RILEY CONSULTANTS

GEOTECHNICAL AND HYDRAULIC INPUTS LONGFORD SHARED PATH, GORE

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GEOTECHNICAL AND HYDRAULIC INPUTS
LONGFORD SHARED PATH, GORE

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GEOTECHNICAL AND HYDRAULIC DESIGN INPUTS LONGFORD SHARED PATH, GORE

1.0 Introduction

Riley Consultants Ltd (RILEY) has been engaged by DC Structures Studio Limited (DC Structures Studio) on behalf of Gore District Council to provide geotechnical and hydraulic design inputs for a proposed shared cycle and pedestrian bridge across the Mataura River in Gore.

Aspects covered within this report include:

- Flood frequency analysis of stream gauge recordings for the Mataura River at Gore, using annual maxima from 1960 to 2020.
- Consideration of climate variability and change with a focus on the Interdecadal Pacific Oscillation (IPO).
- Selection of peak discharges for serviceability and ultimate limit stages.
- Development, calibration, and validation of a two-dimensional hydraulic model in HEC-RAS v5.0.7.
- Prediction of peak flood levels and velocities for the design events, and consideration of the potential effects of the proposed bridge on flood conveyance.
- Design of scour protection works for the west (right) approach embankment.
- A summary of the geotechnical conditions at the bridge site and implications for foundations and anchoring.
- Geotechnical evaluation of foundation piles and backstay anchors.
- Recommendations for earthworks standards and other construction considerations.

The report builds on early work by RILEY, summarised in a report titled Preliminary Geotechnical and Hydraulic Inputs (RILEY Ref: 190376-A, dated 3 December 2019). Our assessment is based on the following sources of information:

i. Preliminary Engineering Report v.3, All Civil Engineering (ACE) Consultants (2019).
ii. Geotechnical Investigation and Factual Report for the Proposed Longford River Track – Mataura Cycle/Pedestrian Bridge, Gore, Insight Engineering (ref: 19033, 2019)
iii. Preliminary Hydraulic Report, Landpro Ltd (Landpro) (ref: 19241, 2019)
iv. Flow gauge recordings and rating information for the Matuara River at Gore from Environment Southland (ES)
v. Topographic information based on LiDAR, manual survey, and river transects supplied by Landpro.
vi. Concept design drawings and indicative structural loadings provided by DC Structures Studio.
vii. Feedback from a meeting held at ES on 27 November 2020, regarding the hydraulic inputs (minutes attached).

2.0 Bridge Location and Structural Form

The proposed structure comprises a 90m span by 3m wide cable stay footbridge constructed of steel and glulam deck, and a 32m high steel mast, refer to Figure 1. It will serve as a central link to a wider cycle network connecting east and central Gore across the Mataura River.

The bridge will span the Mataura River main channel approximately 650m upstream of the existing State Highway 1 bridge in Gore. The river at this location comprises an approximately 80m wide main channel and 320m wide floodplain, with stopbanks on either side. The design includes a 5m high approach embankment at the right (west) abutment, and the left (east) abutment, adjacent to Huron Street, will tie directly into the stopbank.

Figure 1: Artists impression of Longford Shared Path looking upstream towards the east.



3.0 Waterway Design

3.1 Available Hydrological Data

Background information on the Mataura catchment hydrology is presented in a report by OPUS[1]. With an estimated size of 3,524km^2 at Gore, the Mataura catchment is the second largest in Southland, both in terms of area and flow. Flow within the Mataura River is highly variable owing to its alpine headwaters, considerable catchment size, and cyclic behaviour associated with the IPO and Southern Oscillation Index (SOI).

For the purposes of deriving design floods, we have performed a flood-frequency analysis of data from the Gore hydrological gauging station, located ~950m downstream of the site, i.e. 300m downstream of the SH1 bridge.

ES have provided the following information for the purposes of our assessment:

[1] OPUS International Consultants Ltd (2012) *Hydrology of the Mataura River: A constraint on water abstraction at Gore*

I. Annual flow maxima from 1960 to 2019, with updated ratings based on a General Extreme Value (GEV) distribution.

II. Flow and depth hydrographs for the five largest recorded events.

III. Flow and depth hydrographs for the recent February 2020 flood (flood of record).

3.2 Probability Analysis Methods and Climate Variability

An L-moment analysis, applying the method outlined by Hosking[2], was performed to determine which extreme value distribution best fitted the statistical parameters of the data set. L-moment ratios were preferred over other moments as they do not require an assumption of the normality of the dataset and are less sensitive to outliers or skewed distributions.

It is important to note that the period of record (60 years) at the gauging station includes both positive and negative phases of the IPO index, which appear to coincide with periods of above or below average flow conditions, respectively. More recently (post 1999), we appear to be in a period of predominantly negative IPO, meaning that revisions to the flow ratings during this time may downgrade flood estimates, which may affect the accuracy of future predictions.

Three scenarios were considered to test the sensitivity of the distributions and, hence, flood estimates on the dataset:

Scenario 1. Utilises the full length of record (1960 to 2019), containing approximately 40 years of the IPO in a negative phase, and 20 years in a positive phase.

Scenario 2. Covers the period where the IPO is in a predominantly positive phase (1977 to 1999), which favours El Niño conditions.

Scenario 3. Covers the period 1968 to 2010 where the IPO was in a positive phase and negative phase for a similar amount of time (approximately 20 years).

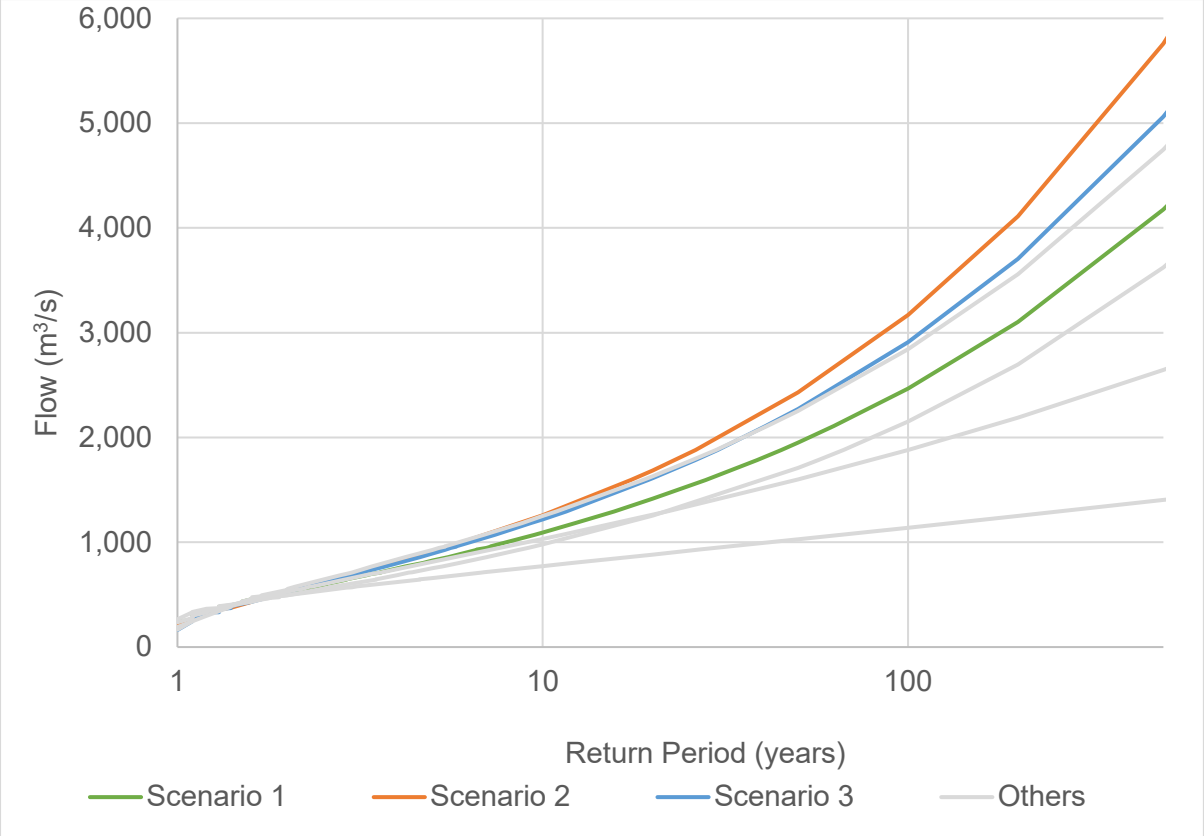
Four additional scenarios applying different combinations of negative and positive phases were also evaluated.

Comparison of the L-Skewness and L-Kurtosis, with the range of common statistical distributions used in extreme event analysis, indicate that Scenario 1 is best estimated by a Log-Pearson Type 3 distribution, and Scenarios 2 and 3 by Log-Generalised Parento distributions. The other scenarios, looking at different combinations of negative and positive phases, indicate that GEV provided the best fit. ES also applied a GEV distribution in their analysis, and this is a common method applied to NZ river statistics.

Figure 2 illustrates the difference between GEV estimates for the three scenarios using Hilltop Software. This illustrates that Scenario 1 predicts lower flows compared to Scenarios 2 and 3, however, sits somewhere in the middle of the seven scenarios evaluated. A check of the results using the Gringorten plotting method indicates that GEV appears to slightly underpredict for floods with a return period up to about the 100 years, other distributions appear more conservative, particularly at high return periods.

[2] Hosking, JRM (1990) *L-moments: analysis and estimation of distributions using linear combination of order statistics*

Figure 2: GEV Distribution fitted to Various Scenarios



3.3 Design Flood Events

Table 2.1 of the NZTA Bridge Manual 3rd Edition (2018) defines the following annual exceedance probability (AEP) for floodwater actions on permanent footbridges:

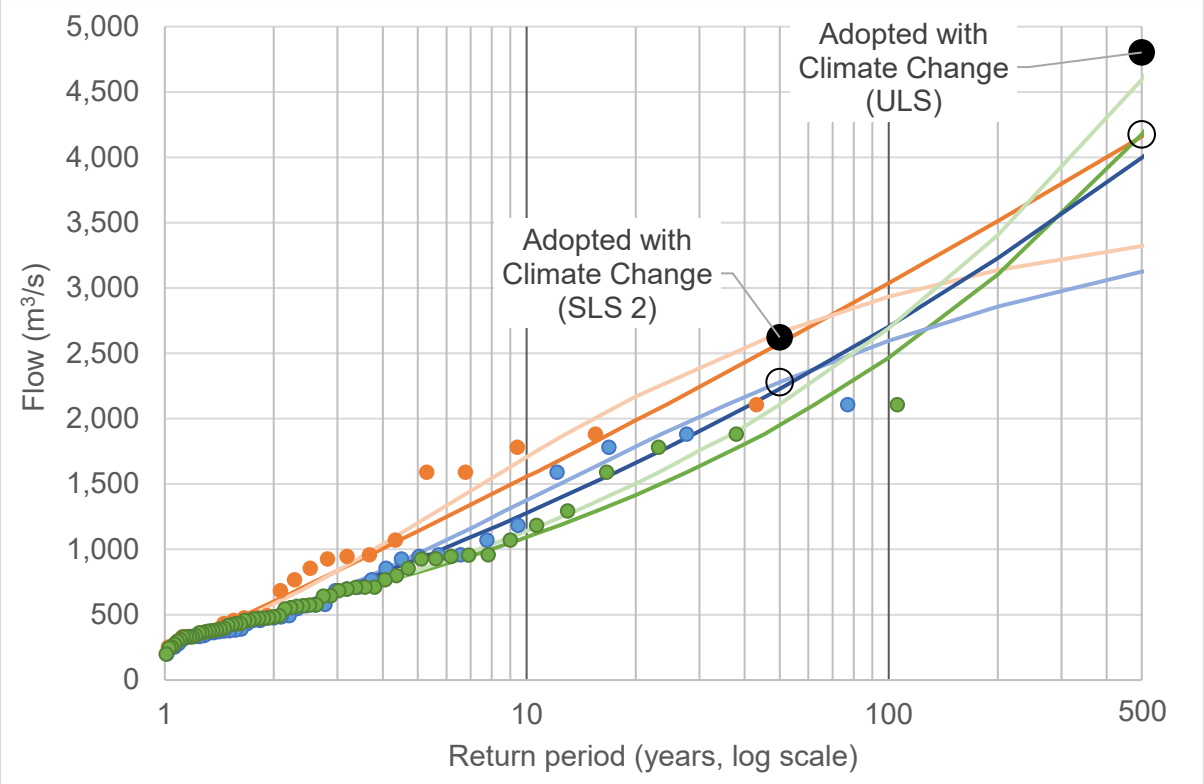
- Serviceability limit state (SLS 1): 1/25 – bridge remains undamaged.
- Serviceability limit state (SLS 2): 1/50 – bridge remains operationally functional.
- Ultimate limit state (ULS): 1/500 – bridge maintains stability.

In addition, the waterway needs to be capable of passing the SLS 2 flood (1/50) whilst maintaining the requisite freeboard in Table 2.4 of the Bridge Manual. Further discussion on this aspect is presented in Section 4.2.

Figure 3 presents the GEV estimate for Scenario 1 alongside Generalised Parento distributions (Scenarios 2 and 3). Estimates are presented with or without logarithmic transformations.

We have adopted a peak flowrate of 2,280m^3/s for the SLS 2 event and 4,176m^3/s (pre-climate change) as illustrated by the black circles in Figure 3. An allowance of 15% above this is also shown (black filled circles) to account for climate change effects in accordance with Section 3.4.

Figure 3: Flood Frequency Analysis Results



3.4 Allowance for Climate Change

In order to provide a basis for climate change allowance over the design life of the bridge, we have compared estimates of high intensity rainfall for the existing scenario against predictions for the RCP 6.0 scenario up to the period 2100. This approach assumes that increases in rainfall result in a commensurate increase in flood magnitude.

The RCP 6.0 scenario was chosen as it represents around a 2° increase in annual mean temperate to 2010, which is consistent with values for Southland in Table 2.3, Climate Change effects and impacts assessment: A guidance manual for local government in New Zealand (Ministry for the Environment, 2008).

Figure 4 illustrates the percentage change for a range of storm durations between ten minutes and 120 hours at two locations in the catchment (headwaters and catchment centroid). A much greater change (10% to 20%) is predicted for the catchment headwaters (at much higher elevation) compared to the catchment centroid (5% to 10%). This is likely a result of the strong correlation between altitude and extreme rainfall often observed in New Zealand catchments. The average increase in rainfall is about 15% for storms up to two hours long, which decreases linearly to about 5% for much longer storm durations.

Overall, we consider an allowance of 15% is reasonable, if not conservative, on the basis of: 1) the bridge site being located downstream of the catchment centroid, and 2) the critical storm durations considered are likely to be in the order of 40 hours+ based on historic flood events.

Figure 4: Percentage Increase in High Intensity Rainfall Predictions Between Now and 2100

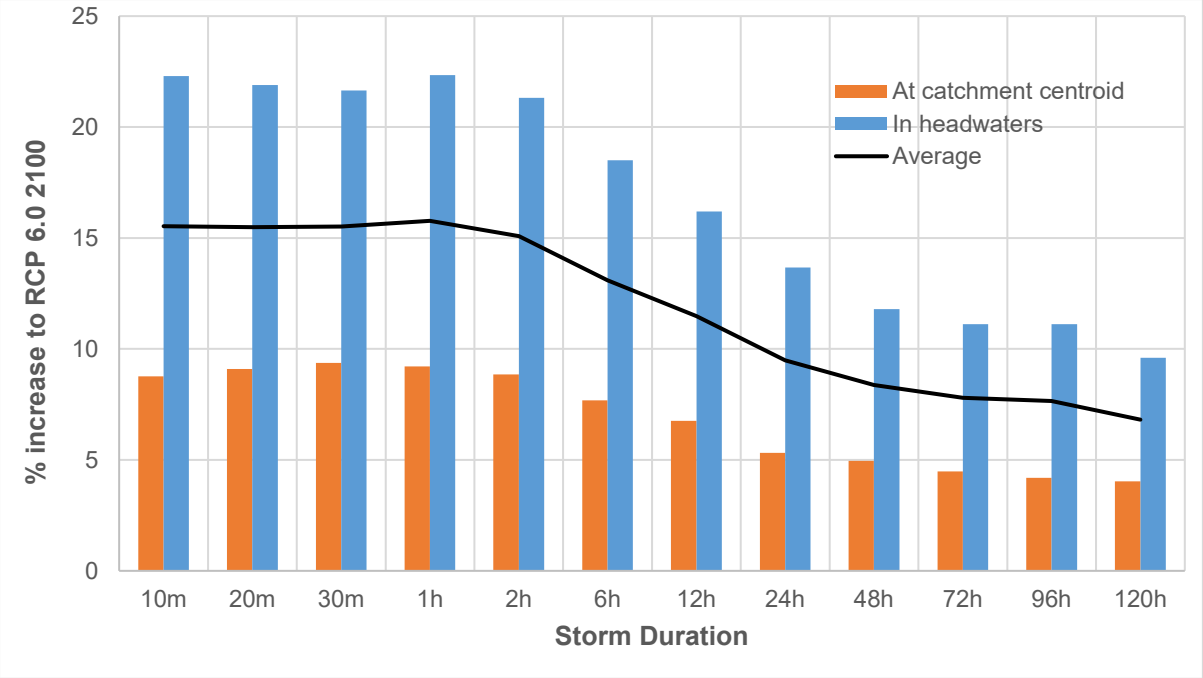


Table 1 summarises the adopted 1/50 and 1/500 peak flows. These are similar, if not slightly above the supplied ES estimates. As shown in Figure 3, the adopted flowrates including climate change are coincident with the maximum predicted flood magnitude for when the IPO is in a predominantly positive phase. This provides some confidence that the adopted peak flowrates are reasonable for the range of possible future scenarios, including climate change and IPO phase, irrespective of the applied extreme value distribution method.

