325	900	
28.0	1690	1333	4.00	1225	325	900	
30.0	1810	1345	4.04	1225	325	900	
32.0	1930	1366	4.10	1225	325	900	
34.0	2050	1377	4.13	1225	325	900	
36.0	2170	1388	4.16	1225	325	900	
38.0	2290	1396	4.19	1225	325	900	
40.0	2410	1402	4.21	1225	325	900	particles flowing into DS
42.0	2530	1408	4.22	1225	325	900	eroding particles clear flow
44.0	2650	1413	4.24	1225	325	900	
46.0	2770	1417	4.25	1225	325	900	
48.0	2890	1421	4.26	1225	325	900	
50.0	3010	1425	4.28	1225	325	900	
52.0	3130	1428	4.28	1225	325	900	
54.0	3250	1432	4.30	1225	325	900	
56.0	3370	1435	4.31	1225	325	900	
58.0	3490	1438	4.31	1225	325	900	

60.0	3610	1441	4.32	1225	325	900	eroding particles clear flow
62.0	3730	1444	4.33	1225	325	900	particles flowing into DS
64.0	3850	1446	4.34	1225	325	900	
66.0	3970	1448	4.34	1225	325	900	
68.0	4090	1451	4.35	1225	325	900	
70.0	4210	1453	4.36	1225	325	900	
72.0	4330	1455	4.37	1225	325	900	
74.0	4450	1456	4.37	1225	325	900	
76.0	4570	1458	4.37	1225	325	900	
78.0	4690	1459	4.38	1225	325	900	
80.0	4810	1460	4.38	1225	325	900	
82.0	4930	1461	4.38	1225	325	900	
84.0	5050	1463	4.39	1225	325	900	particles flowing into DS
86.0	5170	1465	4.40	1225	325	900	eroding particles clear flow
88.0	5290	1466	4.40	1225	325	900	
90.0	5410	1468	4.40	1225	325	900	
92.0	5530	1469	4.41	1225	325	900	
94.0	5650	1471	4.41	1225	325	900	
96.0	5770	1473	4.42	1225	325	900	
98.0	5890	1474	4.42	1225	325	900	
100.0	6010	1476	4.43	1225	325	900	
102.0	6130	1477	4.43	1225	325	900	
104.0	6250	1479	4.44	1225	325	900	
106.0	6370	1480	4.44	1225	325	900	
108.0	6490	1481	4.44	1225	325	900	eroding particles clear flow
110.0	6610	1483	4.45	1225	325	900	particles flowing into DS
112.0	6730	1485	4.46	1225	325	900	
114.0	6850	1487	4.46	1225	325	900	
116.0	6970	1489	4.47	1225	325	900	
118.0	7090	1491	4.47	1225	325	900	
120.0	7210	1494	4.48	1225	325	900	
122.0	7330	1497	4.49	1225	325	900	
124.0	7450	1500	4.50	1225	325	900	
126.0	7570	1503	4.51	1225	325	900	
128.0	7690	1507	4.52	1225	325	900	
130.0	7810	1511	4.53	1225	325	900	EOT


Flow Rate and Pressure Differential v Time
HET04, TP105 1.0m
Dannevirke Dam, NZ
Flow Rate
Pressure Differential
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1000
900
800
700
600
500
400
300
200
100
0
Pressure Differential (mm)
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
Time (s)

Client:	Tonkin Taylor	BH No:	TP105	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	1.0m	GHD Sample No:	SYD23-0521-02
Location:	Dannevirke, NZ		HET04	Client Sample ID:	TP105 1.0m



HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No- SYD23-0521-02
Job No- 12626698
Test No- **HET 04 Std**
BH No- TP105
Depth (m) - 1.0m

1. Shape of erosion channel

UP-STREAM DOWN-STREAM

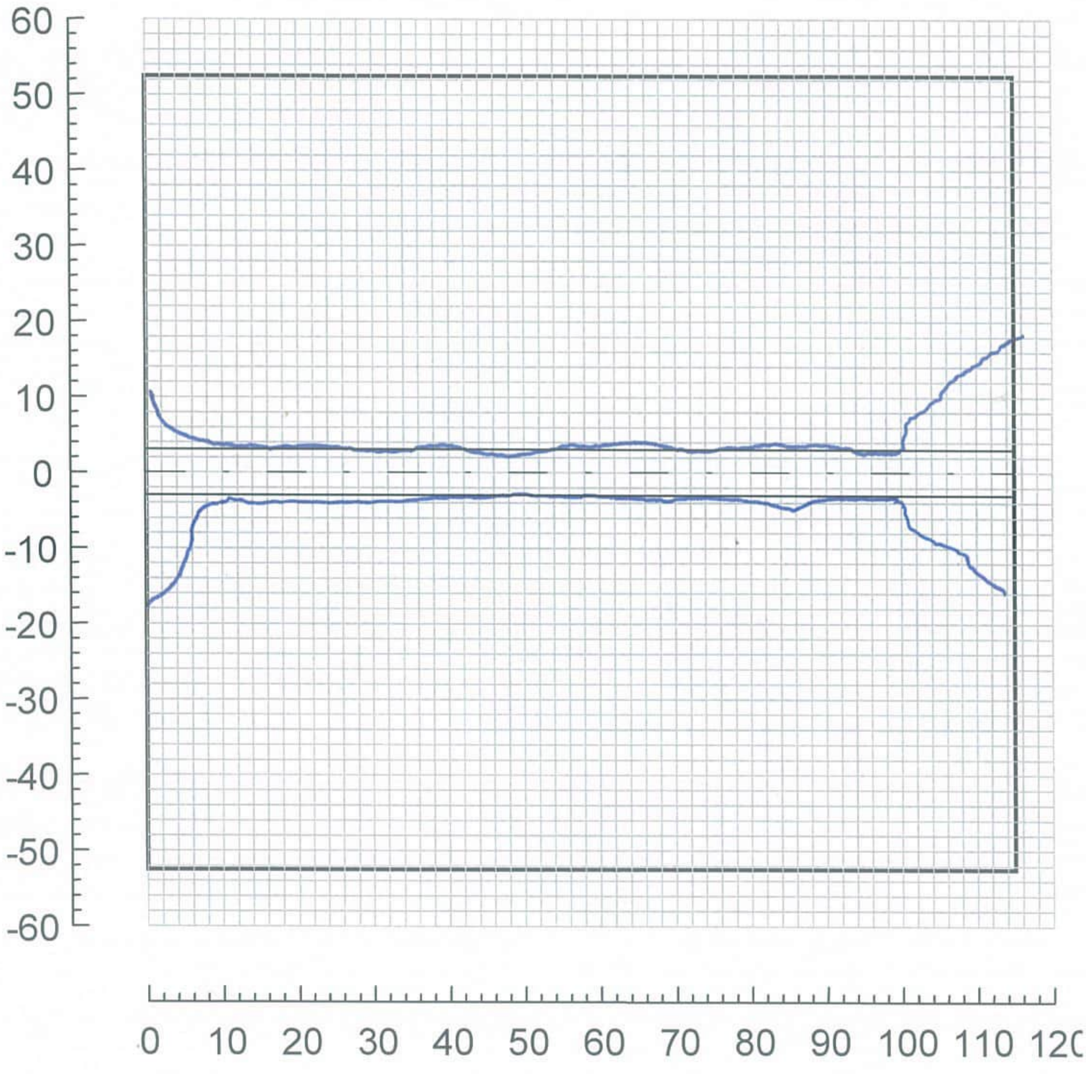
2. Other observations

HOLE EROSION TEST Post-Test Observation of Test Sample

Sample No-	SYD23-0521-02
Job No-	12626698
Test No-	**HET 04 Std**
BH No-	TP105
Depth (m) -	1.0m

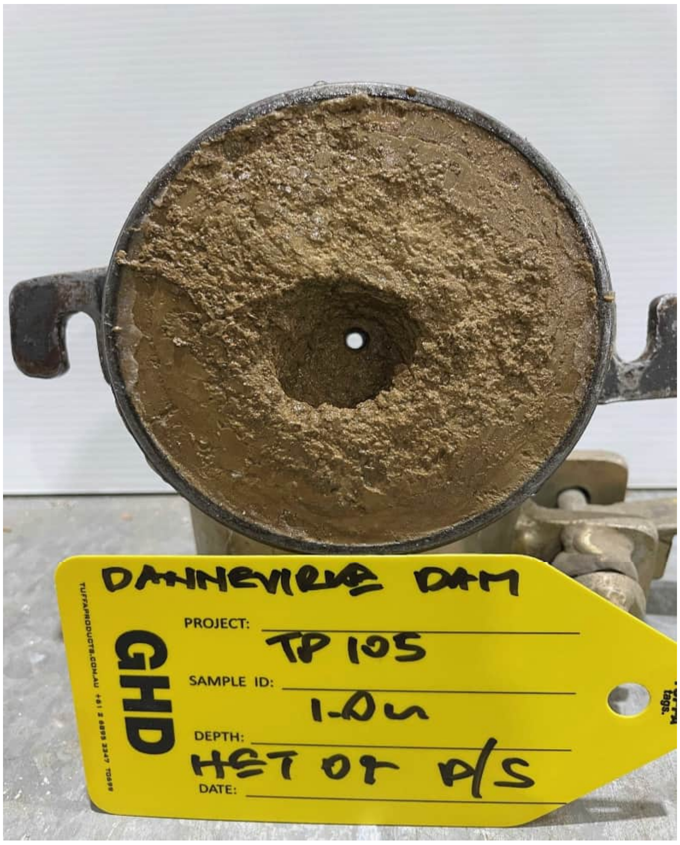
1. Shape of erosion channel

UP-STREAM **DOWN-STREAM**

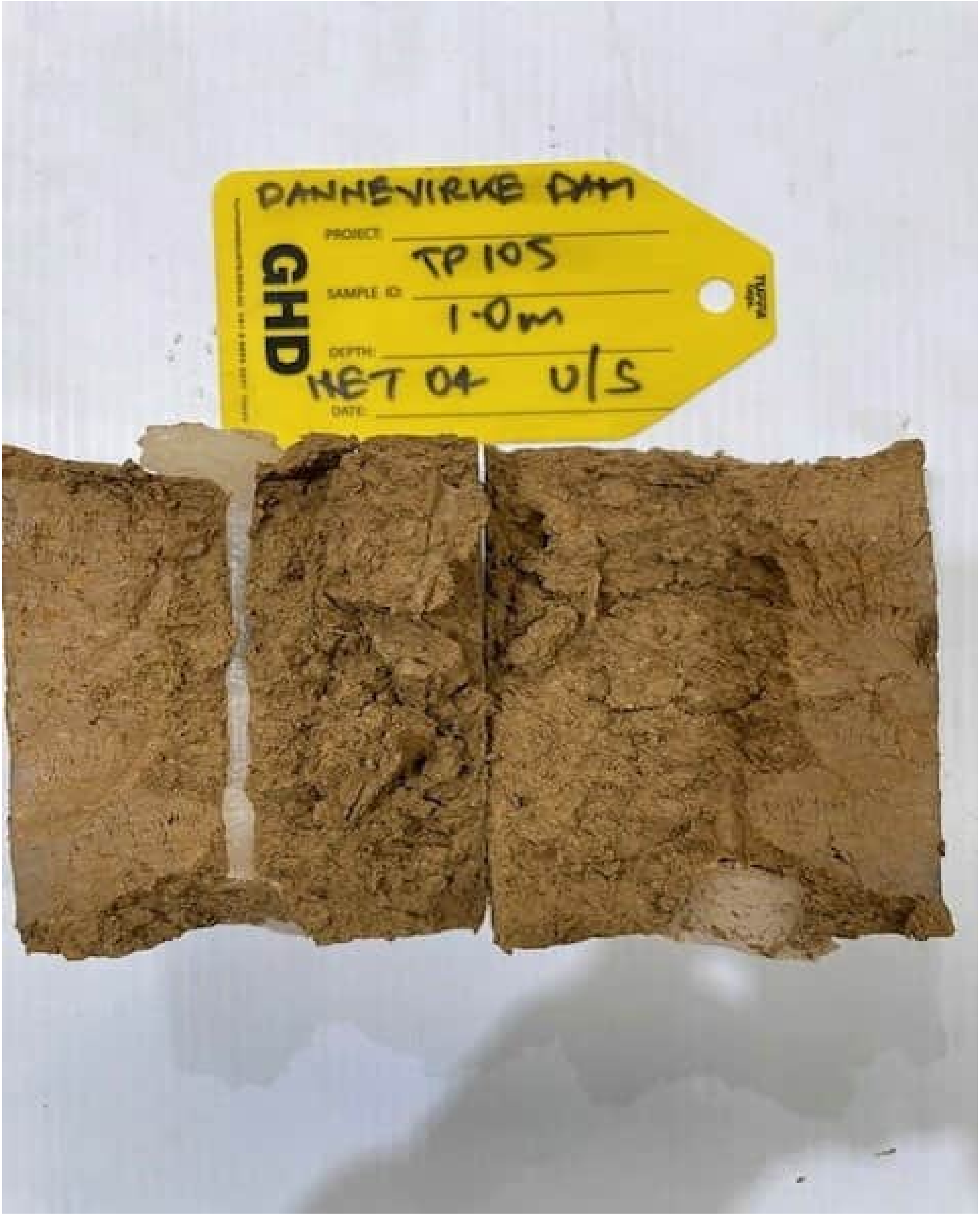


2. Other observations

Client:	Tonkin Taylor	BH No:	TP105	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	1.0m	GHD Sample No:	SYD23-0521-02
Location:	Dannevirke, NZ		HET04	Client Sample ID:	TP105 1.0m



Client:	Tonkin Taylor	BH No:	TP105	Job No:	12626698
Project:	Dannevirke Dam	Depth (m):	1.0m	GHD Sample No:	SYD23-0521-02
Location:	Dannevirke, NZ		HET04	Client Sample ID:	TP105 1.0m



Appendix F Review of ROV inspection reports

Review of ROV inspection reports

Report author and issue date	Inspection date	Time since first leak in July 2021	Time since temp patch repair June 2023	Impound level and subsoil flows	Method of inspection	Depression 2	Depression 3	Depression 4	Depression 5	Depression 6	Depression 7	Other
Bay Dynamics 24/8/2021	Not stated in report	1.8 mths		Not measured, but staff estimate subsoil flows increased from a few L/s to 15-22 L/s following the July 2021 leak.	ROV with multibeam sonar (Oculus) and lighting. Tracking leaks based on sediment movement.	Two depressed areas 10m x 10m and 2m x 2m identified below HDPE liner along western side of reservoir in line with subsoil line 6, the primary trunk line for the subsoil network. Unclear if these two areas correlate with Depressions 2 to 7, as later identified by NPU. An email M Mooney to C Chapman 3/9/2021 noted rippling movement of the HDPE liner, indicating air / water flow under the liner, 10m to either side of the inlet structure and at least 2m below the 5.8m water level on 2/9/2021. The rippling increased in intensity when filling the reservoir and did not stop entirely when valves controlling filling were closed. The ground in this area "felt depressed" when walking on the cover.						During repairs following first leak
Detection Services May 2022	18/5/2022	10.5 mths		Impound level 263-273 mRL, and subsoil flows mostly 3-22 L/s since previous inspection.	ROV with video and lighting, dye tests.	Turbidity too high to navigate and capture meaningful images with method used.						
NPU 1/6/2023	5/5/2023	1yr 10.1mths		Impound level 264.8-272.3 mRL, and subsoil flows mostly 1-28 L/s since previous inspection.	ROV with multibeam sonar (Konsberg MS1000). Tracking leaks based on sediment movement.	"Main leak". Tear 150mm long and 75mm wide, with a further area of HDPE damage 750mm long by 100mm wide. Depression 3.89m^2, 3.47m long, 1.45m wide, 0.5m deep.	Depression 0.04m^2, 0.06m deep. No perforations or evidence of leakage.	"Pinhole leak". 5mm diameter pinhole. Depression 6.22m^2, 0.7m deep.	Depression 2.15m^2, 0.4m deep. No perforations or evidence of leakage.	Depression 1.61m^2, 0.2m deep. No perforations or evidence of leakage.		
NPU 3/7/2023	12 to 19/6/2023	1yr 11.4mths		Impound level 265.5-268.2 mRL, and subsoil flows mostly 23-28 L/s since previous inspection.	Direct diver inspection and repairs, plus ROV with multibeam sonar (Konsberg MS1000), dye tests.	As found: Batter slope collapsed. Depression had enlarged: 1.68 to 2.68m wide, and 4.3 to 4.8m long. 0.9m deep. Repair: Geotextile, then backfill with filter sand, geotextile, then RPVC patch fastened with stainless steel battens, and Selleys All Clear. Dye test indicated no leaks.		As found: Pinhole increased to 20mm diameter. Multiple further pinholes developing, four with penetrations 2mm to 15mm (pinholes 2, 4, 5 and 7), three with flow detected (pinholes 4, 5 and 7), and three with a sharp object beneath (pinholes 1, 3, and 6). Depression: 3.73 to 4.03m wide, and 1.54 to 1.63m long. 0.74m deep. Repair: Geotextile, then backfill with filter sand, geotextile, geogrid, then RPP patch and fastened with butyl tape, Selleys All Clear, and a top layer of sandbags. Dye test indicated no leaks.	4.5 to 5.3m wide, 2.1 to 2.5m long, 0.33m deep. Multiple small depressions trend north towards batter face, about 0.1m deep. Multiple small depressions trend towards east batter face, 0.05 to 0.1m deep.	2.78m wide, 1.48 to 1.55m long, 0.24m deep.		Temporary patch repairs
NPU 13/7/2023	6/7/2023*	2yrs 0.1mths	0.6 mths	Impound level just below 267 mRL, and subsoil flows mostly 4.5-7 L/s since	ROV with multibeam sonar (Konsberg MS1000).	Repair intact. No leaks indicated by test. 0.8m wide by 0.15m deep slump		Repair intact. No leaks indicated by test. Southeast corner has a depression running southeast approximately 1.0m by 1.76m,	Northern side of depression has collapsed down 250mm.	No change.		Grid search between Depression 4 and inlet