Table 1: Adopted 1/50 and 1/500 Flood Magnitudes

Limit State	AEP	Supplied from ES (GEV) (m^3/s)	Adopted (m^3/s)	With Climate Change(m^3/s)
Construction Flood	1/50	1,952	2,280	-
SLS 2	1/50	1,952	2,280	**2,622**
ULS	1/500	4,176	4,176	**4,802**

4.0 Hydraulic Modelling

4.1 Model Development and Calibration

A hydraulic model has been enveloped in HEC-RAS v.5.0.7 to model flow beneath and around the bridge in two dimensions. The following summarises the model particulars:

- **Model boundaries:** upstream of 420m from bridge site, and 1,500m downstream of SH1 bridge, extending west and east beyond the stopbanks either side (Figure 5).
- **Topography:** combined LiDAR, ground survey and river transects supplied by Landpro. Channel bathymetry interpolated in 12d using aerial imagery. Models runs for post-bridge included 3D structure/earthworks profile.
- **Mesh size:** 5m square grid (global), locally increased to 0.5m around abutments.
- **Upstream boundary condition:** inflow hydrograph.

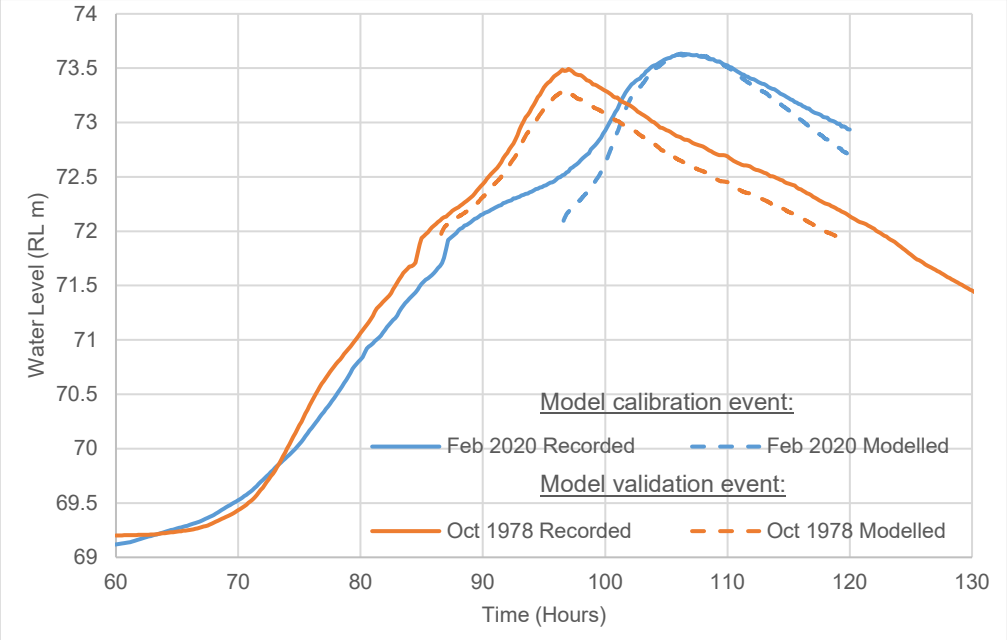
- **Downstream boundary condition:** normal depth with channel slope = 0.00081.
- **Manning's *n*:** channel=0.031; floodplain=0.036. Calibrated from February 2020 flood
- **Model runs:**
    1. During construction: 1/50 AEP without climate change with post-bridge topography.
    2. SLS 2: 1/50 AEP with climate change, pre and post-bridge topography.
    3. ULS: 1/500 AEP with climate change, pre and post-bridge topography.

Figure 5: Hydraulic Model Extents



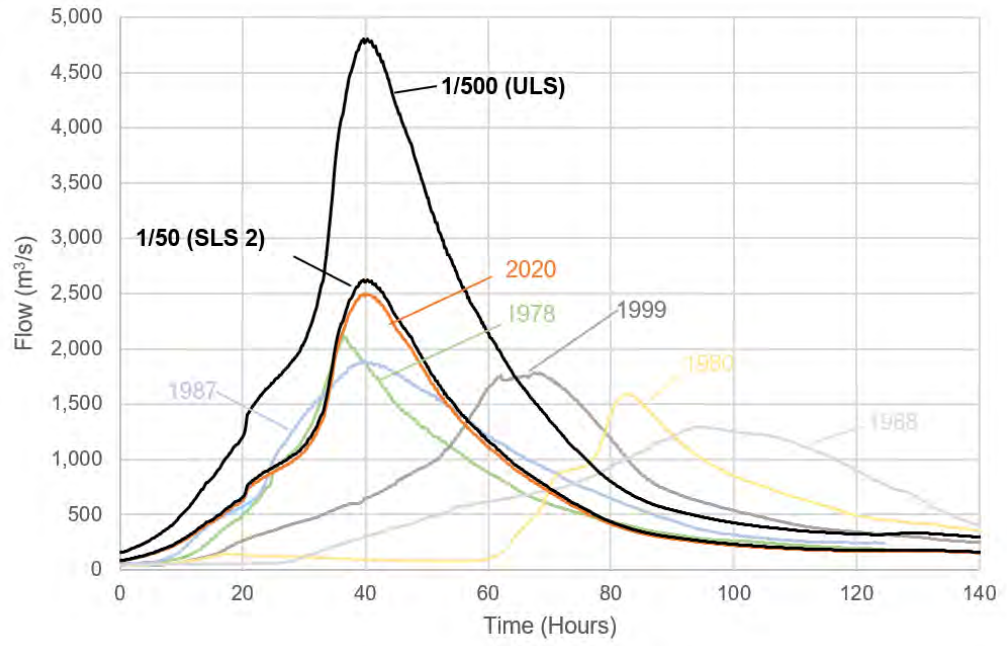
On 5 February 2020, the Mataura River experienced its largest flood of record peaking at 2,450m^3/s at a maximum water level of RL 73.63m at the Gore gauging station. This compares to the previous flood of record being the October 1978 peaking at 2,110m^3/s at a maximum water level RL 73.49m. The February 2020 event was used to calibrate surface roughness values in the hydraulic model. Manning's values for both the main channel and floodplain were varied until a good fit was obtained. The resulting manning's values were n = 0.031 and n = 0.036 for the main channel and floodplain, respectively, which closely matches recommendations by Colin Young at ES and is consistent with what would be expected in other New Zealand rivers of this nature. Predicted peak water levels were within 5% for the October 1978 flood, which was considered acceptable, thereby validating the hydraulic model (Figure 6).

Figure 6: Measured vs. Predicted Water Levels at the Gore Gauging



Design inflow hydrographs were then developed by scaling the February 2020 flood (Figure 7).

Figure 7: Design Inflow Hydrographs Compared to Historic Floods



4.2 Results

Modelled results are presented in Figure 8 and summarised in Table 2. The model predicts elevated water levels and velocities (higher Froude number) in the main channel, with an associated superelevation of the water surface (Figure 8). These results agree with historic photographs illustrating standing waves at this location during the October 1978 flood. Resultant velocities are consistent with, or slightly higher than, previous estimates.

Figure 8: Predicted flood levels

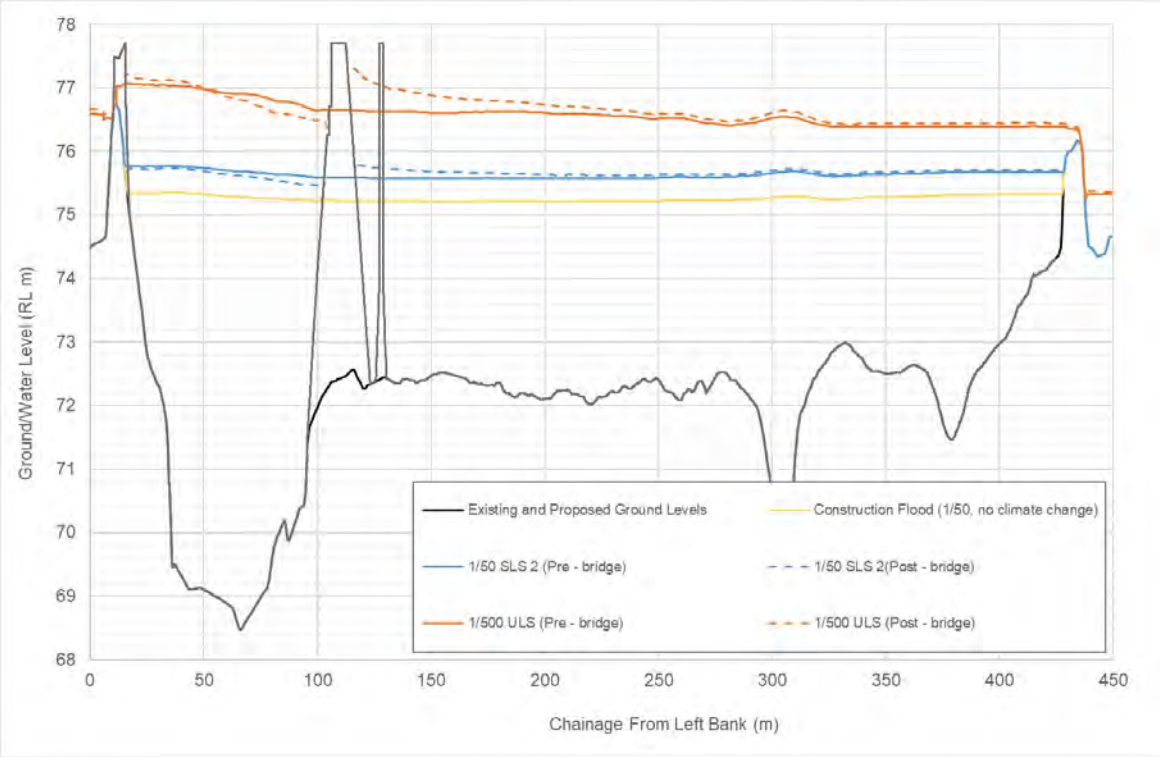


Table 2: Summary of Results from Hydraulic Model

Event	Flow (m^3/s)	Maximum Flood level (RL m)		Comments
		Pre	Post	
1/50 without climate change	2,280	75.40	-	• Similar magnitude flood to October 1978 flood. • Velocities up to 5.5m/s in main channel, 1.5m to 2.0m/s on floodplain (at bridge, less upstream) • Froude number about 0.7 in main channel and 0.4 on floodplain.
SLS 2 1/50	2,612	75.70	75.80	• Similar magnitude flood to February 2020 flood. • Velocities up to 5.8m/s in main channel, 2.0m to 2.5m/s on floodplain (at bridge, less upstream). • Froude number about 0.7 in main channel, 0.4 on floodplain. • Minimal freeboard at stopbanks.
ULS 1/500	4,802	77.05[1]	77.20[1]	• Velocities up to 6.8m/s in main channel, 2.0m to 4.5m/s on floodplain (at bridge, less upstream). • Overtopping of stopbanks on both sides of between 0.3m to 0.5m predicted, likely to cause failure.

Note 1: Overtopping of stopbanks predicted

We understand that the stopbank on the true left was reconstructed and raised in 1986 to the 1978 flood level plus 0.5m freeboard (RL 76.7m). This puts it at least 0.5m higher than the stopbank on the true right, i.e. RL 76.0m (varies). These stopbanks would normally provide an upper limit on the maximum flood level for the bridge. However, it is possible that future raising of the stopbank, at least on the true right side or more likely both in conjunction with hydraulic modelling, would occur within the design life of the bridge. Whilst the design standard for stopbank raising is unknown, a range of 1/100 and 1/200 AEP is likely in keeping with other flood protection schemes across New Zealand within built up areas. Without raising, we would anticipate that stopbanks of this nature and age could sustain minor overtopping, say in the order of 0.2m and only for a few hours if in good condition, after which they could fail and reduce flood levels at the bridge.

Table 2.1 of the Bridge Manual recommends the bridge level be set based on the SLS 2 event, plus freeboard, and that the structure be capable of withstanding the ULS event without collapse. A freeboard of 1.2m is considered appropriate for this bridge in view of the potential for large debris, including entire willow trees as recorded in historic floods, to be carried down the river. Further, we understand that the Pyramid Bridge, located 28km upstream and also on the Mataura River, suffered extensive damage/complete failure as a direct result of debris raft impacting the bridge structure in only a modest flood event.

The preliminary bridge design was based on a soffit level of RL 76.71m, taken as the observed October 1978 flood level plus 1.2m freeboard. Our updated analysis indicates that a soffit level of RL 77.00m on the eastern (left) abutment and RL 76.70m on the west (right) abutment would satisfy the Bridge Manual criteria. Adopting this level would at worst put the soffit 0.10m below the ULS flood level (assuming the stopbank can sustain ongoing overtopping without failure), or more likely, still provide nominal freeboard if the stopbank crest (76.7m on the left bank and 76.1m on the right bank) is taken as the upper limit.

The bridge structure and approach earthworks are seen to have a negligible influence (<1%) on overall flood levels, which is well within the accuracy of the model itself. This not surprising given their total cross-sectional area is only about 3% to 4% of the total floodplain area.

Figure 9: Predicted Flood Levels and Velocities for ULS Event (velocity on secondary axis)

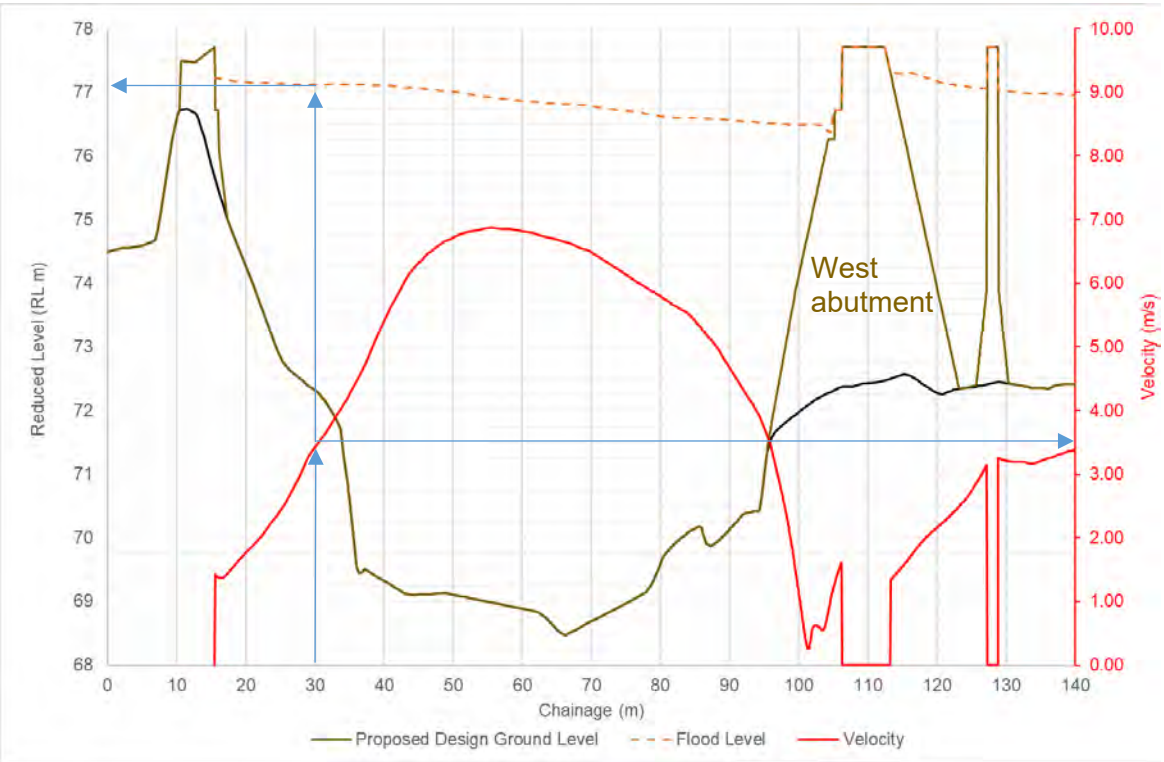
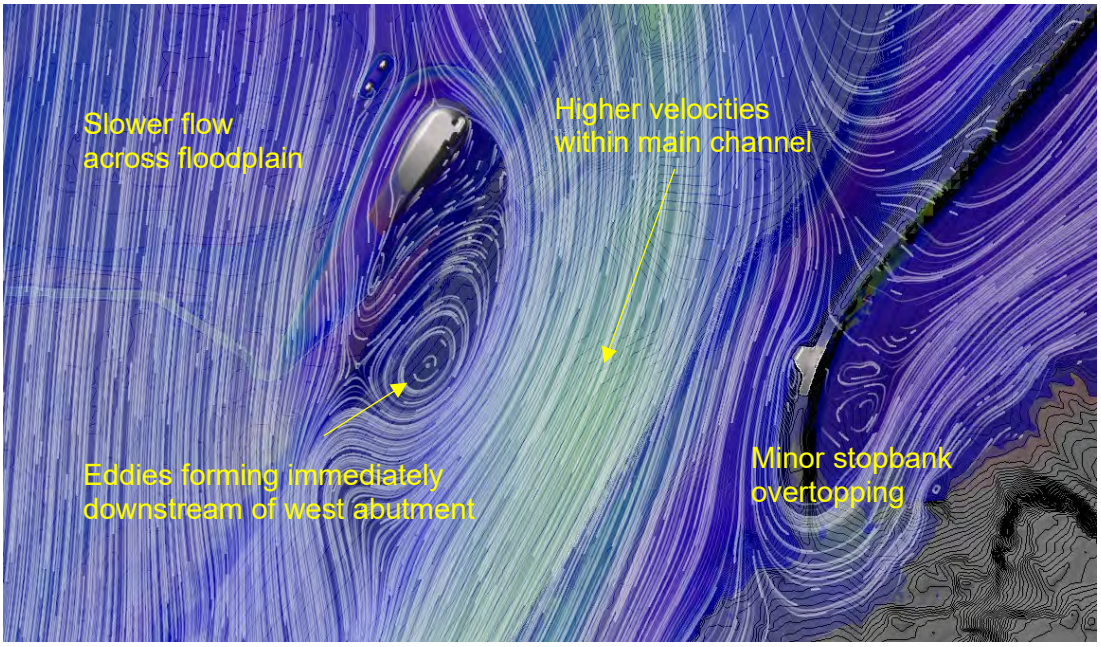


Figure 9 illustrates the maximum flood depth and velocities along the bridge alignment in the ULS event, which can be used for estimating velocity and debris impact loading on the bridge during the ULS event. An example of how to read this figure is shown at chainage (CH) 30m (blue arrows), which indicates a flow depth of 4.8m (RL 77.00m to RL 72.20m) and velocity of 3.5m/s. This same velocity is observed at approximate CH 95m, which is coincident with the toe of the west abutment (velocities on the opposite/western side are generally less than 3.0m/s). Velocities as high as 7m/s and corresponding Froude numbers of 0.8 are estimated in the middle of the main channel.

The concrete surround protecting the back-stay anchor cables will also experience inundation and high-velocity flow during design floods. It is recommended that this element be designed to withstand flood depths of up to around 5m at velocities up to 3.5m/s.

The model also predicts local variations in the flow regime in plan around the west abutment, resulting in varying velocity and depth profiles, and the formation of eddies on the downstream side (Figure 10).

Figure 10: Flow Vectors in Plan Around West Abutment



4.3 Updates to Include 5 February 2020 Event

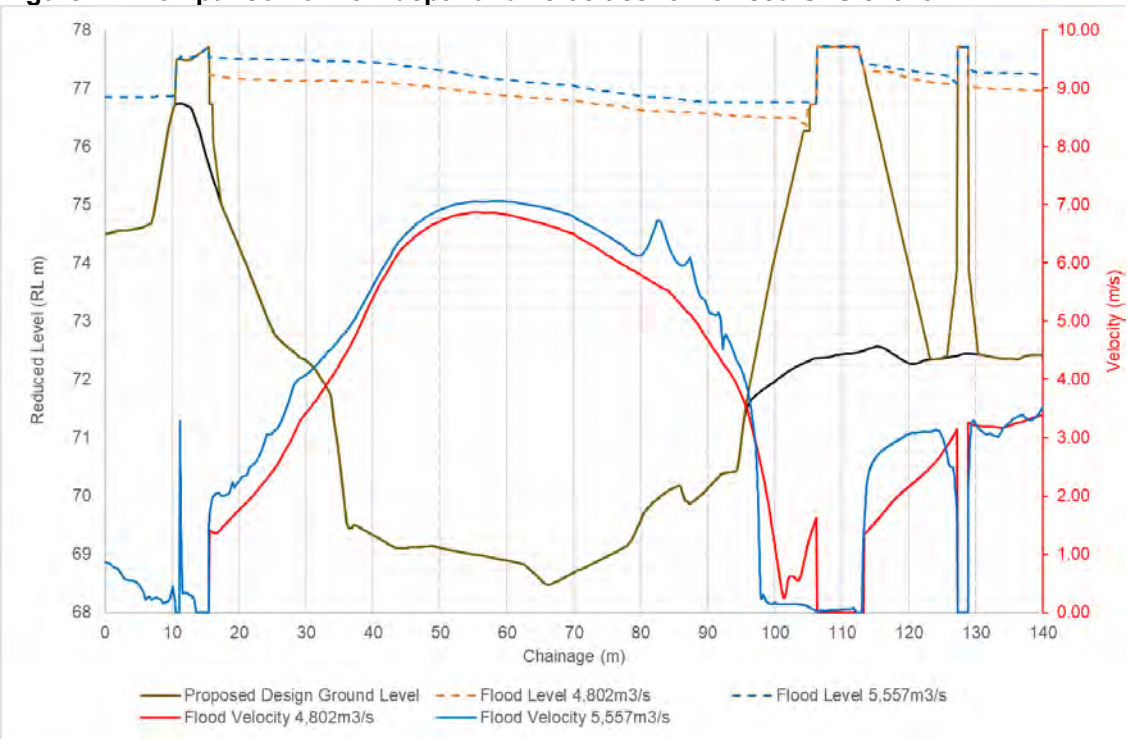
As outlined in Section 4.1, the Mataura River experienced its largest flood of record on 5 February 2020. Our hydraulic model was calibrated to this event based on preliminary flow and depth information available at the time. Subsequently, ES have downgraded their peak flow estimate from 2,450m^3/s to 2,399m^3/s.

Following discussions with our peer reviewer, we have reviewed the flood frequency analysis and rerun our hydraulic model considering the updated information. Applying a GEV EV1 distribution to the updated dataset results in a peak flow estimate of 4,833m^3/s which closely matches the adopted peak flow of 4,802m^3/s. However, the revised estimate does not include an allowance for climate change, which would be 5,557m^3/s including 15% for climate change.

Rerunning the hydraulic model with a flow of 5,557m^3/s results in similar flood levels and velocities to those previously predicted (Figure 11). This suggests that flood levels and velocities during the ULS flood event will be generally insensitive to the adopted flowrate, which is not unexpected given that the channel is flowing bank-full for both flowrates.

Given that the maximum flood level is governed by the adjacent stopbank height, and any future raising being unlikely to accommodate a 1/500 flood as previously outlined, the adopted depths and velocities are still considered suitable for design of the bridge.

Figure 11: Comparison of flow depth and velocities for revised ULS event



4.4 Scour and Erosion Countermeasures

The foundation piles supporting the bridge superstructure will be embedded many meters below present ground level and into competent rock. They are, therefore, not considered to be at risk from scour.

However, the bridge abutments, in particular the west approach embankment, will require protection to minimise erosion-related scour in future design flood events. These protection measures will need to be robust owing to the high velocities and flow depths predicted by the hydraulic model. The following countermeasures have been adopted for the bridge design:

1. Aligning the approach earthworks with the general flow direction.
2. Profiling/shaping the embankment into a teardrop shape.
3. Providing an erosion-resistant surface for the inundated area.

Riprap has been sized using several methods, including those outlined in Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 2 (HEC23, 2009) and CIRIA 551 Manual on scour at bridges and other hydraulic structures (CIRIA, 2002). As illustrated in Figure 9, flow depth and velocities vary considerably across the embankment. It is therefore proposed to install robust riprap, designed to the maximum water depth and velocity, at the toe of the embankment and reduce the size progressively up the embankment. Applying a design depth of 5.6m and velocity of 3.5m/s (Figure 9), we estimate a D_{50}~0.9m or Riprap Class IX is required. Smaller riprap and reinforced grass are proposed further up the embankment where depths and velocities are less, as shown in Table 3.

Table 3: Riprap sizing as a function of water depth

Location	Flow Depth (m)	Velocity (m/s)	Protection
Embankment Toe (RL 72.4m to 74.0m)	5.0	3.5	Riprap Per Table 4
74.0m to 76.3m	3.0	2.0	Riprap with d_{50}~0.4m
76.3m to 78.3	1.0	<2.0	Reinforced grass (Cirtex ECC3 or similar)

The following specifications should apply to the riprap at the toe of the embankment.

i. Grading consistent with the following table:

Table 4: Envelope for Class IX Riprap

	Min (mm)	Max (mm)
d_{15}	560	800
d_{50}	860	1,050
d_{85}	1,200	1,400
d_{100}	-	1,800

ii. Minimum thickness of 1.8m

iii. Underlain by either a 0.8m thick layer of D_{50}~0.4m well-graded gravel (preferred) or heavy-duty geotextile

iv. Toe apron to be provided as practical on site.

v. Rock to be hard, durable, angular rock with specific gravity Sg>2.65

The available quantity of rock this size will depend on nearby quarries, although we are aware of similar rock being utilised for river protection and revetments across Southland.

Heavy-duty turf reinforcement mat (such as Cirtex ECC3 or similar), with their proprietary system of driven anchors spaced at 1m vertical and horizontal centres (if required), is proposed for the upper section of the embankment. These systems generally perform best where vegetation can be established and maintained over the long-term, and when flood durations are sufficiently small to avoid killing the grass. Careful detailing and a robust installation methodology will be required to ensure a robust system. This same system can be applied locally around the eastern abutment, as full riprap is not considered to be warranted.

5.0 Subsurface Conditions

The following sections report presents an interpretation and assessment of ground conditions at the bridge site based on previous geotechnical investigations and provides geotechnical parameters for detailed design of the bridge.

5.1 Geotechnical Investigations

Insight Engineering undertook a geotechnical subsurface investigation and produced a factual report for the bridge site in August 2019. Their scope involved a desktop review of available information; geotechnical site walkover; commissioning, supervision and logging of six sonic boreholes to depths ranging from 4.2m to 9.0m; supplementary Scala penetrometer tests; and preparation of a factual geotechnical report.

Retrieved samples were typically logged as sandy silt, fine sand, and sandy gravel (Alluvium) overlying sandstone and siltstone (North Range Group). BH5 and BH6 also encountered silty sand and gravelly silt (Embankment Fill). Standard penetration tests (SPTs) recorded uncorrected N values of 10 to 50, i.e. medium dense to dense conditions. Rock fragments were recovered as slightly to moderately weathered, strong to very strong, with closely spaced joints and occasional sheared/shattered zones.

Localised rock outcrops were observed along the river, which is described as steeply dipping siltstone and sandstone with closely to very closely spaced joints.

Static groundwater levels of between 1.3m and 2.1m were recorded following drilling, and these are likely to be coincident with levels in the Mataura River.

5.2 Geological Setting

The regional geological setting of the site comprises an interbedded sequence of quaternary fluvial sediments eroded from the inland schist basement, overlying weakly metamorphosed sandstone (greywacke) and siltstone (argillite) of the North Range Group (Murihiku Terrane).

The GNS Science 1:250,000-scale QMAP and accompanying bulletin for the area (Map 20 Murihiku Area 2003) indicates the site is underlain by modern to postglacial floodplain deposits (Q1a) consisting of unconsolidated gravel, sand, silt and clay, and minor peat. These alluvial deposits are underlain by the North Range Group, which consists of well-bedded siltstone and fine sandstone. The North Range Group rock unit dips 60° to 70° to the south-west and trends approximately north-west to south-east, where it is mapped adjacent to the alluvium near the site.

5.3 Seismicity

For the purposes of structural design, the site is assessed as subsoil category Class B – Rock, in accordance with NZS1170 – Structural Design Actions, based on the shallow depth to rock observed in the boreholes and the fact that the bridge will be supported on a series of bored concrete piles socketed into the rock. For the purposes of liquefaction assessment, we have also considered the site as Class C – Shallow Soil, owing to the thickness of embankment fill observed.

No active faults requiring specific consideration for design are mapped within 20km of the site. However, two active faults, the Blue Mountain No. 1 Fault and Spylaw Fault, are mapped within 30km of the site, along with several unnamed inactive faults, mapped by GNS Science.

5.4 Stratigraphy

A simplified geological model was developed for the site for the purposes of evaluating foundation conditions and the implications for design. The model comprises embankment/stopbank fill where present, overlying a thin mantle of alluvium overlying basement rock:

- **Embankment Fill** consists of a layer of medium dense, Silty SAND overlying very stiff gravelly SILT. Uncorrected SPT's of between 16 to 22 were recorded in the gravelly SILT. No testing was undertaken on the sand.
- **Paleosoil or possible buried topsoil** was identified below the embankment fill in BH6 only.
- **Alluvium** was encountered below the topsoil on the west bank and below the fill on the eastern embankment. This layer is typically described as soft to firm, low plasticity, sandy SILT and medium dense, poorly graded, fine SAND with some Silt, overlying medium dense to dense, sub-rounded, sandy GRAVEL.
- **North Range Group Siltstone and Sandstone** is present across the site below the alluvial sediments on the banks and is present on the riverbed. Sandstone and siltstone layers were recorded as interbedded and typically slightly to moderately weathered, and moderately strong. Rock fragments were recovered as slightly to moderately weathered, strong to very strong, with extremely closely to closely spaced joints and occasional sheared/shattered zones. A zone of completely weathered to highly weathered rock is indicated below the eastern embankment in BH6. SPTs recorded uncorrected N values of 50+ to bouncing, i.e. medium dense to dense conditions.

Groundwater is anticipated to lie at or about the adjacent Mataura River level.

6.0 Liquefaction Analysis

6.1 Introduction

A principal consideration for seismic analysis of bridge structures is the potential for liquefaction and associated consequences (e.g. lateral spreading). This is because bridge sites often contain loose alluvial sediments and high groundwater tables.

In their factual report, Insight Engineering indicate that the western side of the site is mapped as having medium potential liquefaction risk based on work by GNS Science[3].

A liquefaction analysis was, therefore, performed on the borehole logs using the Boulanger & Idriss (2014) deterministic method. This method is recommended by the Ministry of Business, Innovation and Employment (MBIE) and incorporates a number of case histories from the 2010 and 2011 Christchurch earthquake events. It should be appreciated that liquefaction assessments using SPTs alone are less reliable when compared to other methods such as cone penetration tests (CPTs).

Groundwater levels adopted in our analysis were based on measurements during drilling, which coincide with normal river level. A sensitivity analysis assuming groundwater at the surface (flood conditions) was also undertaken.

[3] GNS Science. (Jan 2012). Amplified ground shaking and liquefaction susceptibility, Invercargill City

6.2 Design Ground Motions

Two earthquakes scenarios were assessed in accordance with NZS1170 – Structural Design Actions, for an Importance Level 2 structure with a 100-year design life.

Peak horizontal ground accelerations and effective magnitudes were calculated using the procedure from the NZTA Bridge Manual. Table 5 summarises the scenarios considered. Estimates are presented for both Class B and Class C, both of which were considered in our assessment. We recommend that the bridge itself be assessed assuming Site Class C conditions, owing to the depth to rock and embankment fill on both abutments.

Table 5: Peak Ground Acceleration Values for Geotechnical Assessment

Site Class	SLS	ULS
Class B – Rock		
Moment Magnitude (M_w)	6.03	6.13
Annual Probability of Exceedance	1/25	1/500
Peak Ground Acceleration	0.05g	0.19g
Class C – Shallow Soil		
Moment Magnitude (M_w)	6.03	6.13
Annual Probability of Exceedance	1/25	1/500
Peak Ground Acceleration	0.06g	0.26g

6.3 Results

No significant liquefaction is predicted for either the SLS or ULS event based on the SPT results, even with the groundwater at the surface.

Notwithstanding that, we have undertaken a close review of potentially susceptible layers from the borehole descriptions and have identified pockets of fine sand with some silt, and silty fine sand. These layers are thickest in BH1, BH4, and BH5. Uncorrected SPT N values were around 10 to15 where recorded and descriptions suggest fines content in the order of 20%.

Assuming the soils are fully saturated, the lowest SPT N=10, a fines content of 20% and a contributing thickness of between 1.5m to 3.0m, results in ULS free-field settlements of between 5mm and 10mm. In reality, we note that these are above the normal river level and hence, a flood and earthquake would need to coincide for these to present an issue.

Notwithstanding that, a conservative geotechnical pile design has assumed no contribution in skin friction from any of the soil units, and instead relies on end bearing within the rock socket. Further, a slope stability scenario was modelled assuming fully liquefied parameters in the underlying sands. Despite this presenting an extreme worst-case scenario, the Factor of Safety (FoS) against stability was around 1.0, suggesting only minor movement is possible.