Report author and issue date	Inspection date	Time since first leak in July 2021	Time since temp patch repair June 2023	Impound level and subsoil flows	Method of inspection	Depression 2	Depression 3	Depression 4	Depression 5	Depression 6	Depression 7	Other
				completion of June repairs.	Flow indication device attached to ROV.	under eastern side of repair. Southern end drops into the depression that extends south of the repair (0.96m x 0.89m, $0.73m^2$) which was unable to be filled with filter sand during the June repair.		$1.54m^2$. The southwest corner has multiple pinhole locations, but only pinholes 4 and 5 were found this inspection. Pinhole 4 (6mm dia) was repaired with butyl tape, which appeared to reduce flow by 1 L/s.				structure. Minor undulations observed.
NPU 19/9/2023	24/8/2023	2yrs 1.8mths	2.2 mths	Impound level just below 267 mRL, and subsoil flows mostly 3.5-5.5 L/s since previous inspection.	ROV with multibeam sonar (Konsberg MS1000).	Change detected - the southeastern corner depression has grown to $6.52m^2$.		No change.	Change detected. Multiple “small gutters” on eastern side. Depression forming in northeastern corner 1m by 1.5m, 0.3m deep. 200mm wide “gutter”, running east to west, merging with the depression at the centre of the north side. All features appear to be aligned east to west.	No change.	New depression identified 22m west of centre hatch, 10m north of Depression 2. 3.05m by 1.60m, 0.5m deep. HDPE lap runs through depression and is lifting i.e., in tension.	Six diagonal subsoil lines were tracked. No undulations found.
NPU 24/10/2023	12/10/2023 **	2yrs 3.4mths	3.8 mths	Impound level just below 267 mRL, and subsoil flows mostly 2.5-4.5 L/s since previous inspection.	ROV with multibeam sonar (Oculus).	No change.		No change.	Further change on northern and southern face, but not to the extent noted in previous surveys. The east-west aligned gutter that merges with the depression at the centre of the north side, has increased from 200mm to 300mm wide. Depression has expanded 200mm beyond chalk line at southern face. Soil/gravel was found on top of the liner.	No change. Wiggly white lines were noted in the centre of the depression and sent to Viking for comment, who advised the shape of these lines means stress on the liner is unlikely. Lack of silt in depression noted by NPU as unusual. Photos show silt “aligned” as if flow may have passed over?	No change.	Visibility poor. Northern batter face inspected. Vertical gutter found just below water line 150mm by 2m in the centre of the face.
NPU 5/4/2024	8/3/2024	2yrs 8.2mths	8.6 mths	Impound level just below 267 mRL until 21/12/2023, then just below 268 mRL. Subsoil flows mostly 2-4 L/s since the previous inspection.	ROV with multibeam sonar (Oculus).	Change detected – additional depression 1m by 1m, 0.35m, formed beyond south edge. No perforations or flow detected.		No change.	Unable to monitor change – chalk lines faded. No tears or leaks detected.	Limited ability to monitor change – chalk lines faded. Possibly unchanged.	No change.	NPU recommends full inspection with 360° multibeam sonar.

* TBC. Inspection date assumed based on date stated in Section 9 of report. Date of inspection stated as 5/5/2023 in Section 1 of report and date stamps on photos are 29-30/6/2023.

** TBC. Inspection date assumed based on date stated in Section 10 of report. Date stamp on photos is 26/8/2023.

Appendix G Concept design for remedial options

- G1: Drainage and stability berm
- G2: Upstream filter blanket
- G3: Downstream filter diaphragm

G1 Drainage and stability berm

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REV	DESCRIPTION	CAD	CHK	DATE	APPROVED	DATE
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DESIGNED	TXH	SEPT.23
DRAWN	HUGO	SEPT.23
DESIGN CHECKED	DMK	SEPT.23
DRAWING CHECKED		

DRAWING STATUS
PRELIMINARY DRAFT

PROJECT PHASE
25% DETAILED DESIGN

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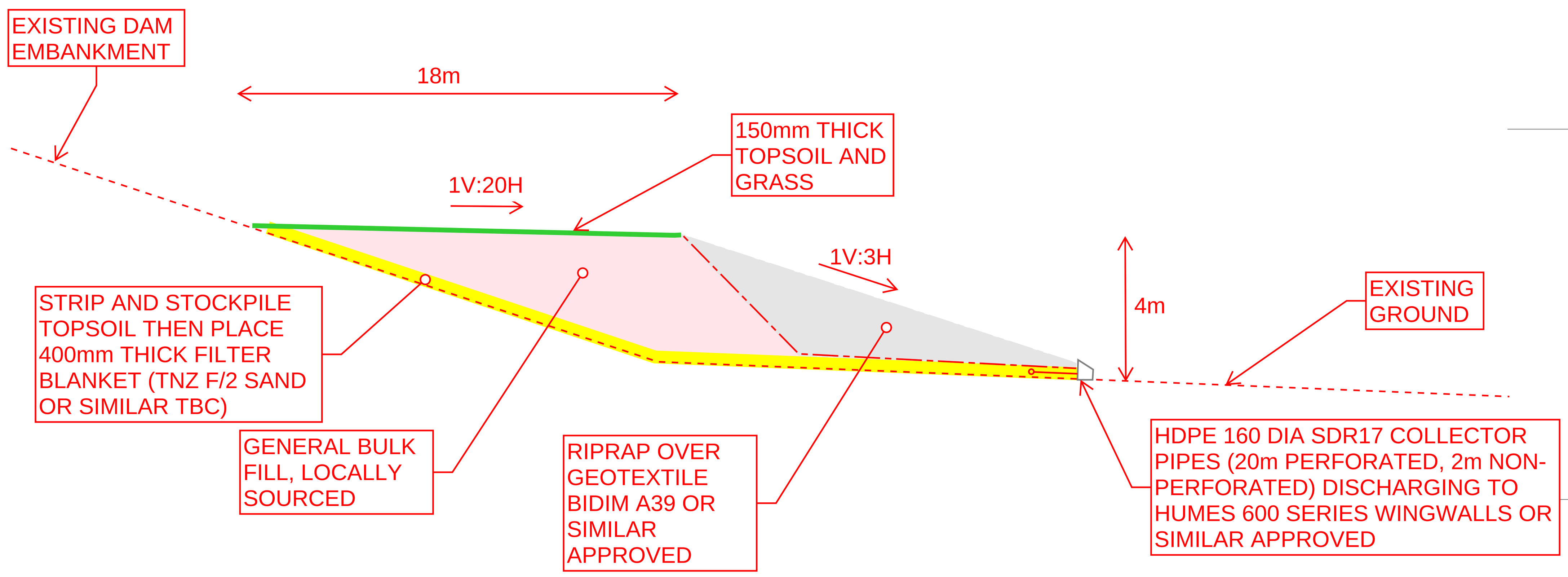
CLIENT TARARUA DISTRICT COUNCIL