7.0 Geotechnical Parameters

Table 6 provides a summary of the recommended geotechnical design parameters for the primary geological units at the bridge site.

Table 6: Recommended Geotechnical Design Parameters

Unit	Bulk Density γ (kN/m³)	Effective Cohesion c'(kPa)	Effective Friction, Φ'(°)	Undrained Shear Strength Su (kPa)	Elastic Modulus E (MPa)	Poisson's Ratio v
Engineered Fill*	20	0	35	-	25	0.25
Existing Stopbank Fill	18	1	32	-	9	0.3
Alluvium (silt and sand)	16-18	0-2	28-30	0-25	4-14	0.3
Alluvium (sandy Gravel)	20	0	38	-	38	0.3
North Range Group	22	Hoek-Brown parameters: UCS=20MPa, GSI=40, mi=7-17, D=0			78	0.15

*Earth pressure coefficients for engineered fill as follows: K_0=0.4; K_a=0.2; K_p=5.9

8.0 Foundation Considerations

8.1 Bridge Piles

The bridge superstructure will be supported on a series of 0.6m diameter bored concrete piles, socketed into competent North Range Group siltstone or sandstone rock at both abutments.

8.1.1 Axial Capacity

Ultimate geotechnical capacities of individual piles were assessed using the method for rock socket pile design by Sinclair and Every (2006)[4]. Pile lengths were estimated based on a minimum socket length of one pile diameter (1D) into competent rock, to be verified during construction. Design geotechnical capacities include a strength reduction factor Φ = 0.5, in accordance with Module 4: Earthquake resistance foundation design (MBIE, 2016). A check was also carried out in accordance with B1/VM4, assuming dense gravel parameters.

Table 7: Ultimate and Allowable Pile Capacities

	Location	
	Supporting Bridge Mast	**Supporting East Abutment**
Pile type	0.6m diameter, 7m long[1], reinforced concrete	0.6m diameter, 6m long[1], reinforced concrete
Toe RL	RL 68.54m	RL 69.6m
Ultimate skin friction[2] (kN)	280	280
Ultimate end bearing[2] (kN)	1,540	1,540
Ultimate geotechnical capacity[2] (kN)	1,820	1,820
Allowable geotechnical capacity in tension[3] (kN)	210	207

Notes.
1. Lengths as measured from the underside of the capping beam
2. Values shown include a strength reduction factor Φ = 0.5
3. Geotechnical capacity in tension estimated as 2/3 the positive skin friction with strength reduction factor plus the weight of the pile

[4] Sinclair & Every (2006) *Designing for Rock Socket Piles. Earthquakes and Urban Development pg. 341-351*

Structural modelling indicates an average factored ULS pile load beneath the mast is 1,099kN in compression, and the maximum on a single pile 1,225kN, i.e. the bridge can be fully supported on end bearing alone with skin friction contributing ~15% of the overall capacity.

Only one pile develops in tension, at eastern abutment under flood conditions. The factored load is 85kN (tension) which is much less than the 207kN allowable geotechnical capacity in tension, noting that this is estimated ignoring all skin friction in the soil, includes a reduction of 2/3 of the positive skin friction in the rock socket, and then a SRF=0.5.

8.1.2 Lateral Springs

Bi-linear elastic plastic soil springs have been developed based on the borehole results. Three methods were analysed including methods proposed by Cubrinovski (2009), Terzaghi (1955) and others. A range of 1/3rd to 3 times is presented for sensitivity modelling.

The potential influence of adjacent slopes in generating passive earth pressures/lateral soil restraint in the piles is outlined in Section 8.1.4.

Table 8: Lateral Soil Springs for Static and Seismic Conditions

Meters below capping beam	Mast		East Abutment	
	K Range (kN/m)	Limiting Force (kN)	K Range (kN/m)	Limiting Force (kN)
1	6,000-55,000	110	3,700-34,000	90
2	6,000-55,000	315	3,700-34,000	250
3	6,000-55,000	500[1]	1,300-12,000	135[1]
4	6,000-55,000	500	4,500-40,000	135
5	1,300-12,000	500	18,000-160,000	135
6	9,500-86,000	500	18,000-160,000	135
7	18,000-160,000	500	18,000-160,000	135

Notes.
1. Maximum limiting force based on the results from slope stability analysis

A check on the springs was made in RSPile, which returned similar displacements for dummy lateral load of 100kN at an indicative fixity of ~3m (5D).

No degradation is required for seismic conditions as no liquefaction or other strength loss is predicted.

The above spring values are for individual piles and adjustments should be made for group effects. The interaction effects of the pile group, due to the displacement field (i.e. shielding) generated under lateral loading, should be accounted for in the design. Factors ranging between 100% for a spacing of 8D+ to 25% for 3D have been suggested to account for this, which is conservative in view that Cubrinovski (2009) recommends ignoring such effects for spacings of 6D and greater.

Springs connected to the abutment/pile capping beams can be modelled as a passive spring generated at a displacement of 2.5% of the height. A strength reduction factor of 0.5 is recommended for the maximum lateral earth pressure in accordance with NZGS/MBIE Guidelines.

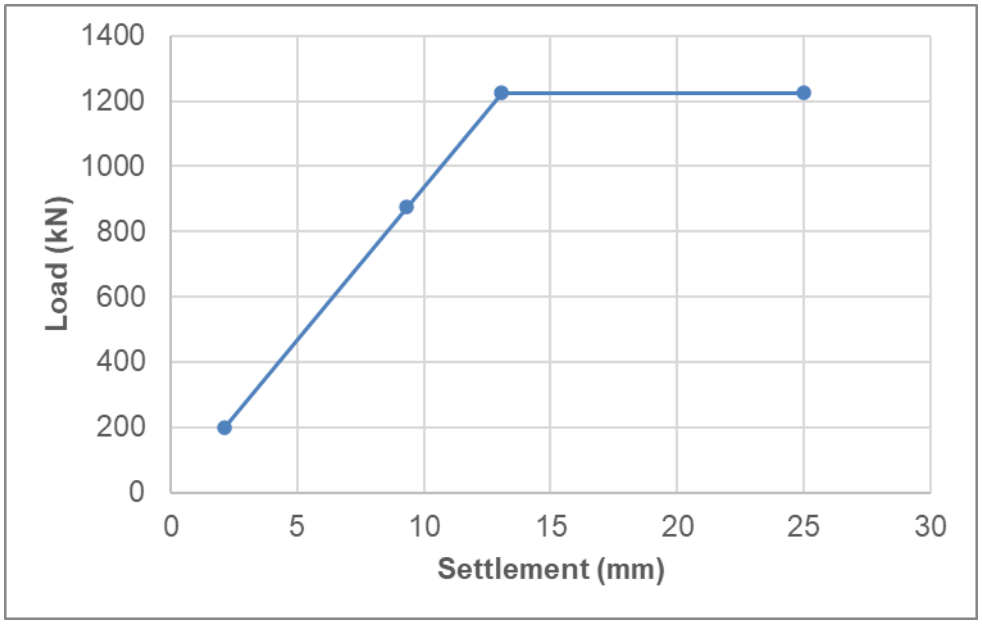
8.1.3 Vertical Springs

Vertical springs were also estimated to be input into the structural model, refer to Table 8. Settlements were estimated by the Sinclair and Every (2006) method with a check using Flemming (1992), with the piles estimated near their allowable/working load. Figure 12 illustrates the bi-linear spring model adopted up to the design ultimate axial capacities listed in Table 7.

Table 8: Vertical Soil Springs

	K (kN/m)
Vertical spring	94,000

Figure 12: Vertical Spring Model



The suspected buried topsoil layer identified beneath the stopbank (eastern abutment) has the potential to settle during the design life of the bridge. However, this has been in place for at least 34 years (since the stopbank was raised in 1986) and likely much longer, and both primary and secondary consolidation due to the stopbank should, therefore, be completed. The net additional load imposed on this layer from the approach works will only be minor, particularly considering the bridge and abutment beam are pile-supported.

8.1.4 Slope Stability

We have performed slope stability analyses in Slide v8.0 to verify the stability of both abutments under the range of expected normal (static) and extreme load cases (flood and earthquake), and hence any requirements to degrade or remove lateral resistance from the piles.

<u>West embankment:</u>

The west embankment will be constructed of imported granular fill compacted to a high standard and includes large diameter riprap which will act as a buttress. The bridge loads generally have a have a restorative/beneficial effect against instability that would impact the piles, i.e. towards the river.

Under static conditions with the maximum lateral loads applied from the bridge, we estimate a minimum FoS of 1.5, which is considered acceptable. The failure surface modelled is a very shallow failure developed parallel with the slope angle. This does not intersect the pile or passive wedge, and on that basis no reduction in the passive wedge, nor additional load, is warranted. However, if we conservatively project the base of the passive wedge at an angle of 45+Φ/2 from the pile, it intersects the outer embankment slope beneath around 3m. It is therefore reasonable to reduce the limiting force outlined in Table 7 to say the maximum value at 3m, i.e. 500kN for beneath the Mast and 135kN beneath the east abutment. This would be conservative, as in reality the full resistance should be available beyond around 8D (i.e. 4.8m) or less as discussed in Section 8.1.2.

Under rapid drawdown conditions (post-flood), a FoS of 1.4 is estimated, which is expected given the cohesionless, free-draining fill proposed.

Under an SLS earthquake, a FoS of 1.3 is estimated which is considered acceptable. However, under a ULS earthquake a FoS just under 1.0 is predicted towards the river/bridge (east), thereby indicating that slope deformation is possible. Applying the methods of Jibson, Ambrasseys and menu and Bray and Travasarou we estimate coseismic deformations will be order of up to say 20-40mm for the slope in front of the piles. No failures intersect the pile nor passive wedge and is therefore unlikely to impact or threaten the bridge. A FoS of 1.0 is also predicted assuming liquefied parameters and no seismic loading (i.e. post-earthquake assuming liquefied parameters).

East embankment:

This will be formed into the existing stopbank, which may not be as robust as the west embankment.

Under static conditions the current FoS without the bridge in place is estimated at ~1.3 which is less than the target of 1.5. However, the east abutment wall/pile cap has been designed to resist at-rest earth pressures which have a restorative/beneficial effect on stability. Including a force of 15KN/m to the wall in the model results in a FoS of 1.75. The failure surface intersects the pile at ~0.5m, and we understand that the soil springs to 1m has been switched off and therefore any passive resistance above this ignored.

Under rapid drawdown conditions (post-flood), a FoS of 1.3 is estimated, which reflects the generally good performance of the stopbank since its reconstruction.

Factors of safety under SLS and ULS earthquake are estimated at >1.5 and 1.0 respectively, which are considered acceptable. A FoS of 1.0 is also predicted assuming liquefied parameters and no seismic loading (i.e. post-earthquake assuming liquefied parameters).

The Insight Engineering report indicates some local instability in the existing stream banks. Preparations for constructing the western embankment will include tidying up local instability features that may impact the embankment. It is considered that these can be inspected, and the extent of remediation/removal specified during construction.

8.2 Anchors

Anchorages for the mast backspan stay-cables will be provided by a series of grouted bar anchors. One anchor will be provided to each back-stay cable to enable cables to be readily stressed, i.e. six in total. Analysis by DC Structures Studio indicate ultimate (factored) structural loads of up to 560kN per anchor.

We estimate that the underlying rock will provide an ultimate bond strength of in excess of 400KPa, based on geotechnical parameters derived in RocLab and from experience with anchoring into similar rock types. This suggests that 7m long, 0.15m diameter anchor, with 42mm Williams 150 KSI bar will have a design geotechnical capacity of ~660kN, inclusive of a strength reduction factor Φ=0.5. It is recommended that a minimum 4m free length be provided based on the depth to rock from BH3. This results in 11m long anchors. Anchors are to be installed with Class I (double-corrosion) protection.

Comparison with average ultimate bond stresses in Table 7 of the FHWA[5] indicate these are reasonable, if not conservative. Presumptive ultimate values of load transfer, presented in Table 8 of FHWA for sandstones, indicate a design geotechnical capacity of around 145kN/m, corresponding to anchor lengths of about 5m.

Anchor groups are spaced 400mm (<3D) apart given site constraints, which is closer than normal applications (10D+). Group effects have, therefore, been assessed using Eq. 5-17 of FHWA NHI-05-039 for uplift capacity of micropile groups in cohesionless soil. Results indicate a total soil/rock weight of 2,914kN (971kN per anchor) for each group, which compares to the maximum single cable load of 560kN. The uplifted blocks between the deadman anchor groups are spaced 12m apart, which means that uplifted blocks do not intersect.

It is recommended that at least one trial anchor be installed prior to production anchoring to confirm the design parameters and installation methodology to be used. It is recommended that the trial anchor be drilled to the full depth (11m) and the test interval specified based on the drilling results (so that a representative or conservative section can be tested). All production anchors should be tested and locked off to a specified load to be confirmed during detailed design.

Each set of three anchors will be housed within a concrete surround above ground to provide protection against flood and debris loading from the Mataura River. Horizontal forces can be resisted by a combination of passive soil resistance generated on the leading face, by the concrete foundation friction and by self-weight. Earth pressure coefficients can be estimated using the geotechnical parameters in Table 5 and a strength reduction factor of between 0.45 to 0.6. The generation of passive resistance will require the concrete surround to deflect, and we understand this movement can be accommodated for within the free length cavity. Friction coefficients against sliding for the concrete surround will depend on the soil, with a value of 0.5 recommended for granular engineered fill. Additional restraint can be provided via deepening foundation keys and/or with the addition of shear piles as required.

9.0 Earthworks

No major cut earthworks are proposed for the project, although some minor profiling may be required to prepare foundations for earthworks and/or to form the east abutment into the stopbank. Fill earthworks are required to form the west approach embankment and this should be constructed to a high standard. All fill should be placed and compacted in accordance with the recommendations of NZS 4431:1989 with a target compaction of 95% of MDD or other nominated by the engineer. As with any earthworks projects, the expectation is that:

1. All topsoil, organic matter, and other unsuitable materials is removed from the construction areas prior to fill placement.
2. An appropriate compaction methodology is established early on by way of supervised plateau testing.

[5] Federal Highway Administration 1999 Geotechnical Engineering Circular No.5 – Ground Anchors and Anchored Systems FHWA-IF-99-015

3. Compaction levels are verified by routine quality assurance/quality control (QA/QC) testing (using nuclear density meters or other approved methods).
4. RILEY has the opportunity to attend routine and critical inspections during construction, and review QA/QC results as they are available.

Robust sediment control measures, including provisions for surface and groundwater management, with a specific emergency management plan for incoming floods, should be considered and instigated during bridge construction.

10.0 Limitation

This report has been prepared solely for the benefit of DC Structure Studio Ltd as our client with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

Recommendations and opinions in this report are based on data from limited test positions. The nature and continuity of subsoil conditions away from the test positions are inferred, and it must be appreciated that actual conditions could vary considerably from the assumed model.

During excavation and construction, the site should be examined by an engineer or engineering geologist competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. It is possible that the nature of the exposed subsoils may require further investigation and the modification of the design based upon this report.

Riley Consultants Ltd would be pleased to provide this service to Gore District Council and believes the project would benefit from such continuity. In any event, it is essential Riley Consultants Ltd is contacted if there is any variation in subsoil conditions from those described in the report as it may affect the design parameters recommended in the report.

APPENDIX A

Supplied Consultant Reports

Gore District Council

Request for Proposal

Longford Shared Path Design Contract - Request for Proposal Contract No. 2019/11

Appendix B – Preliminary Engineering and Geotechnical Report

Gore District Council
29 Bowler Avenue
Gore 9710

Appendix B – Preliminary Engineering Report

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1.0 Summary

The purpose of this report is to accompany both the LandPro Preliminary Hydraulic Report and the Factual Geotechnical Report produced by Insight Engineering dated August 2019 on the site of the proposed new Longford Track Cycle Bridge and to provide some bridge-related commentary to the possible design options and restrictions to same.

This report should be taken as a preliminary finding and are a basis for the information available. It is essential that an appropriately qualified and experienced team of engineers, with the appropriate skills, be engaged to undertake a detailed analysis of the site at the final bridge design stage.

2.0 Site Location

The site is situated on the Mataura River in Gore approximately 650m upstream of the State Highway 1 bridge and is intended to be a combined cycle and pedestrian link from Gore to the east and the more open land to the west.



Site Location Plan of the river crossing (from the Factual Geotechnical Report)

The site is located at the western end of Church Street to cross the Mataura River in East Gore to a mainly pastoral area on the west bank.

3.0 The Project

The Council adopted the Southland Cycle Strategy (2016-26) in 2018. The strategy intends to promote alternative modes of travel, to unlock and strengthen social

connections through the uptake of cycling. The main barriers to increasing the uptake of cycling in Gore are severance and safety. This project is to address the issue of severance, being that of the Mataura River.

Many similar projects have been completed around New Zealand, and council wishes that this project will cumulate in an iconic structure to allow full development on the western side of the Mataura River. This link is the most important part of the proposed network, providing a connection between East Gore and businesses, schools, shopping and sporting facilities in Gore. This central link, along with a new river crossing, will lead to a significant step-change in the provision of safe walking and cycling connections in the town.

Completing the primary cycle route aligns closely with the Government Policy Statement (GPS) for Land Transport. The recommended option includes a new 3m wide, single-span bridge pedestrian/cycle bridge crossing of the Mataura River on the extended Church Street alignment with the associated cycling track.

The Council are looking for a suitably qualified supplier to deliver a unique and special design as a showcase for its design acumen. Designing a single span bridge and shared path (track) with associated landscaping for the use of pedestrian and cyclist.

4.0 Project Scope

From the Request for Proposal (RFP) the following project scope includes the following but not limited to:

- Undertaking the preliminary and detailed design for the Longford Shared Path as the centre link of the cycling network in Gore and producing the construction documents;
- A future proof single-span bridge design (based on Bridge Manual – 3rd edition, NZTA) and provision of the structure straightness to cater for future infrastructures such as water and/or telecom services crossing the Mataura River;
- Designing all elements of the pedestrian and cycling track (centre link);
- Designing a cost-effective, value for money, unique and sustainable bridge and cycling track consistent with the local values and rural nature of Gore;
- Undertaking value engineering and preparing a firm and accurate estimate for the construction costs as well as the cost of the asset lifecycle; and
- All required engineering and design processes, including safety in design (SiD).

5.0 Expectations of Council

The expectation of the council is that the successful supplier will be required to undertake preliminary and detailed design for the Longford Shared Path as per the approved architectural impression. The following comprehensive documents shall be provided to the Gore District Council for evaluation of the design by the supplier:

- Architectural design
- Preliminary design report
- Risk analysis and safety in design (SiD)
- Final design report that includes:
    - Site information, such as location and layout.
    - Impact assessment on adjacent properties and services.
    - Value engineering that includes material selection, constructability analysis and updated construction costs.
    - Detailed calculations.
    - Drawings showing location, detailed sections and instructions to the constructor for the bridge, catered services, track and associated and landscaping.
    - Details of provision for future infrastructures.
    - Specification for construction.
    - Schedule of quantities.
    - Estimate for lifecycle costs of the asset.
    - Nominated minimum levels of construction supervision.
    - Environmental assessment and a design compliance statement, referring to the NZTA Bridge Manual – 3rd edition, and the Gore District Council Subdivision and Development Bylaw 2019 and any required dispensations.
    - Fortnightly progress reports.

Please see the RFP for further information on roles, responsibilities, team members, timescales and methodology.

5.0 Gore District Council Information

In addition to the Topographical Survey data, and Preliminary Hydraulic Sensitivity Analysis Report, Council has the following information available:

- Existing Services to this site; http://www.goredc.govt.nz/do-it-online/online-maps/
- Council's Subdivision and Development Bylaws 2019; http://www.goredc.govt.nz/your-council/plans-policies-and-bylaws/bylaws/

6.0 Involvement of Parties

As well as the Gore District Council the following parties also have an involvement:

- Environment Southland (ES) – Responsible for Mataura River flood protection infrastructure and regional resource management authority
- New Zealand Transport Agency (NZTA) – Funding partner for Gore District Council
- Te Ao Mārama Inc. – Affected party for Resource Consents relating to the Mataura River
- Ngāi Tahu – Affected party for Resource Consents relating to the Mataura River
- Fish and Game (FaG) – Affected party for Resource Consents relating to the Mataura River
- Land Information New Zealand (LINZ) – Affected party for Resource Consents relating to the Mataura River
- Department of Conservation (DoC) – Affected party for Resource Consents relating to the Mataura River

7.0 Discussions with Environment Southland (ES)

Several discussions have taken place with ES, mostly with Colin Young, Technical Services Manager who has assisted with historical flooding information, cross-sections and photos and will, of course, be involved in the final design.

ES's main concern is that there is a main flow channel (left bank) and an overflow or flood channel or berm on the right that needs to be kept as open as possible. Any sort of embankment structure protruding into the flood zone would be an issue, and that is the main reason why a longer single span is suggested. The river profile would also lend itself to the suggestion of a second or land span, smaller to the first to allow the bridge to stay above the maximum flood level and only have a single abutment or tower structure within the flood zone. The historical depth of floodwater at maximum flows in the flood zone is approximately 3-3.5m with an estimated velocity of 3m/s.

The flood bank on the eastern side follows the river and the road above, it was constructed out of local materials and has no cohesive materials. With a long flood duration, some piping of water through this berm could be expected.

8.0 Geotechnical Aspects

This area is seen to be one of Pleistocene river deposits overlying basement sedimentary rocks of well-bedded siltstone, sandstone, tuff, shell beds and conglomerates. There is also a small local outcrop deposit of siltstone (Stag Stream siltstone of the North Range Group). Exposed rock can be clearly seen outcropping in the river profile and in mid-stream channel.

GNS Science records from June 2006, states that there has been little seismic activity in this area, with the nearest known fault being some 22km away. Likewise, the main Alpine fault is some 188km away to the northwest. This does, however, present possible problems in that the likelihood of a major Alpine fault rupture affecting the Gore area, has been assessed as 30% over the next 50 years. There are also several un-named inactive faults mapped by GNS Science within approximately 0.5 km to 3.5 km of the site.

There are “sheep creeps” visible in the river bank profiles, indicating local slumping of materials, mainly on the western side of the river.

From the six preliminary boreholes drilled (with a rotary sonic drilling rig), drilled to a depth of between 4.3 and 9.0m depth, comprised a surficial layer of native, loose to dense alluvium overlying weathered, jointed rock. The rock typically comprised slightly to moderately weathered, SANDSTONE and SILTSTONE with extremely closely to very closely spaced joints, which was commonly recovered as angular sand and gravel. The rock is typically strong to very strong, which while offering a good anchorage will prevent any great depth of foundations. See the factual report for more information.

The boreholes were concentrated mainly on the western side with two holes on the eastern side.

9.0 Seismic Design

As a guide and as mentioned above, the seismic activity in this area is very low and as mentioned above, the active faults are from 22km away, so the design of the structure will be mostly gravity or flood governed.

For the lateral restraint and for the transfer of seismic forces it is expected that piles will be needed at the west end, and the depth of piles is expected to be in the order of 4m to 5m, or an alternative anchor-based system. For the east side, the piles will need to go deeper, well below the flood bank base, and will terminate at 7m to 9m depending upon the rock stiffness and the loads associated with the superstructure.

10.0 Form of Structure

The council have asked for a 3m wide, single-span bridge, being aesthetically pleasing, affordable and sensitive to the landscape and environment. Also, they ask that the supplier will deliver a unique and special design as a showcase for its design acumen. The contract has a relatively high-cost component (40% price) in the tender attributes, which also displays a clear wish for an economical solution more suited to the small South Island town of Gore and having a lasting impact on the social and economic wellbeing of a rural community.

Suitable structures would, therefore, include a lightweight truss or arch, a suspended span or a cable-stayed bridge, arching or bridging over as much of the extended flood plain as possible. The narrow deck and lighter loading would lend itself to an original and unique design incorporating some architectural flair with an attribute for "Artistic impression" worth another 20% of the tender marks.

A track record of similar or projects will be required.

It is expected that the structure will incorporate an allowance for services to cross the river at this point, thus allowing for future possible development to the west. An allowance for this shall be included in the dead loading of the bridge model.

12. References

1. Longford Shared Path Design Contract - Request for Proposal
2. Longford Bridge Shared Pathway_Hydraulic sensitivity analysis_Report
3. Longford Cycle Bridge Factual Geotechnical Report

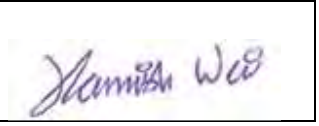
13. Limitations

This report has been prepared in accordance with the brief as provided. It is intended for the sole use of our client, Gore District Council, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

The recommendations in this report are based on the conditions indicated from historical information available, published sources, site assessments and subsurface investigations described in this. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics, ground conditions and hydraulic properties.

12. Quality Assurance

This report prepared, checked and approved by the following people:

Activity	Person	Position	Signed
Prepared by	Martin Cockitt	Civil Engineer ACE Consultants Ltd.	
Checked and approved for release by	Hamish Weir	Project Manager LandPro Ltd.	

Publication versions

Version	Stage	Date
1.	Draft for comment	14/8/2019
2.	2nd draft for comment	15/8/2019
3.	Final	15/8/2019

Geotechnical Investigation and Factual Report for the Proposed Longford River Track – Mataura Cycle/Pedestrian Bridge, Gore

Prepared for:	Gore District Council C/- Landpro Ltd 23 Medway St East Gore Gore 9710
Date:	13/08/2019
Our Reference Number:	19033
Report prepared by:	Mike Pretty Engineering Geologist
Reviewed by:	Jana Kruyshaar CMEngNZ

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Record of Distribution

Sent to	Date	Report Status	No. of Copies
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Appendix 2: Machine Borehole Logs

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Appendix 4: Scala Penetrometer Records

1 Introduction and Scope of Work

LandPro Ltd (LandPro), on behalf of Gore District Council (GDC), requested that JKCM Ltd, trading as Insight Engineering (IE), undertake a geotechnical subsurface investigation and report on factual data at the location of the proposed new Longford Track Cycle Bridge (herein referred to as "the site", refer to Figure 1), as outlined in our Short Form Agreement (reference P19033, approved on 3 July 2019).

We understand that GDC propose to construct a new cycle and pedestrian bridge spanning the Mataura River at the location of the site, and that a geotechnical subsurface investigation and factual report is required. The findings of the investigation are to be used for the detailed design of earthworks and the bridge foundations (by others).

Our scope of work for this project includes:

- A desktop review of available information for the site;
- Organisation of fieldwork including liaison with drilling subcontractors, service locators and freight logistics;
- Fieldwork, including a geotechnical site walkover, part time supervision of borehole drilling, and logging of materials from the boreholes, as well as supplementary Scala Penetrometer tests to assess near surface strength characteristics;
- Preparation of borehole logs in general accordance with NZGS Field Description of Soil and Rock;
- Preparation of a factual geotechnical report (this document), presenting the findings of our desktop study and fieldwork only.

2 Site Description

The site is located at the western end of Church Street adjacent to the Mataura River in East Gore (Figure 1).

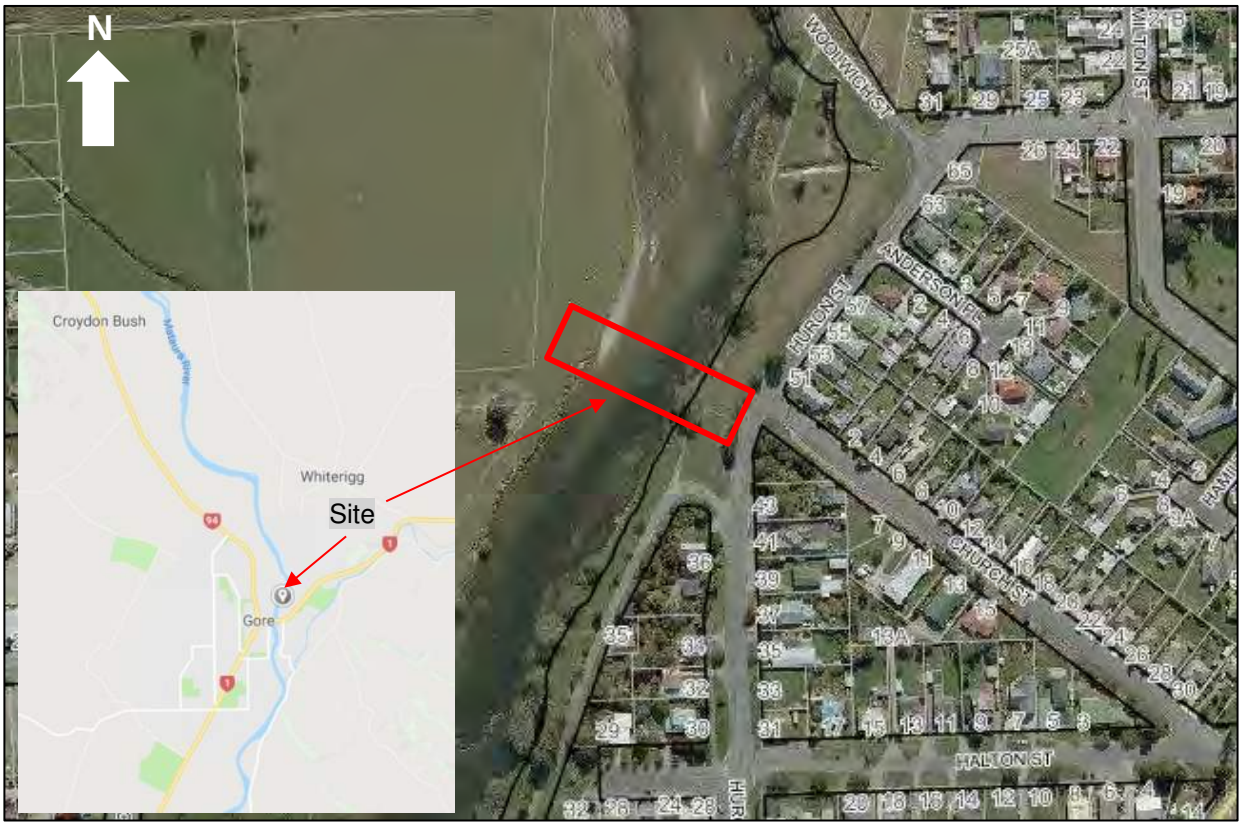
The eastern side of the site (on the eastern side of the river) is bound by Huron Street and River Terrace with residential properties located further to the east and south. An approximately 4 m high earthfill stop bank is present on the eastern side of the river between the river bank and Huron Street.. A public walking track is located between the stop bank and edge of the river.

The western side of the site (on the western side of the River) is predominantly undeveloped land, primarily used as pasture for stock grazing. The western side is generally flat to gently sloping and contains no structures.

Topographic survey of the site was carried out by LandPro and a contour plan is presented as Appendix 1 of this report.

Photographs of the site are presented in Figure 2 below, and further information on the surface conditions at the site is presented in Section 4.1 of this report.

Figure 1: Site Location Plan



Images sourced from GDC Intramaps and Google maps

Figure 2: Site Photographs (taken 29 July 2019)



Photograph 1: View of the proposed bridge location, looking northwest towards the western bank of the Mataura River. Proposed bridge alignment shown as dashed line.



Photograph 2: View of the proposed bridge location, looking southeast towards the eastern bank of the Mataura River.

Figure 2: Site Photographs (taken 29 July 2019 continued).



Photograph 3: View of the proposed bridge location, looking northwest, from the stop bank on the eastern side of the Mataura River.



Photograph 4: View looking north showing the stop bank on the eastern side of the Mataura River.

3 Desktop Review

3.1 Mapped Geology

GNS Science online maps indicate the underlying local geology on the eastern side of the Mataura River as Pleistocene river deposits described as "***sandy greywacke gravel overlain by loess; schist-greywacke-quartz sandy gravel in the Clutha catchment (Q2al)***".

The Mataura River channel and the flat terrace to the west are mapped as Holocene river deposits, described as "u***nconsolidated gravel, sand, silt, clay, and minor peat of modern to postglacial flood plains, may be terraced"***.

At a regional scale, these alluvial deposits locally overlie basement sedimentary rocks of the North Range Group comprising well bedded siltstone, sandstone, tuff, shell beds and conglomerate. The North Range Group rock unit dips 60° to 70° to the southwest and trends approximately northwest – southeast where it is mapped adjacent to the alluvium near the site.

In addition, the GNS Science "Geology of the Murihiku Area" 1:250 000 geological map (map number 20) show a thin outcrop of Stag Stream siltstone (North Range Group) at the site locality.

3.2 Reported Geological and Seismic Hazards

A report on Geological Hazards was prepared by GNS Science dated June 2006. The report was prepared for the Southland District Council Lifelines Study, and broadly describes the primary and secondary effects of seismicity on the Southland District, as well as land stability and volcanism. We also viewed the Environment Southland natural hazards map and GNS Science online maps.

The Geological Hazards report discusses that there has been little seismic activity in eastern Southland since European settlement. The report states that the majority of seismic activity reported for eastern Southland is related to movement of the Pacific – Australian tectonic plate boundary, to the west of Southland, which has resulted in moderate shaking.

3.2.1 Mapped Active Faults

There are no active faults mapped by GNS Science within approximately 22 km of the site. The nearest active faults (broadly defined as those having moved at least once during the last 125,000 years) include the Blue Mountain No. 1 Fault and the Spylaw Fault, both located to the north east of the site. The data presented in Table 1 is extracted from a study complied by Opus Consultants for the Otago Region Council, entitled "*Seismic Risk in the Otago Region: Study Report*".