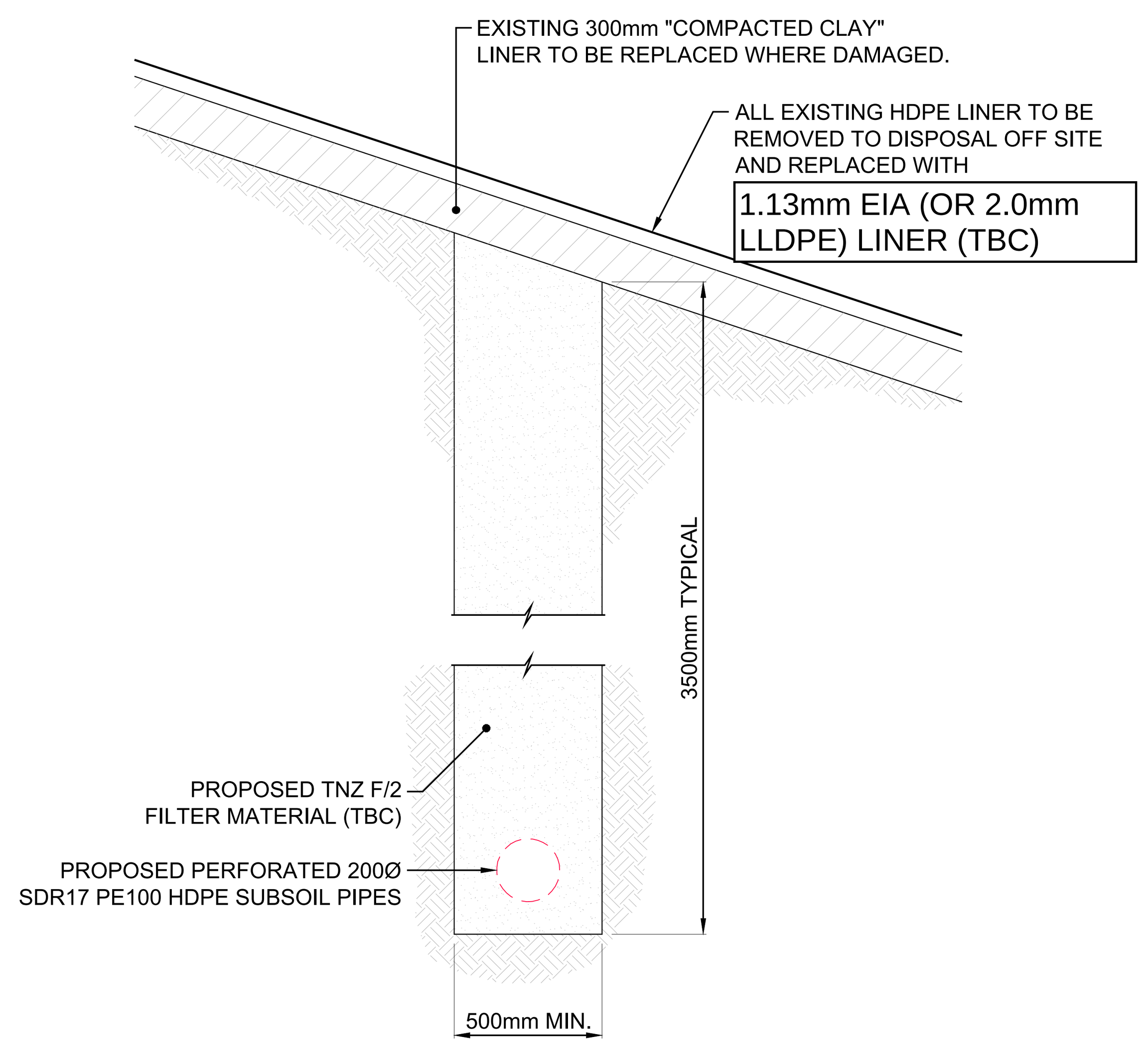
PROJECT DANNEVIRKE RAW WATER RESERVOIR REMEDIATION

TITLE GENERAL
GENERAL ARRANGEMENT PLAN

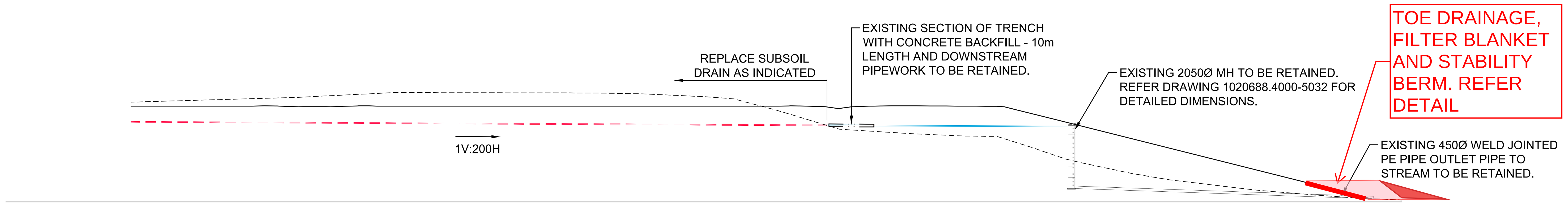
SCALE (A1) 1:1000 DWG No. 1020688.4000-5005 REV 1*

25% DESIGN DRAWING SET MARKED UP WITH ADDITIONAL DRAINAGE AND STABILITY BERM
REV 1*
19/07/2024





RESERVOIR LINER AND SUBSOIL DRAIN BEDDING DETAIL
N.T.S



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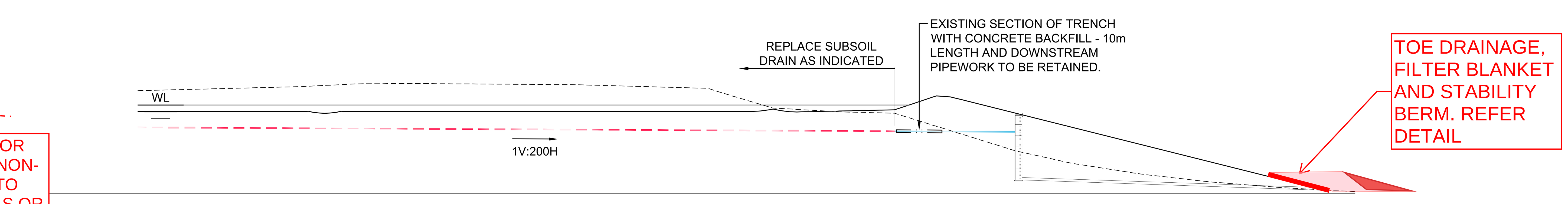

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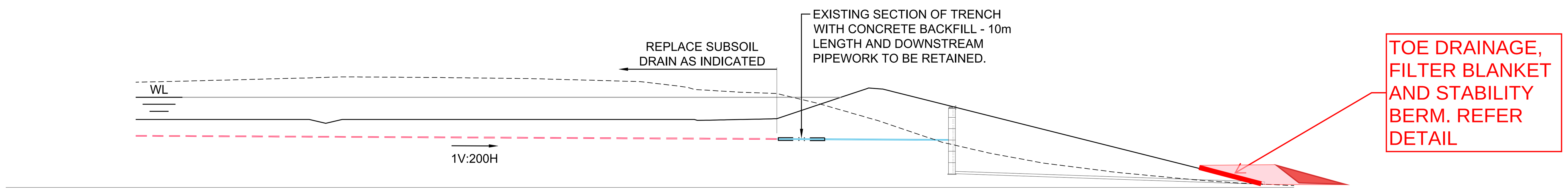

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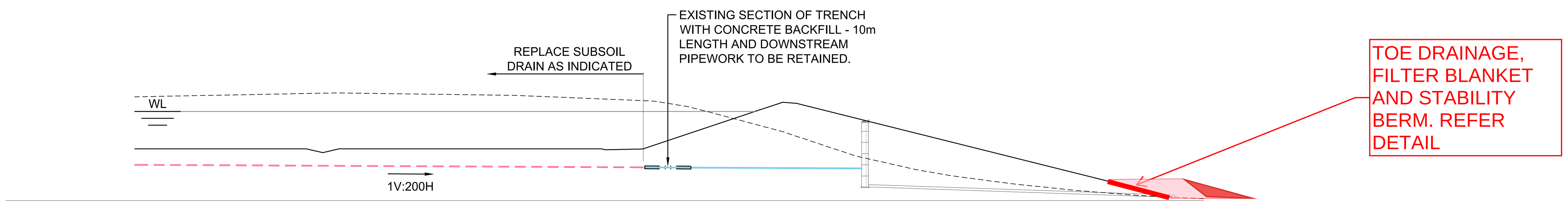
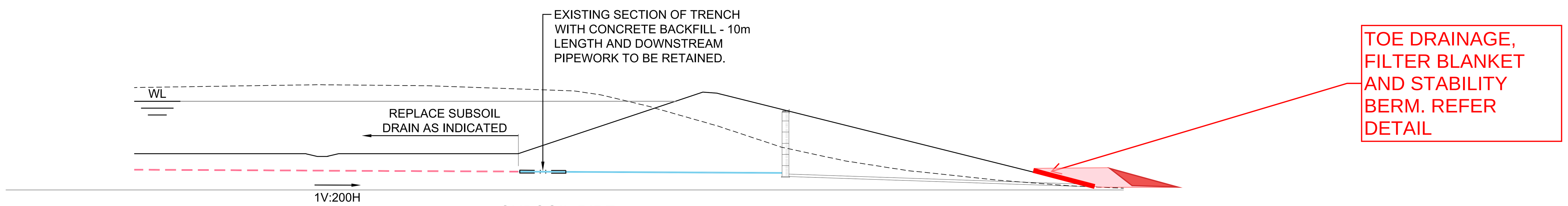
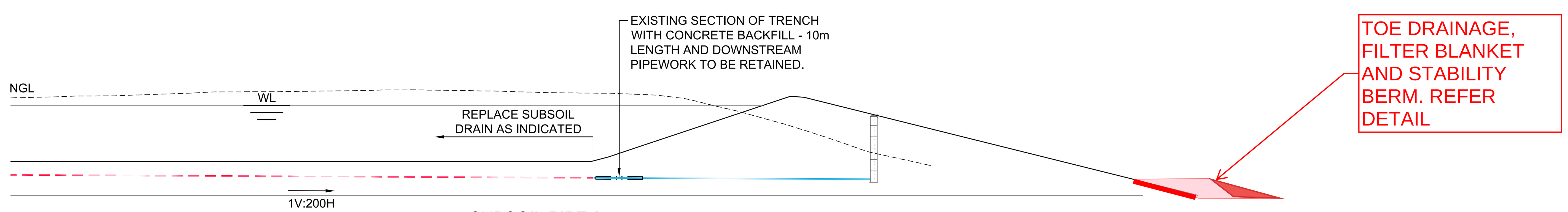


SUBSOIL PIPE 4



SUBSOIL PIPE 5



SUBSOIL PIPE 6

0 18.75 37.5 m
1:750 (A1)
1:1500 (A3)

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REV	DESCRIPTION	CAD	CHK	DATE	APPROVED	DATE
1	25% DETAILED DESIGN ISSUE	HUGO	TXH	15.09.23		