Table 1: Mapped Active Faults

Fault Name	Estimated magnitude	Estimated recurrence interval	Approximate distance from site
Blue Mountain No. 1 Fault	6.4	800 years >8,000 years	22 km (northeast)
Spylaw Fault	6.3	1,300 years 5,000 – 10,000 years	27 km (northeast)

The Alpine fault is mapped (by GNS) transitioning offshore approximately 188 km northwest of the site. It continues offshore to the west of Fiordland and presents a seismic hazard to the region, along with other active faults mapped throughout Central Otago and Western Southland. Rupture of these faults has the potential to result in ground shaking in Gore. The likelihood of a major Alpine fault rupture has been assessed as 30% over the next 50 years (Berryman et al., 2012).

3.2.2 Mapped Inactive Faults

Several un-named inactive faults are mapped by GNS Science within approximately 0.5 km to 3.5 km of the site. These faults are depicted as solid and dashed black lines in Figure 3.

Figure 3: Mapped Inactive Faults



Images sourced from GNS Science Web Maps

3.3 Environment Southland Hazards Database

3.3.1 Mapped Liquefaction Risk

The western side of the site is mapped as having a potential Liquefaction Risk: Medium. The Hazard Database mapping includes reference to a report prepared by GNS Science dated January 2012, entitled "***Amplified ground shaking and liquefaction susceptibility, Invercargill City***", as well as their report on Geological Hazards dated June 2006.

3.3.2 Flooding

The site is located within a major floodplain of the Mataura River. However, predicted flood levels are not reported.

3.3.3 Groundwater

Available groundwater information was reviewed. The nearest monitoring wells are located approximately 400 m to the east of the site and are reported as being used for groundwater quality monitoring. Environment Southland provided some information related to these wells (numbered F45/0436 to F45/0473) including the depth of initial groundwater measurements (measured when drilled). The typical depths to water in these wells ranged between 2.0 m and 2.4 m below the ground surface.

3.4 Aerial Photographs

Publicly available aerial photographs reviewed as part of this investigation were taken between 1938 and 1984 and are available from retrolens.nz. We also viewed images taken from 2010 to 2018 which are available on Google Earth.

No geotechnically significant changes to the ground surface are visible in the photographs. Fluctuations in the river level are apparent. Increased vegetation and trees along the riparian zone, particularly the western bank are noted between 1938 and 1948. Some of the trees appear to have been removed prior to 1984. No significant changes to the river channel shape are visible in the photographs.

The most recent image available from Google Earth shows what appears to be exposed rock in the river channel. The rock is primarily visible in the channel bed on the eastern side under the proposed bridge alignment (refer to Section 4.1.2 below).

4 Fieldwork

IE visited the site on 29 July 2019 and conducted a site walkover as well as limited subsurface investigations in locations indicated in Appendix 1 of this report.

4.1 Surface Observations

Our observations are discussed in the following sections and annotated on Figure 4 below.

4.1.1 Riverbank Stability

As seen from Photograph 5 in Section 2 of this report, local instability of the riverbank is visible in places, particularly along the western bank. The instability is characterised by slumping of the surficial materials and development of near vertical scarps in places. The scarps range in height between approximately 0.5 m to 1.5 m in the vicinity of the proposed bridge alignment. The slumping is estimated to extend approximately 3 m back from the bank in this area.

Further to the north of the site, slumping is more prominent and slump features extend up to approximately 12 m back from the riverbank.

We did not observe indications of slumping on the eastern side of the river in the area of the proposed bridge alignment. The eastern bank is relatively densely vegetated with grass and occasional flax.

4.1.2 Local Rock Exposures

Rock is exposed at several locations at the site and in the vicinity.

Rock exposures are visible on the eastern side of the riverbed in the area around the proposed bridge alignment. These outcrops continue intermittently north approximately 300 m; and south approximately 500 m from the site. These exposures are also visible in aerial photographs as discussed in Section 3.4 previously. The rock typically comprises steeply dipping siltstone and sandstone with closely to very closely spaced joints.

4.1.3 Figures and Photographs

Figure 4: Surface Observations

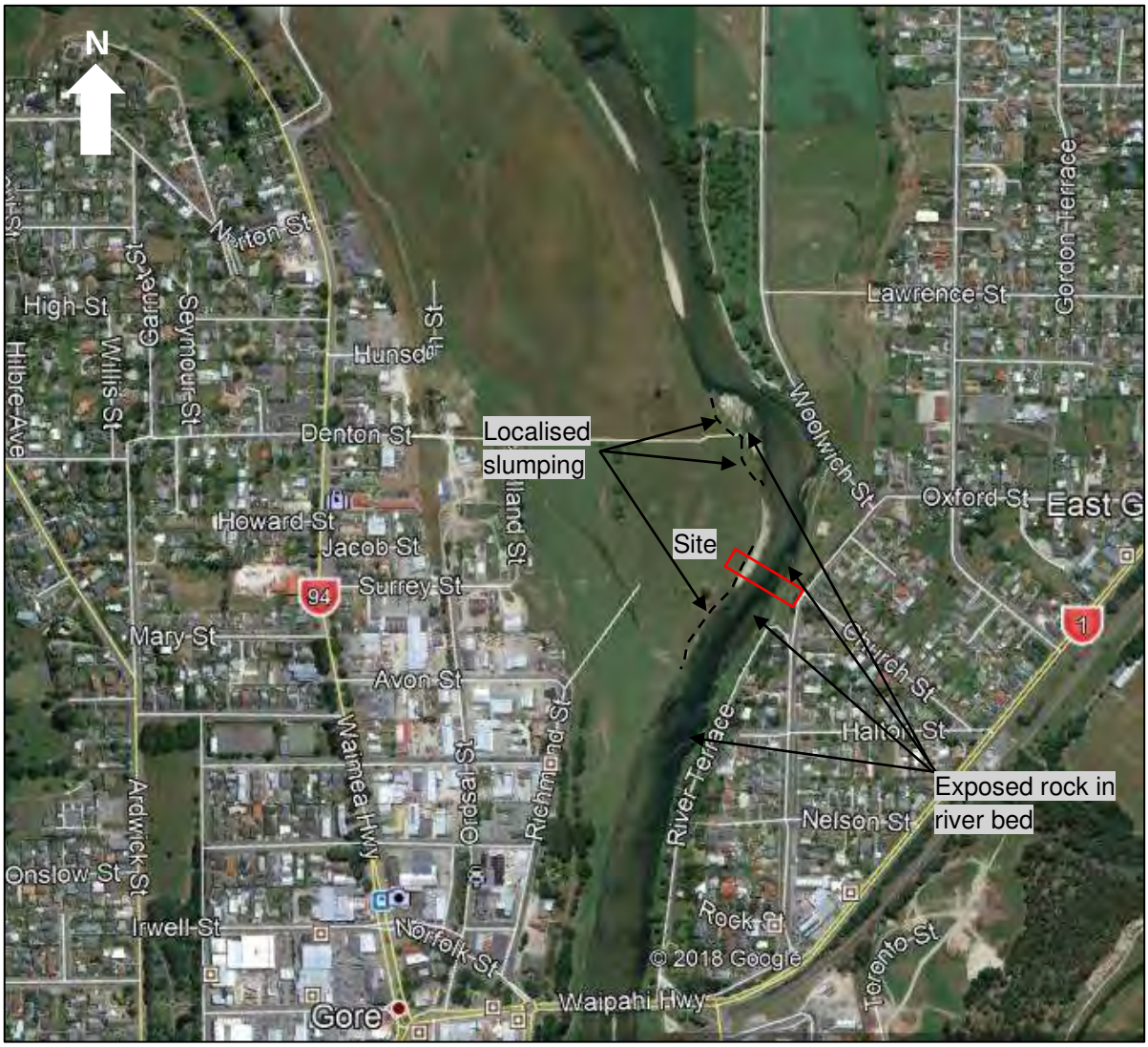


Image sourced from Google Earth 2018

Figure 5: Slumping and Rock Exposure Photographs



Photograph 5: View of slumping (indicated by white dashed lines) of the western bank of the Mataura River at the site location. River gravel alluvium is also clearly shown in the foreground.



Photograph 6: View looking west from the top of the stop bank on the eastern side of the Mataura River, showing the extent of slumping at the site.



Photograph 7: Slumping to the north of the site.



Photograph 8: Close up view of scarp at the site.



Photograph 9: Rock exposure in the Mataura River, to the north of the site.



Photograph 10: Rock visible in the Mataura River bed, at the site (eastern side).

4.2 Subsurface Investigations

4.2.1 Machine Boreholes

Borehole drilling was carried out by McNeill Drilling using a track mounted rotary sonic drilling rig. Six boreholes were drilled to depths ranging between 4.2 m and 9.0 m below the ground surface. The boreholes were terminated in hard materials as instructed by IE.

Standard penetration tests (SPTs) were generally carried out at 1.5 m intervals during drilling, and static water levels were measured upon completion of drilling of BH01 to BH05.

The boreholes were backfilled with bentonite upon completion of drilling and piezometers were not installed.

In broad terms, the materials encountered by boreholes numbered BH01 to BH05 comprised a surficial layer of native, loose to dense alluvium overlying weathered, jointed rock. The alluvium typically comprised fine silty SAND, sandy SILT and sandy, fine to coarse GRAVEL. The rock typically comprised slightly to moderately weathered, SANDSTONE and SILTSTONE with extremely closely to very closely spaced joints, which was commonly recovered as angular sand and gravel. The rock is typically strong to very strong (see note below). The materials encountered by BH06 comprise FILL to a depth of 4.9 m, as this borehole was drilled through the stop bank on the eastern side of the river. The fill is underlain by highly to completely weathered rock that was recovered as silty GRAVEL. Slightly weathered rock was encountered from 8.9 m depth near the end of the hole.

It should be noted that rock strength terms apply to the strength of individual clasts of rock rather than the rock mass itself in accordance with logging guidelines. These terms are based on field checks of limited samples recovered from drilling during logging.

Full descriptions of the materials encountered are presented on the borehole logs. The logs were prepared in general accordance with the New Zealand Geotechnical Society Field Description of Soil and Rock (2005), are presented as Appendix 2. Photographs of the core are presented as Appendix 3.

Table 2 summarises the machine boreholes.

Table 2: Machine Borehole Summary

Borehole No.	Location	Ground surface Elevation[1] (mRL)	Termination Depth (m)	Depth to groundwater (m)	Depth to rock (m)	Depth to SPT N-value >50[2]
BH01	West bank	71.84	6.0	1.3	2.5	3.0
BH02	West bank	72.47	6.0	1.7	2.7	2.0
BH03	West bank	72.59	4.2	1.6	3.1	3.0
BH04	West bank	72.78	4.5	2.1	3.1	3.0
BH05	East bank	72.62	4.5	1.9	2.9	3.0
BH06	East bank (through the stop bank)	76.70	9.0	Not recorded	6.5[3]	7.5

[1] Ground surface elevations are in terms of Dunedin to Bluff Vertical Datum 1960
[2] SPT values are presented as recorded during drilling (uncorrected). SPT Hammer efficiency is reported as 90.7%
[3] The rock encountered in BH06 is highly to completely weathered and recovered as silty GRAVEL to a depth of 8.9 m.

4.2.2 Scala Penetrometer Tests

The Scala tests were completed at the western side of the site for the purpose of assessing the relative strengths of the near surface materials. The tests were terminated at depths ranging between 1.3 m and 1.9 m due to practical refusal (on inferred dense alluvium).

Copies of the Scala test results are presented in Appendix 4.

5 References

1. Turnbull, I. M.; Allibone, A.H. (compilers) 2003. Geology of the Murihiku area. Institute of Geological and Nuclear Sciences1:250 000 geological map 20. 1 sheet plus 74 p. Lower Hutt, New Zealand: Institute of Geological and Nuclear Sciences Ltd.

2. GNS New Zealand Active Fault Database. Available online at :< https://data.gns.cri.nz/af/ >. Accessed August 2019.

3. GNS Webmap Institute of Geological and Nuclear Sciences 2013: 1:250,000 Geology. Viewed at: http://data.gns.cri.nz/geology/ .

4. Glassey, P.J. GNS Science Consultancy Report 2006/100 (2006). Geological Hazards, Southland District Council Lifelines Study.

5. Environment Southland Natural Hazards GIS mapping. Viewed at: http://gis.es.govt.nz/index.aspx?app=natural-hazards

6. Environment Southland Groundwater GIS mapping. Viewed at: http://gis.es.govt.nz/index.aspx?app=groundwater

7. Opus Consultants Ltd 2005. Seismic Risk in the Otago Region, Report No. SPT:2004/23

8. Retrolens Historical Image Resource. Viewed at retrolens.nz

6 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Gore District Council, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

iv. This Limitation should be read in conjunction with the IPENZ/ACENZ Standard Terms of Engagement.

v. This report is not to be reproduced either wholly or in part without our prior written permission.

Appendix 1

LandPro Ltd Bridge and Borehole Location Plan

A3



Note:
- Coordinates in terms of NZGD 2000 Bluff Circuit

- Levels in terms of Dunedin to Bluff Vertical Datum 1960
  Origin of Levels AE0C RL= 75.13

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NOTE:
- All dimensions shown are in metres unless otherwise shown
- Copyright on this drawing is reserved
- Check any electronic data against the hardcopy plan to ensure it is the latest version
- If this plan is being used as part of sale and purchase agreement then it is done so on the basis that it is preliminary only, final dimensions and areas may vary on final survey

Longford Shared Pathway
Mataura River Bridge Location

Datum & Level
Bluff 2000
DBVD 1960

N

Rev.	Date	Revision Details	By
1	6-08-19	Survey Geotech Test Locations	HW

Surveyed	HRW	Signed		Date	8-07-19	Job No.	19241
Drawn	HRW	Signed		Date	8-07-19	Scale	1:500 @ A3
Designed		Signed		Date		Drawing No.	01
						Rev.	1

L:\19241 - Gore District Council - Longford Bridge Shared Pathway\12d 19241 GDC Bridge Project Tue Aug 06 10:54:14 2019

Appendix 2

Machine Borehole Logs

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Matuara River, Gore				
Ref:	19033	**Date:**	29/7/19	**Page:**	1 of 2

Machine Borehole BH01

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%)
TS	Organic fine SAND with some silt and trace gravel; dark brown; poorly graded. [TOPSOIL]							
ALLUVIUM	Fine SAND with some silt; brownish grey; medium dense; moist; poorly graded.		1					
	Orange mottles and silty from 1.2 m depth							
	Some medium to coarse gravel; sub-rounded; quartz and sandstone from 1.5 m depth						4,4,5,3,4,7 N = 19	
	Sandy fine to coarse GRAVEL with minor silt; grey; dense; wet; well graded; sub angular to sub-rounded; quartz and sandstone.		2				4,4,10,8,8,15 N = 41	
NORTH RANGE GROUP SILTSTONE	Slightly to moderately weathered, bluish grey SANDSTONE; strong to very strong; very closely spaced joints (20 - 50 mm).							
	Brownish grey from 3.0 m depth Extremely closely spaced joints from 3.0 m to 3.2 m depth		3				24,26 N = 50 (90mm)	
	Moderately weathered from 3.7 m to 4.0 m depth		4					
	Moderately weathered, bluish grey SILTSTONE; very weak to moderately strong; extremely closely spaced joints (<10 mm).							
	Slightly to moderately weathered, bluish grey SANDSTONE; strong to very strong; extremely to very closely spaced joints (5 - 30 mm). Recovered as angular sand and gravel.		5				8,30 Bouncing	

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 29/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09295
Longitude: 168.95227

NOTES: TS = TOPSOIL
Static water level recorded at 1.3 m depth following drilling
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Mataura River, Gore				
Ref:	19033	**Date:**	29/7/19	**Page:**	2 of 2

Machine Borehole BH01

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%) 20 40 60 80
NORTH RANGE GROUP SILTSTONE	continued- Slightly to moderately weathered, bluish grey SANDSTONE; strong to very strong; extremely to very closely spaced joints (5 - 30 mm). Recovered as angular sand and gravel.		6					
	EOH 6.0 m		7 8 9 10					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 29/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09295
Longitude: 168.95227

NOTES: TS = TOPSOIL
Static water level recorded at 1.3 m depth following drilling
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Mataura River, Gore				
Ref:	19033	**Date:**	29/7/19	**Page:**	1 of 2

Machine Borehole BH02

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%) 20 40 60 80
TS	Organic fine SAND with some silt; dark brown; poorly graded. [TOPSOIL]							
ALLUVIUM	Sandy SILT; brownish grey; 'firm'; moist; low plasticity. Trace organic material and charcoal at 0.3 m depth.		1					
	Fine SAND with minor silt and trace gravel; brownish grey; 'loose to medium dense'; moist; poorly graded.							
	Sandy fine to coarse GRAVEL with minor cobbles and trace silt; brownish grey; medium dense; wet; well graded; sub angular to sub-rounded; quartz and sandstone. Cobbles up to 80 mm.						1,2,2,2,6,7 N = 17	
	Very dense from 2.0 m depth		2				10,12,13,13,14,10 N = 50 (290mm)	
NORTH RANGE GROUP SILTSTONE	Slightly to moderately weathered, grey SILTSTONE; moderately strong, extremely to very closely spaced joints (10 - 50 mm). Recovered as angular gravel and silt.		3				4,11,29,21 N = 50 (80mm)	
	SANDSTONE from 3.4 m to 3.5 m depth; moderately strong to strong.							
	Slightly to moderately weathered, bluish grey SANDSTONE; strong to very strong; very closely spaced joints (20 - 60 mm). Light grey clay coating on rock clasts.		4				7,35 Bouncing	
	Some clayey soil present in 4.0 m SPT sample including minor white clayey soil, possible highly weathered siltstone, minor shear zone or drilling related.		5					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 29/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09290
Longitude: 168.95215

NOTES: TS = TOPSOIL
Static water level recorded at 1.7 m depth following drilling
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Mataura River, Gore				
Ref:	19033	**Date:**	29/7/19	**Page:**	2 of 2

Machine Borehole BH02

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%) 20 40 60 80
NORTH RANGE GROUP SILTSTONE	continued- Slightly to moderately weathered, bluish grey SANDSTONE; strong to very strong; extremely to very closely spaced joints (5 - 30 mm). Recovered as angular sand and gravel.		6					
	EOH 6.0 m		7 8 9 10					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 29/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09290
Longitude: 168.95215

NOTES: TS = TOPSOIL
Static water level recorded at 1.7 m depth following drilling
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Mataura River, Gore				
Ref:	19033	**Date:**	29/7/19	**Page:**	1 of 1

Machine Borehole BH03

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%)
TS	Orgnic silty fine SAND; dark greyish brown; poorly graded. [TOPSOIL]							
ALLUVIUM	Sandy SILT; greyish brown; 'soft to firm'; moist; low plasticity. Grades to SILT with minor sand.		1					
ALLUVIUM	Fine SAND with trace silt; brownish grey; dense; moist; poorly graded.						3,7,9,11,12,11 N = 43	
ALLUVIUM	Sandy fine to coarse GRAVEL with trace cobbles; grey; 'dense'; well graded, sub-rounded; quartz and sandstone. Sand; fine to coarse, well graded, sub-rounded.		2					
NORTH RANGE GROUP SILTSTONE	Slightly weathered, grey to bluish grey SANDSTONE; strong to very strong. CORE LOSS - 50 mm core recovered between 3.1 m and 4.2m depth so may not be representative. Contact depth inferred from SPT and drillers logs.		3				17,29,21 N = 50 (50mm)	
	EOH 4.2 m		4				30 Bouncing	
			5					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 29/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09284
Longitude: 168.95194

NOTES: TS = TOPSOIL
Static water level recorded at 1.6 m depth following drilling
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Mataura River, Gore				
Ref:	19033	**Date:**	29/7/19	**Page:**	1 of 1

Machine Borehole BH04

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%)
TS	Orgnic fine SAND with minor silt; dark brown; poorly graded. [TOPSOIL]							
ALLUVIUM	Fine SAND with some silt; brownish grey; 'medium dense'; moist; poorly graded. CORE LOSS - 0.3 m recovered from 0 to 1.5 m depth, sand unit is partly inferred.		1					
ALLUVIUM	Sandy fine to coarse GRAVEL with trace cobbles; grey; medium dense; well graded, sub-rounded; quartz and sandstone. Sand; fine to coarse, well graded, sub-rounded. Occasional sand lens (<100 mm thick)		2				3,6,6,9,7,6 N = 28	
ALLUVIUM	Dark brown silt coating clasts from 2.6 m depth, possible paleosurface/paleosol.		3				4,11,13,25,12 N = 50 (170mm)	
NORTH RANGE GROUP SILTSTONE	Slightly to moderately weathered, grey SILTSTONE; strong; extremely to very closely spaced joints (5 - 50 mm). Recovered as angular sand/gravel with grey silt/clay coating.		4					
NORTH RANGE GROUP SILTSTONE	Slightly weathered, dark grey SANDSTONE; strong to very strong; very closely spaced joints (20 -50 mm).							
	EOH 4.5 m		5					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 29/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09286
Longitude: 168.95209

NOTES: TS = TOPSOIL
Static water level recorded at 2.1 m depth following drilling
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Church Street, Gore				
Ref:	19033	**Date:**	30/7/19	**Page:**	1 of 1

Machine Borehole BH05

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%) 20 40 60 80
FILL	Fine to coarse GRAVEL with some silt; brownish grey; well graded; 'dense'; sub-angular to angular; sandstone. [FILL]							
ALLUVIUM	Silty fine SAND with trace gravel; brownish grey; 'medium dense'; dry to moist; poorly graded.		1					
							3,1,1,3,3,3 N = 10	
	Trace organic material at 2.3 m depth (degraded wood), some brown mottling from 2.3 m depth		2					
	Gravelly SILT; dark brown (paleosurface/residual soil)							
NORTH RANGE GROUP SILTSTONE	Slightly weathered, grey to bluish grey SANDSTONE; strong to very strong; extremely to very closely spaced joints (10 - 50mm). Recovered as angular sandy gravel.		3				11,30 Bouncing	
			4					
	EOH 4.5 m						1,40 Bouncing	
			5					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 30/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09330
Longitude: 168.95311

NOTES: TS = TOPSOIL
Static water level recorded at 1.9 m depth following drilling
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Church Street, Gore				
Ref:	19033	**Date:**	30/7/19	**Page:**	1 of 2

Machine Borehole BH06

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%)
TS	Orgnic sandy SILT; dark brown; low plasticity. [TOPSOIL]							
FILL	Silty fine SAND; greyish brownish; 'medium dense'; dry to moist; poorly graded. [FILL]							
	Gravelly SILT with minor sand; greyish brown; 'stiff to very stiff'; dry to moist; low plasticity. Gravel; fine to coarse, sub-rounded to sub-angular, well graded, sandstone. [FILL]		1 2 3 4				6,6,5,6,5,6 N = 22 1,2,2,4,4,6 N = 16 3,4,3,4,4,7 N = 18	20 40 60 80
	SILT with minor gravel - see next page		5					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 30/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09337
Longitude: 168.95329

NOTES: TS = TOPSOIL
Static water level not recorded
SPT values presented as recorded (uncorrected)

Project:	Longford River Track - Mataura River Cycling / Pedestrian Bridge				
Client:	Gore District Council C/- Landpro Ltd				
Location:	Church Street, Gore				
Ref:	19033	**Date:**	30/7/19	**Page:**	2 of 2

Machine Borehole BH06

REDUCED LEVEL (RL) / INFERRED GEOLOGY	DESCRIPTION OF SOIL	SOIL SYMBOL	DEPTH (m)	SAMPLE TYPE	WATER CONTENT (%)	WATER LEVEL	SPT Blowcounts and Lab Testing	Core Recovery (%) 20 40 60 80
ALLUVIUM	continued from 4.9 m depth - SILT with minor gravel; dark brown with grey and orange brown mottles; 'firm', moist, moderate plasticity. Possible paleosol or buried topsoil layer.							
	SILT; brownish grey with orange-brown mottles; 'stiff'; moist; low plasticity.							
	Fine to coarse SAND with some gravel and minor silt; greyish brown; medium dense to dense; moist to wet; well graded. Gravel; fine to coarse, sub-rounded, quartz and sandstone.		6				1,3,3,4,5,4 N = 16	
NORTH RANGE GROUP SILTSTONE	Highly to completely weathered, SANDSTONE recovered as soil: Silty fine to coarse GRAVEL with some sand; bluish grey; dense to very dense; moist; well graded; sub-angular to sub-rounded. Grainsize proportions vary.		7				6,7,16,19,15 N = 50 (215 mm)	
	Brownish grey from 7.8 to 8.6 m depth		8					
	Slightly weathered, bluish grey SANDSTONE; strong to very strong; closely spaced joints (50 mm).		9				11,14,29,21 N = 50 (95 mm)	
	EOH 9.0 m		10					

DRILL METHOD: Sonic Coring
DRILLER: McNeill Drilling
DATE DRILLED: 30/07/2019
CORE DIAMETER: 85 mm
SPT EFFICIENCY: 90.70%
LOGGED BY: MP

Latitude: -46.09337
Longitude: 168.95329

NOTES: TS = TOPSOIL
Static water level not recorded
SPT values presented as recorded (uncorrected)

Appendix 3

Machine Borehole Core Photographs



BH01 Box 1: 0m to 2.8 m



BH01 Box 2: 2.8 m to 6.0 m



BH02 Box 1: 0 m to 2.7 m



BH02 Box 2: 2.7 m to 5.9 m



BH02 Box 3: 5.9 m to 6.0 m

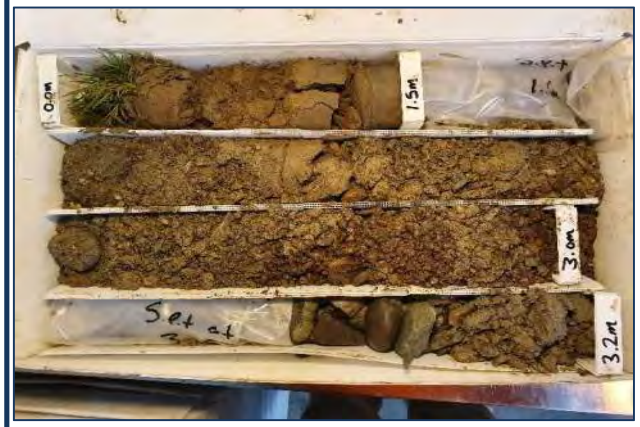
Description	Core Photographs	**Photos**	BH01 & BH02
Project	Proposed Longford River Track Cycle/Pedestrian Bridge - Mataura River, Gore	**Date Taken**	07/08/2019
Client	Gore Dstrict Council C/- LandPro Ltd	**Taken by**	MP
Project Number	19033	**Approved by**	JK



BH03 Box 1: 0m to 3.0 m



BH03 Box 2: 3.0 m to 4.2 m

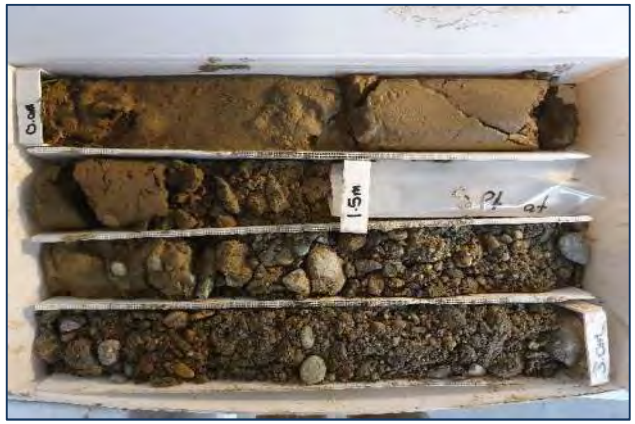


BH04 Box 1: 0 m to 3.2 m



BH04 Box 2: 3.2 m to 4.5 m

Description	Core Photographs	**Photos**	BH03 & BH04
Project	Proposed Longford River Track Cycle/Pedestrian Bridge - Mataura River, Gore	**Date Taken**	07/08/2019
Client	Gore Dstrict Council C/- LandPro Ltd	**Taken by**	MP
Project Number	19033	**Approved by**	JK



BH05 Box 1: 0m to 3.0 m



BH05 Box 2: 3.0 m to 4.5 m



BH06 Box 1: 0 m to 3.0 m



BH06 Box 2: 3.0 m to 5.9 m



BH06 Box 3: 5.9 m to 7.5 m



BH06 Box 4: 7.5 m to 9.0 m

Description	Core Photographs	**Photos**	BH05 & BH06
Project	Proposed Longford River Track Cycle/Pedestrian Bridge - Mataura River, Gore	**Date Taken**	07/08/2019
Client	Gore Dstrict Council C/- LandPro Ltd	**Taken by**	MP
Project Number	19033	**Approved by**	JK

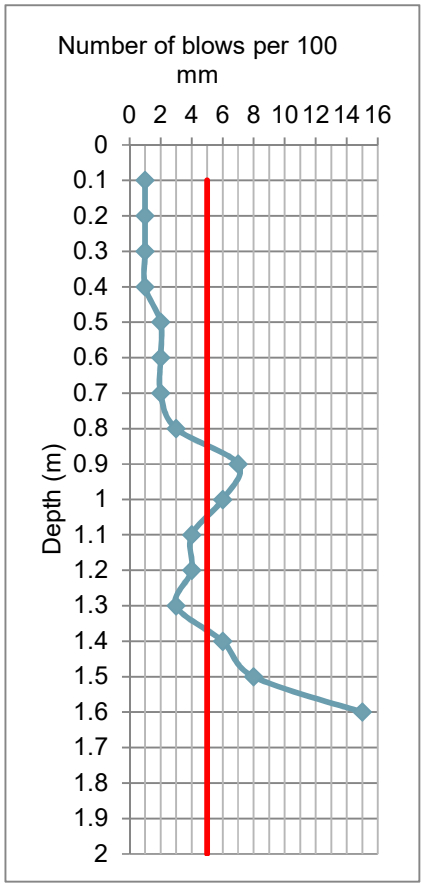
Appendix 4

Scala Penetrometer Records

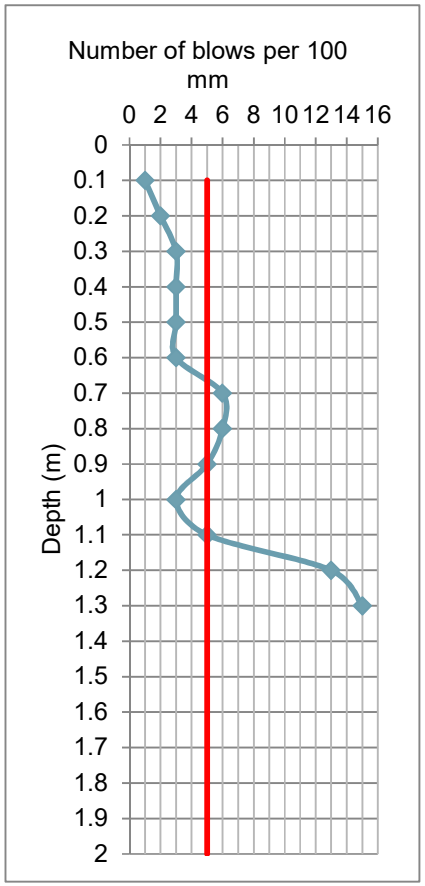
Insight Engineering - Scala Penetrometer Test Records

Project No.:	19033	**Client:**	Gore District Council C/- Landpro Ltd
Date:	29/06/2019	**Location:**	Refer to site plan

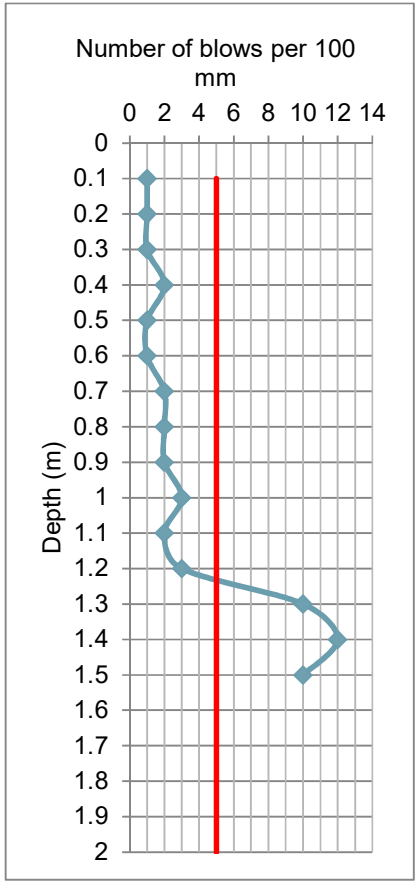
Test 1



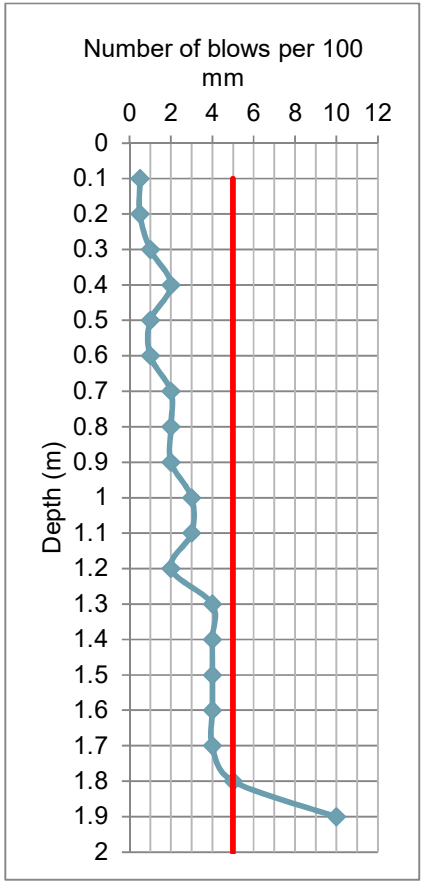
Test 2



Test 3



Test 4



Notes: The blue line indicates the test results.
All tests terminated at practical refusal (15+ blows per 100 mm or bouncing)

Gore District Council
Request for Proposal

Longford Shared Path Design Contract - Request for Proposal Contract No. 2019/11

Appendix C – Preliminary Hydraulic Report

Gore District Council
29 Bowler Avenue
Gore 9710

Contract for Consultancy Services

Preliminary Hydraulic Report

LANDPRO
Make the most of your land

Date: 21 August 2019 File Ref: 19241

Subject: Gore District Council – Longford Bridge Shared Pathway: Sensitivity Analysis of Hydraulic Parameters Related to bridge location

1. Introduction

This report is a desktop analysis undertaken to provide some estimates and analysis of potential flood flow velocities in the Mataura River at the proposed Longford Bridge location. It is intended to provide data as a basis for future detailed hydraulic design associated with bridge design work.