DESIGNED	DSMT	SEPT.23
DRAWN	HUGO	SEPT.23
DESIGN CHECKED	DMK	SEPT.23
DRAWING CHECKED		

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DRAWING STATUS: PRELIMINARY DRAFT
PROJECT PHASE: 25% DETAILED DESIGN

CLIENT: TARARUA DISTRICT COUNCIL
PROJECT: DANNEVIRKE RAW WATER RESERVOIR REMEDIATION
TITLE: RESERVOIR LINER & SUBSOIL DRAINAGE
RESERVOIR LINER AND SUBSOIL DRAINAGE SECTIONS

SCALE (A1) 1:750 DWG No. 1020688.4000-5031 REV 1*

G2 Upstream filter blanket

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25% DESIGN DRAWING SET MARKED UP ADDITIONAL UPSTREAM FILTER BLANKET

REV 1*
19/07/2024

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REV	DESCRIPTION	CAD	CHK	DATE	APPROVED	DATE
1	25% DETAILED DESIGN ISSUE	HUGO	TXH	15.09.23		

DESIGNED	DSMT	SEPT.23
DRAWN	HUGO	SEPT.23
DESIGN CHECKED		
DRAWING CHECKED		

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PROJECT PHASE: 25% DETAILED DESIGN

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CLIENT: TARARUA DISTRICT COUNCIL

PROJECT: DANNEVIRKE RAW WATER RESERVOIR REMEDIATION

TITLE: RESERVOIR LINER & SUBSOIL DRAINAGE
RESERVOIR LINER AND SUBSOIL DRAINAGE PLAN

UNDER REVISION

SCALE (A1) 1:500 DWG No. 1020688.4000-5030 REV 1*

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REV 1*
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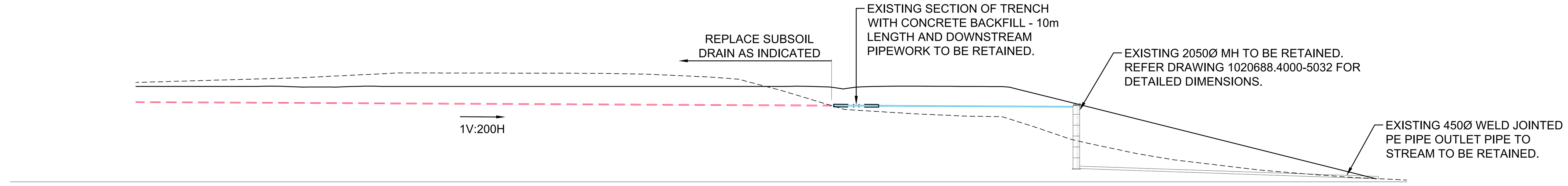

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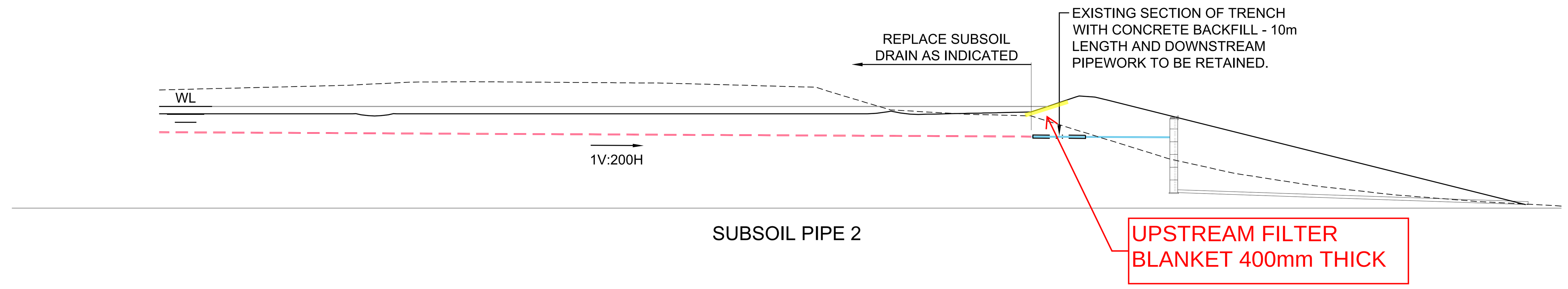

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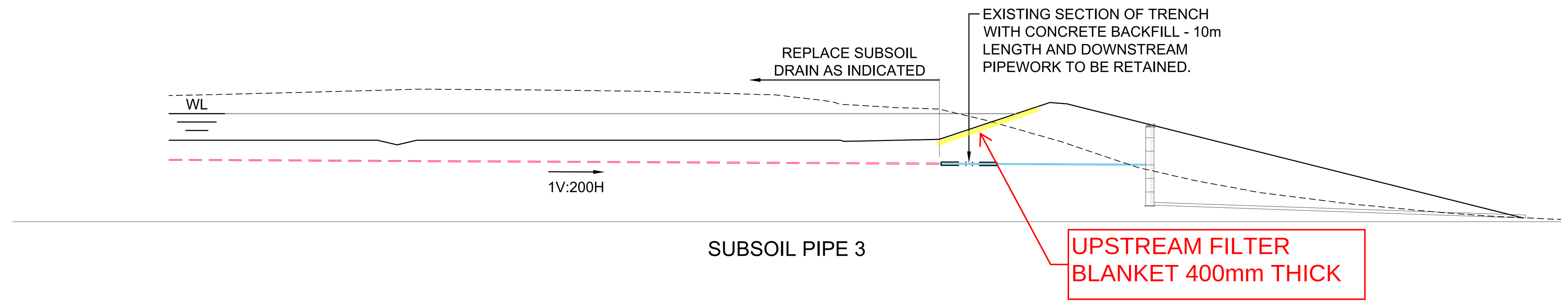

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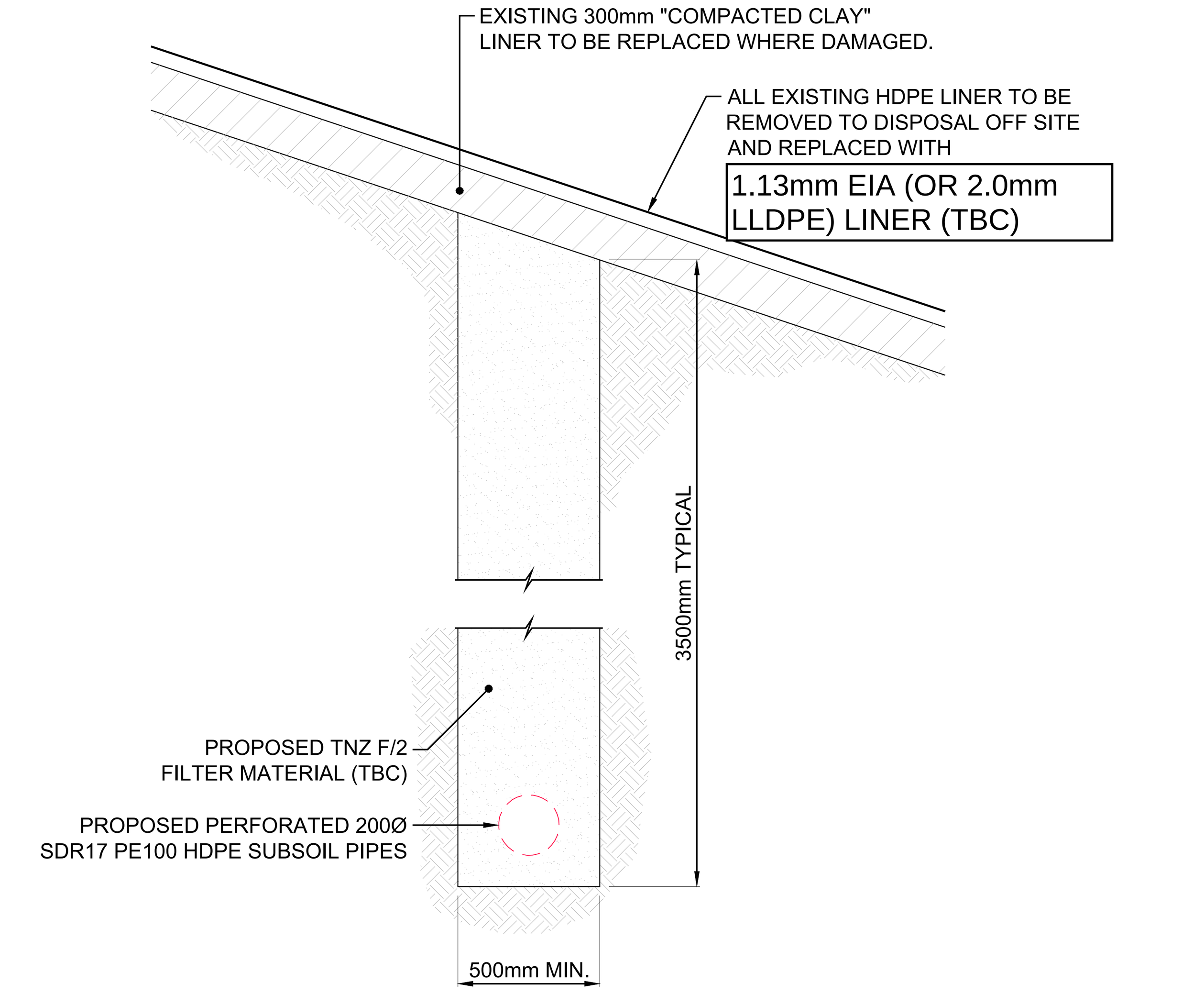
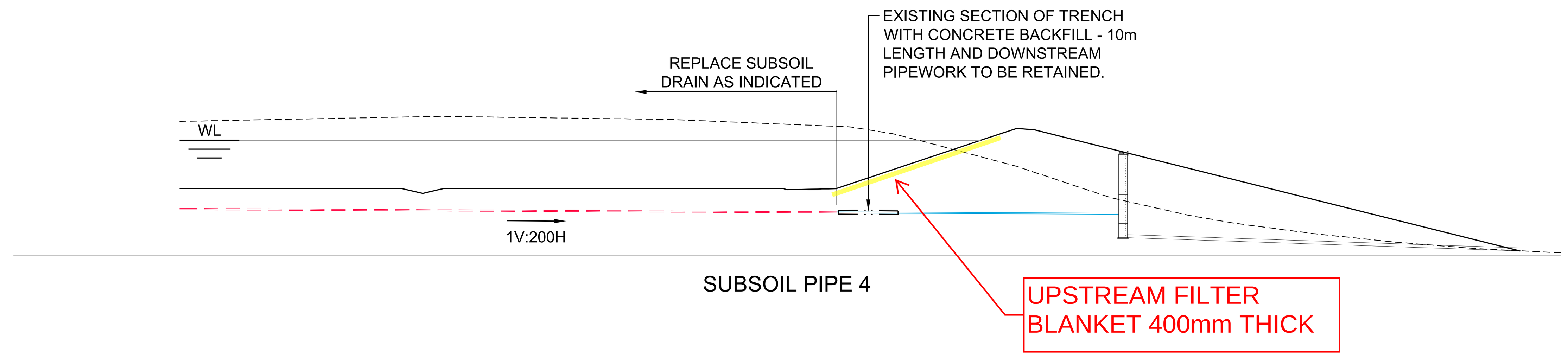


RESERVOIR LINER AND SUBSOIL DRAIN BEDDING DETAIL
N.T.S



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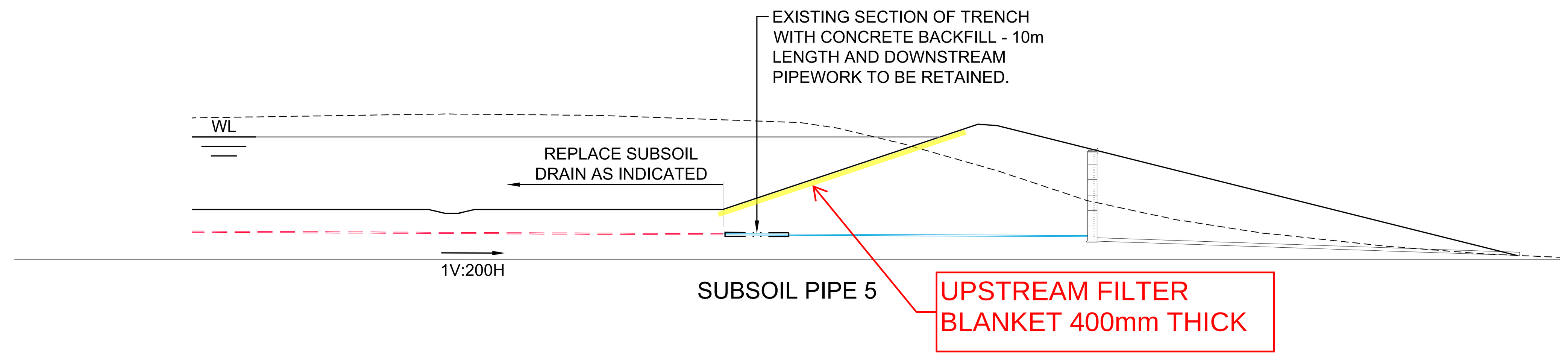

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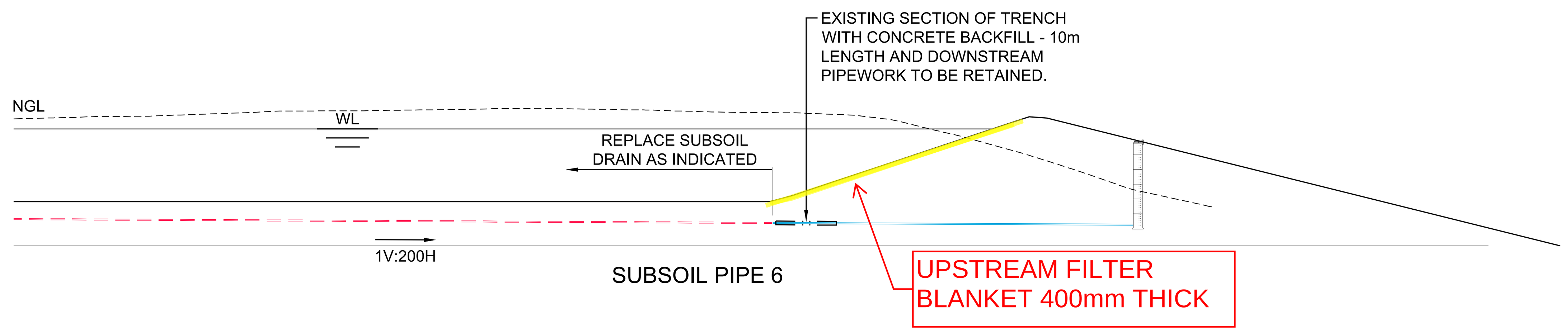

SUBSOIL PIPE 6

0 18.75 37.5 m
1:750 (A1)
1:1500 (A3)

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REV	DESCRIPTION	CAD	CHK	DATE	APPROVED	DATE
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DESIGNED	DSMT	SEPT.23
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DESIGN CHECKED	DMK	SEPT.23
DRAWING CHECKED		

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PROJECT PHASE: 25% DETAILED DESIGN

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CLIENT **TARARUA DISTRICT COUNCIL**
PROJECT **DANNEVIRKE RAW WATER RESERVOIR REMEDIATION**
TITLE RESERVOIR LINER & SUBSOIL DRAINAGE
RESERVOIR LINER AND SUBSOIL DRAINAGE SECTIONS
SCALE (A1) 1:750 DWG No. 1020688.4000-5031 REV 1*

G3 Downstream filter diaphragm and berm

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					DESIGN CHECKED	DMK	SEPT.23	PROJECT PHASE	
					DRAWING CHECKED			25% DETAILED DESIGN	
					NOT FOR CONSTRUCTION			THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION PURPOSES UNLESS SIGNED AS APPROVED	
1	25% DETAILED DESIGN ISSUE	HUGO	TXH	15.09.23					
REV	DESCRIPTION	CAD	CHK	DATE	APPROVED		DATE		

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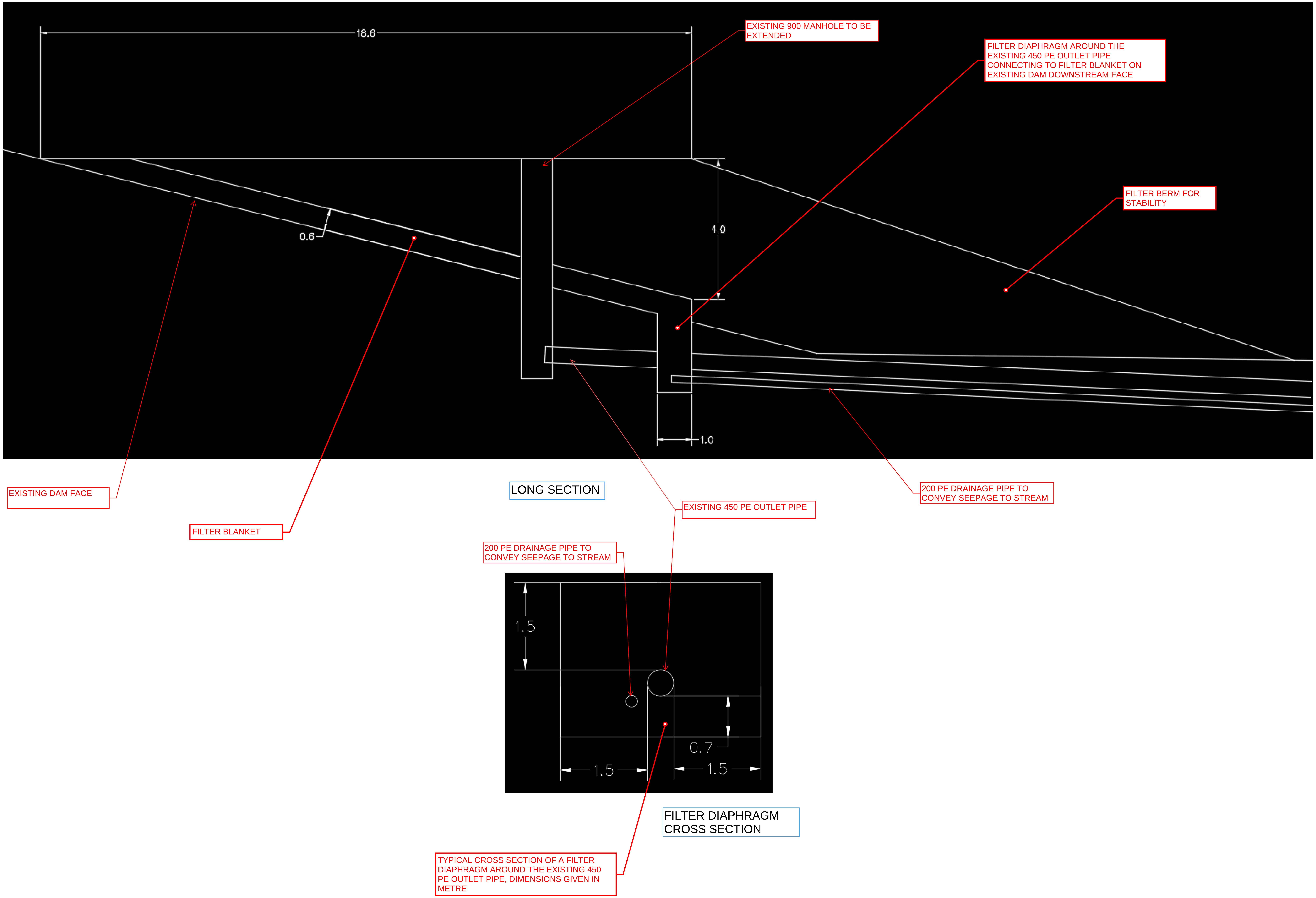
CLIENT TARARUA DISTRICT COUNCIL
PROJECT DANNEVIRKE RAW WATER RESERVOIR REMEDIATION
TITLE GENERAL
GENERAL ARRANGEMENT PLAN
SCALE (A1) 1:1000 DWG No. 1020688.4000-5005 REV 1*


EXISTING 900 MANHOLE TO BE EXTENDED
FILTER DIAPHRAGM AROUND THE EXISTING 450 PE OUTLET PIPE CONNECTING TO FILTER BLANKET ON EXISTING DAM DOWNSTREAM FACE
18.6
4.0
0.6
1.0
FILTER BERM FOR STABILITY
EXISTING DAM FACE
FILTER BLANKET
LONG SECTION
EXISTING 450 PE OUTLET PIPE
200 PE DRAINAGE PIPE TO CONVEY SEEPAGE TO STREAM
200 PE DRAINAGE PIPE TO CONVEY SEEPAGE TO STREAM
1.5
0.7
1.5
1.5
FILTER DIAPHRAGM CROSS SECTION
TYPICAL CROSS SECTION OF A FILTER DIAPHRAGM AROUND THE EXISTING 450 PE OUTLET PIPE, DIMENSIONS GIVEN IN METRE

Appendix H Cost estimates

Level of design

The cost estimates presented in this report are based on concept design level arrangements. Concept design is an early stage of design development, and it is likely that the final design will differ. The options presented are based on limited information and informed judgement. Calculations, modelling, further investigations, and detailed design have not yet been undertaken.

Overall remedial works costs

The overall remedial works costs have been referred to for the purposes of putting the costs of the options in context.

Costs for outlet works, inlet works, and instrumentation have not been updated since our "Concept design and preliminary cost estimates for remediation options" report (T+T July 2023). They do not yet reflect the 25% design.

Costs for reservoir works have been updated based on the 25% design, as modified following 25% design workshops (change from HDPE to 2.0mm LLDPE or 1.13mm EIA liner, and add 1.52mm EIA floating cover), and rates for geomembrane based on quotes from suppliers provided in September 2023.

Cost build up

The preliminary cost estimates have been built up as summarised in Table G1.

Table H1: Build up of preliminary cost estimates

Item	Allowance
A. Direct works	Physical works on site (varies with option)
B. Indirect works (Preliminary & General)	25% of [A] Includes both fixed (5%) and time-related costs (20%) Includes establishing site perimeter, staging areas and offices, environmental controls, dewatering, demobilisation, and disestablishment.
C. Contractor risk	5% of [A + B] Allows for buildability, methodology, programme, and contractual risks.
D. Contractor off site overhead and profit	12.5% of [A + B + C]
E. Subtotal	[A + B + C + D]
F. Lower contingency	20%
G. Upper contingency	40%
H. P50 estimate	[E – F% to +G%]

At this early stage of design, a contingency range is adopted based on judgement and experience. During a more detailed stage of project development, the contingency would typically be calculated based on a Monte Carlo analysis. At the current concept design stage, contingency typically ranges from 25% to 40%, and up to 65% for non-standard projects. The percentages adopted for the current project are presented in Items F and G in Table H1.

The cost estimate for each of option is a “middle” or “P50” estimate, meaning there is notionally a 50% chance of the actual cost being greater or smaller than the estimate. The contingency is expected to be spent and brings the base estimate up to the P50 level.

A reasonable allowance for a cost range around the “middle estimate” is -30% to +100% at the concept design stage. The asymmetry of these percentages reflects a right-skewed distribution i.e., that cost estimates tend to run over more often than under.

Limitations, exclusions, and assumptions

The concept sketches for remedial works and associated cost estimates have been developed to a “concept design” level for comparing options and should not be relied upon in terms of absolute financial feasibility.

The following costs are excluded from the assessment:

- Government taxes,
- Design, technical advice, and investigation costs,
- Legal costs,
- Financing,
- Consenting,
- Insurance,
- Environmental offsets and compliance (other than a nominal allowance for erosion and sediment control),
- Construction monitoring, quality assurance, and certification,
- Land purchases,
- Subsidies or compensation for farm disruption,
- Operational costs,
- Mitigation measures to offset water supply risk while the reservoir is dewatered for the works,
- All bulk fill is assumed to be imported from within 25km of site,
- LPF is assumed to be sourced from the field immediately beside the reservoir,
- Cut to waste from “full rebuild” is assumed to be cut to waste and spread on site. No disposal cost has been included.
- All material to be carted off site is assumed to be disposed within 25km of site,
- Instrumentation items have been captured as a provisional sum,
- Construction cost volatility due to changes in costs of commodities subject to currency exchange fluctuations or world demand such as steel and fuel; the degree of demand for relevant potential contractors in the market at the time of bidding.

Cost estimates

Item	Middle “P50” estimate	Cost range -30% to + 100%
Overall remedial works cost, including: • Reservoir works (liner system, subgrade, subsoil network, floating liner, ring beam) • Outlet works • Inlet works • Instrumentation	$ 8.0 M to $ 9.3 M	$ 6.0 M to $ 17.2 M
Drainage and stability berm	$ 2.8 M to $ 3.2 M Adds 35% to overall remedial works cost	$ 2.1 M to $ 6.0 M
Upstream filter blanket	$ 0.7 M to $ 0.8 M Adds 8% to overall remedial works cost	$ 0.5 M to $ 1.4 M
Full rebuild of eastern dam (from “Concept design and preliminary cost estimates for remediation options” report (T+T July 2023).	$ 12.2 M to $ 14.3 M Adds 154% to overall remedial works cost	$ 9.3 M to $ 26.5 M

Appendix I Peer review comments

Comment No.	Report Section	Review Comment	T+T proposed response 8 Aug
	General		
1.	General	• An appendix documenting liquefaction and cyclic softening assessments is needed to support conclusions.	• Please refer to Appendix D8 of the 25% Design Report. • We will add a cross reference to this previous appendix in Section 2.3.4 of the subject report.
2.		• An appendix documenting how geotechnical parameters in Table 2-8 were derived is needed.	• We will provide further reasoning on how the geotechnical parameters have been developed in Table 2-8 footnotes.
3.		• An appendix documenting the seismic deformation estimates is needed.	• We will add these calculations to Appendix C.
4.	2.2 Geotechnical conditions	• An interpretation of the Stage 2 geotechnical investigation data by each geologic unit or fill material is needed. Including: • Cone Penetration Tests (CPTs), • Standard Penetration Testing (SPTs), • Hand-held shear vane testing	• A summary of the CPT, SPT and shear vane test results for each geological unit is already presented in Table 2-8. • We will add a reference forward to this table from Section 2.2.2.

5.	2.2.2 Project specific geotechnical investigation	• The interpretation of the original Hole Erosion Test (HET) is needed.	• The intent of the exclusion in Table 2 and Attachment 1 of our Work Package Plan (7 June 2024) was that interpretation of the HET testing was excluded. • Our current advice is that there is a high risk related to the potential failure mode in Figure 0.1 in the existing situation, which could deteriorate rapidly to an emergency at any time. Our current recommendation is that TDC should work towards reducing this risk as quickly as practicable. We deferred the HET interpretation because it is unlikely to change this advice and recommendation. • The risk in the current situation depends on multiple areas of uncertainty i.e., defect presence, defect persistence, defect size, hydraulic head, and soil resistance. Interpretation of the HET testing will reduce (but not eliminate) uncertainty about one of these areas, namely soil resistance. • If the interpretation indicates material resistance is better than typical, the other areas of uncertainty still pose a high risk. If the interpretation indicates material resistance is worse than typical, the risk of the potential failure mode will be judged higher, but the other areas of uncertainty and the critical need for the reservoir for water supply, mean that TDC is still unlikely to immediately move to an emergency response of dewatering the reservoir. We expect there would need to be a very clear sign of imminent failure before TDC would dewater the reservoir due to the implications for water supply. • That said, interpretation of the HET results will provide value in improving the understanding of the current risk, we just do not expect it to improve the understanding enough to change the go / no go decision for TDC. We would be pleased to offer a price for interpretation of the HET testing.

6.	2.2.5 Ground model	• The Ground Model should include groundwater but does not.	• We will update our figures to include locations of observed seepages and measured groundwater including inferred groundwater level for analyses. • We will also add a section in 2.2.5 summarising our interpretation of the groundwater regime at the site.
7.		• Table 2-7 and Table 2-8 should include the clay liner (compacted Loess) as modelled in the stability analysis. • The report should state on how the clay liner was modelled.	• We will add a description and parameters adopted for the clay liner (compacted Loess) in Table 2-7 and Table 2-8.
	3. Stability of western reservoir rim		
8.	3.1.1 Methodology – Seepage Analysis General	• Results of the transient analysis should also include plots of pore pressure.	• We will add this additional output plot to the report.
9.	3.1.1 Methodology – Stability model	• The report should clearly state how the groundwater was modelled. It appears it was modelled to be representative of the existing conditions with a compromised liner condition.	• We believe the approach to modelling groundwater is already described in Section 3.1.1 and shown visually in Appendix C.

10.		• Information to support modelling the western rim with seepage (piezometric surface) daylighting along the ridge near downstream toe is needed.	• The groundwater level is based on: 12 months of groundwater monitoring undertaken within BH02 (as presented in Table 2-6), and; observations of groundwater seepage, most notably at mapping waypoint WP03 (elevation 263 m RL, as presented in the Stage 2 Factual Report, T+T December 2023) a flowing seepage was observed which was generally consistent with other evidence of seepage at this elevation along the western rim e.g. hydrophilic vegetation and soil slumping. • We will add specific cross references to Table 2-6 and WP03 to the description of how groundwater was modelled in Section 3.1.1. As per item 6 above, we will also update the ground model figures to show the ground water and seep locations.
11.		• The report should clearly state how the reservoir is modelled in the stability analysis. Some analyses show it as a surcharge load (hatched area) while other analyses do not show the surcharge load (applies to eastern embankment also).	• The operational range of the reservoir ranges from Full Supply Level to close to empty. The reservoir is assumed to be drawn down where this is more critical for stability performance. We can add this statement into the report.
	4. Stability of eastern dam embankment		
	5. Potential for internal erosion of the eastern dam embankment, including the foundation and backfill around the subsoil outlet pipe		

12.	General	• The likelihood of continuation, progression and intervention are not discussed for the PFM (Figure 0.1). We note that Appendix D covers continuation (i.e., there is no filter so it continues) but progression does not include a discussion of all the steps (i.e., pipe filling action and flow limited). Intervention is not discussed for all cases.	• The design intention for the remedial solution is that a Potential Failure Mode due to concentrated leak erosion is mitigated through the prevention of initiation in relevant loading conditions. The remedial solution will provide confidence that hydraulic conditions will be insufficient to create a driving head for concentrated leak erosion to initiate. • As indicated in Section 5.5, further work is recommended to assess the likelihood of concentrated leak erosion through cracks induced by a seismic event.
13.	5.1.4.2.2 Implications for risk following remediation	• This section does not address the risks under seismic loading, only normal loading. All design loads (i.e., OBE and SEE) along with consideration of the NSHM ground motions should be addressed. As stated in the report NSHM provide the current understanding of New Zealand seismic loads. Although there is uncertainty in how the NHSM will be considered in the NZ Dam Safety Guidelines, it would be prudent to consider them now rather than possible re-design later.	• We will add a brief discussion regarding the risks in seismic loading. A preliminary indication of what this will include is provided below. • In a seismic event which does not compromise the liner, as for normal operating conditions, hydraulic conditions are not expected to be sufficient to initiate concentrated leak erosion. • In an event where the liner is compromised, the preliminary transient seepage analysis discussed in Section 3.1.2 and Section 4.1.2 has shown that with the liner removed (representing damage following a SEE) only the first few meters of the dam embankment have time to become saturated if the dam is drawn down over the expected three to four days. This small advance of the wetting front is not expected to be sufficient for concentrated leak erosion associated with the conduit to occur. The risk is reduced further by the upstream filter blanket. • The assessment of concentrated leak erosion due to cracks in the embankment induced by seismic activity are excluded from the current scope of works. As detailed in Section 5.5.1, it is recommended that a respective assessment is undertaken. These will further inform the susceptibility of the dam to concentrated leak erosion in a seismic event.

14.	5.1.4.3 Concentrated leak erosion risk in transverse cracks caused by settlement in earthquakes	• This is a different potential failure mode to the internal erosion PFM (Figure 0.1) and should be discussed is a separate section of the report.	• We will give further consideration to the arrangement of headings and sub-headings in the report.
15.	Table 5-2 and Table 5-3	• The Plasticity Index (PI) should be presented for each unit based on laboratory test results instead of the either Yes/No or qualitatively “generally non-plastic’	• The Plasticity Index is already presented for each unit in Table 2-2. • There is already a reference back to Table 2-2 in the headings of Table 5-3. We will add a similar reference back in Table 5-2.
16.	Table 5-7	• Remedial options do not address the internal erosion PFM as shown in Figure 0.1	• The remedial options address two risks relating to internal erosion as described in the two bullet points preceding the table. The second of these bullet point risks relates to Steps 4 to 7 of the PFM in Figure 0.1. The third column of Table 5-7 describes how each remedial option is intended to reduce the two bullet point risks. • Steps 1 to 3 of the PFM in Figure 0.1 are addressed by remediation of the subsoil drains, subgrade, and liner system as described in Section 5.3.1.
	Items raised by peer reviewer at meeting on 5 August (added by T+T)		
17.	Seismic stability analyses	• Concerned about using drained parameters for seismic stability analyses.	• We have used the staged pseudo-static option available in Slope/W for seismic analysis. We consider this option along with the use of c’ and ø’ to define the shear strength prior to seismic loading is appropriate (refer to Geostudio manual on multi-stage pseudostatic analysis). However, for the softened Loess we have considered both drained and undrained parameters.

18.	Internal erosion remedial solutions	• Concerned about bypass of the upstream filter. • Suggested we consider extending the upstream filter should some distance up the northern cut batter to avoid bypass.	• The cost estimates in Appendix G for the overall remedial works allows for a provisional 400mm sand filter across the floor of the reservoir. • The cost estimates for the upstream filter blanker cover the eastern batter slope only (with a few extra meters beyond the corner), albeit Table 5-7 in Section 5.3.2 mentions the risk of extending the blanket to cover the western batter slope too. • Having given this further consideration following the meeting on 5 Aug, we do not believe the benefit provided by extending the blanket along the northern cut face is compelling. The subsoil drains on the northern face are inferred to be trenched in insitu Tamaki Alluvium. In the upgrade design, these subsoil drains are fully replaced with a more durable pipe and filter compatible bedding, which will be specified to provide a very low risk of defects along the remediated subsoil drains in line with modern practice. In the case of the upstream filter on the eastern face, this will be placed immediately against existing defects (if present) associated with the subsoil outlet trench (i.e., for the section of eastern dam embankment not being replaced) or any new defects that could open up towards the crest of the dam embankment in the SEE.
19.	Section 5.1.4	• Suggested adding comment on the potential for arching across the trench to lead to defects subject to concentrated leak erosion	• We will add a comment regarding arching as suggested.
20.	Section 5.1.4.2	• Found last bullet point confusing. Believes the example “cracking or hydraulic fracture in poorly compacted layers in the embankment” is actually what we have assessed rather than what we have not assessed.	• We will clarify the text in the report. “Cracking or hydraulic fracture in poorly compacted layers in the embankment” is a reference to Initiation Mechanism 14 (IM14) in Fell et al. (2015), which has not been assessed directly. However, as indicated in Section 5.1.4.2 “two mechanisms [assessed] are considered to have provided an indication of the susceptibility to concentrated leak erosion in general terms.”

21.	Significance of stability non-compliances	• Requested information on the extent and location of non-compliant slip surfaces, not just the critical / worst slip surface, which is sometimes relatively shallow or doesn't threaten containment of reservoir contents.	• We believe this is already covered in the comments column in Table 0.1 and in descriptions in Sections 3.1.2.1, 3.1.2.2, 4.1.2.1, and 4.1.2.2. • We believe the key takeaway from the stability non-compliances is that they are much, much lower risk than the potential failure mode shown in Figure 0.1.