This report should be taken as a preliminary finding and as a basis for the information available. It is recommended that an appropriately qualified and experienced hydraulic engineer be engaged to undertake detailed analysis at the final bridge design stage.

2. Background Information

The proposal, part of a far larger scheme, is for a shared cycle/pedestrian bridge across the Mataura River in Gore approximately 650m upstream of the State Highway 1 bridge (Figure 1). At this location, there are stopbanks on each side of the river, with the main channel tending to run close to the true left stopbank while there is a wide flood berm within the true right stopbank. The new bridge will span the main permanent channel of the Mataura River (Figure 2), starting on the top of the true left flood bank and terminating on the flood berm on the true right bank of the main channel. The right bank end of the bridge will be subject to inundation during flood flows that breach the usual channel and cover the right bank flood zone. Therefore, there will be flowing water around the bridge footing structure during large floods.

The Mataura River in this location frequently floods and has flooded outside of the main river channel on numerous occasions. The largest recorded flood was in October 1978 with a peak flow of 2,297m^3/s and a water level 4.69m "above normal" at the nearby Mataura River at Gore monitoring station (Environment Southland, 2019). Table 1 shows the key flow statistics for this site.

Table 1: Mataura River at Gore Flow Statistics (Environment Southland, 2019)

Mataura River at Gore flow statistics	
Catchment area	3,524 km^2
Mean flow	65.06 m^3/s
Flood statistics	
Highest recorded Flow	2,106 m^3/s
Highest recorded flood	2,297 m^3/s on 14/10/1978
Second highest recorded flood	1,915 m^3/s on 11/03/1987
Third highest recorded flood	1,844 m^3/s on 18/11/1999
Flood return periods	
Mean Annual Flood	565 m^3/s
5 year (20% AEP) Flood	820 m^3/s
10 year (10% AEP) Flood	1,087 m^3/s
20 year (5% AEP) Flood	1,405 m^3/s
50 year (2% AEP) Flood	1,932 m^3/s
100 year (1% AEP) Flood	2,436 m^3/s

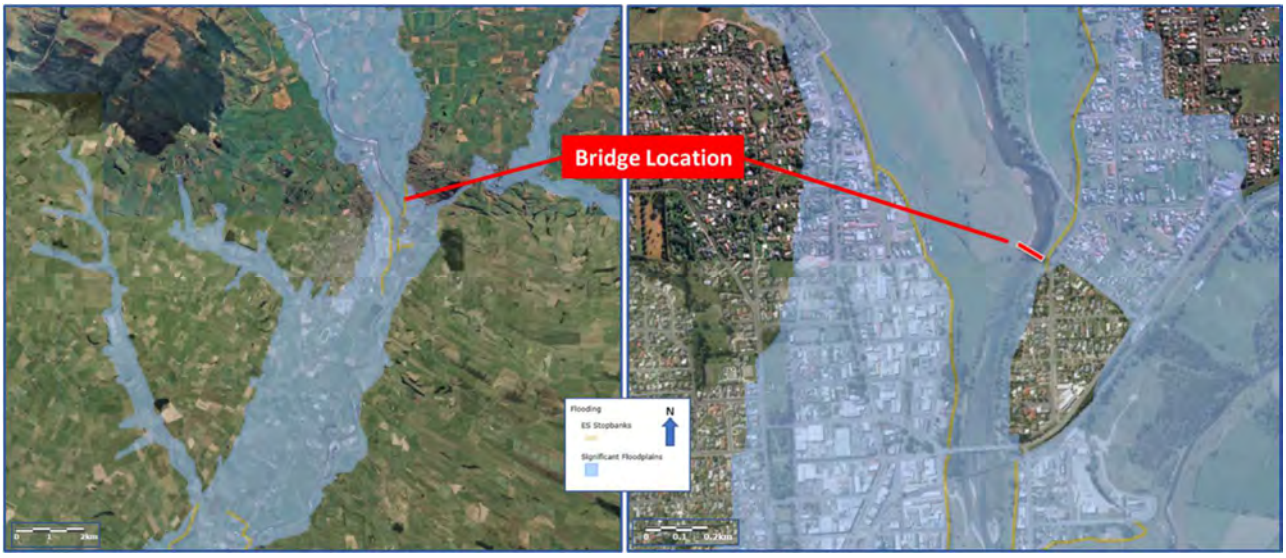


Figure 1: *Environment Southland flood maps and proposed bridge location (Environment Southland, 2019).*

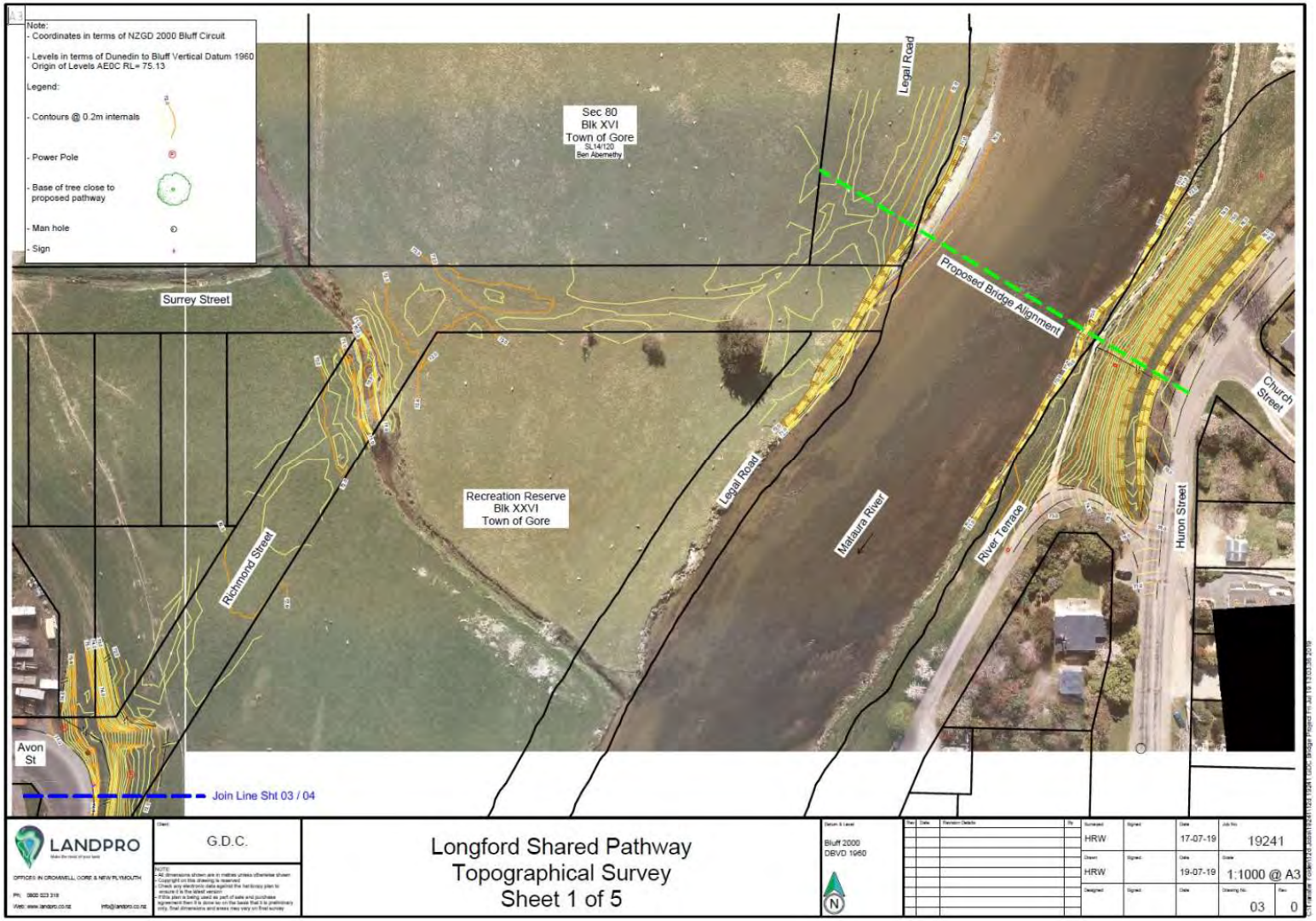


Figure 2: *Detailed plan of proposed bridge location.*

Historical photographs of previous floods clearly show that this reach of the Mataura River can flood from stopbank to stopbank in large events (Figure 3). It can be clearly seen that the right bank end of the bridge would be completely surrounded by water in any flood that breaches the main channel.

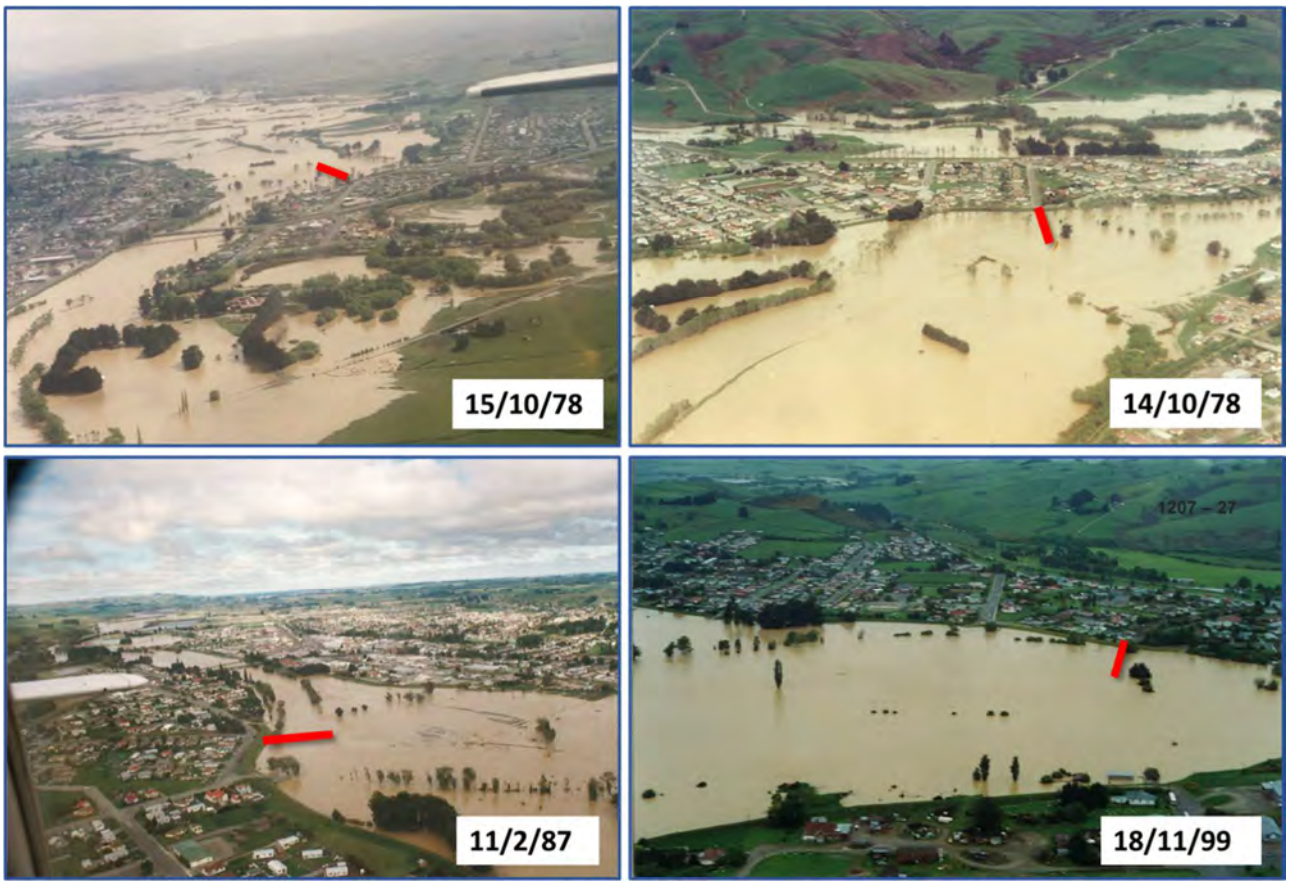


Figure 3: *Historic flood photos of bridge location (approximate bridge location denoted by red line) (Environment Southland, 2019).*

It can also be seen in Figure 4, that there is a definite split in flow proportions during large flood events, between the main channel and the flood berm. The large standing waves in the main channel, evident in the photos, indicate that there are high velocities in this main area. There are relative low velocities across the flood berm by comparison. However, there is evidence that there is still flow occurring across the flood berm, shown by the lines of turbulence stretching downstream from the submerged fence as shown below.

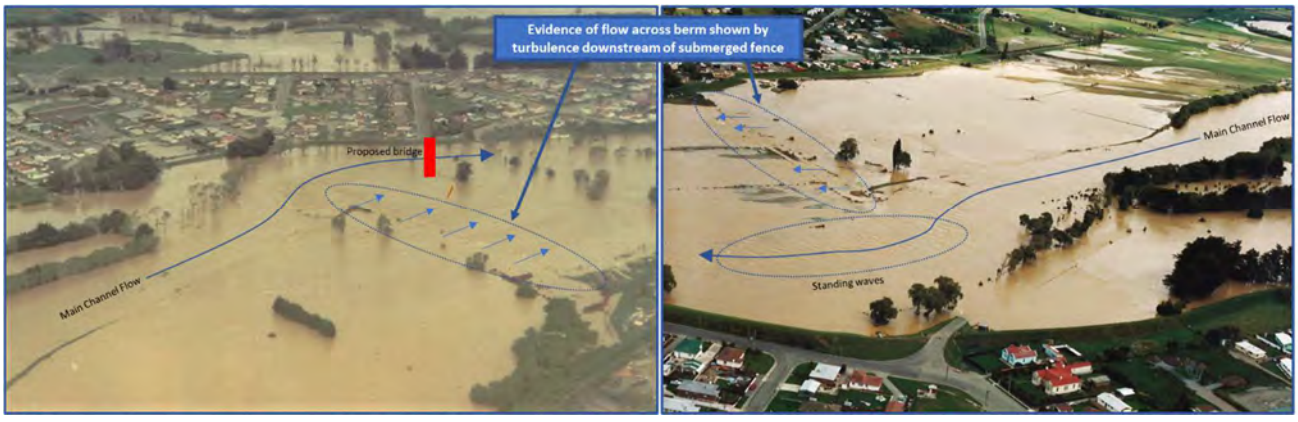


Figure 4: *Analysis of aerial flood images for evidence of flow velocity across flood berm and in main channel. Note turbulence downstream of the submerged fence on the flood berm. Standing waves in the main channel indicate high velocities (photos (Environment Southland, 2019).*

3. Methodology

The key hydraulic parameter for the design of the right bank end of the bridge is the velocity of water that will surround the structure during flood flows. Ideally, this would be modelled using complex hydraulic models, but as the timeframe and budget do not allow for that at this point in the project. The advice of Environment Southland (ES), has been obtained. They are responsible for the management of the river channel in this area for the purpose of protecting the town of Gore from flooding. Therefore, this project has been discussed with Colin Young, Technical Services Engineer at ES, and he has provided some advice on aspects of this analysis. ES has a flood model that covers this reach of the Mataura River that could help to resolve velocities, but the results are not readily available as the model will require updating with new model runs. ES do, however, have good cross-section data and have made these available for this analysis.

The general approach taken with this analysis is to run a basic sensitivity analysis to assess velocities using two fundamental hydraulic relationships including:

$$Q = VA$$

and therefore:

$$V = Q/A$$

Also used is the Manning's formula for estimating velocity in an open channel:

$$V = R^{2/3}S^{1/2}/n$$

Where:

V = Average velocity (m/s)
Q = Discharge (m^3/s)
A = Cross section area (m^2)
R = Hydraulic Radius (m)
S = Channel slope (m/m)
n = Manning's roughness coefficient

The key parameters have been evaluated as follows:

Discharge (*Q*)

For simplicity, a near "worst-case scenario" has been used for estimation of severe flood flows. In this case, the peak flood flow for the 14th October 1978 flood (Table 1) was used: 2297 m^3/s.

Cross-section area (*A*) and Hydraulic Radius (*R*)

The analysis is based on cross-section number 21 provided by ES (Figure 5). This is taken on the approximate location of the proposed bridge (Figure 6). Colin Young of ES has provided an estimate of 75.51m for the 1978 flood peak water level at cross-section 21, based on a backwater curve calculated for that location from the main flow site downstream. It should be noted that based on this data, the depth of the water during this flood at the approximate location of the right bank bridge end would be approximately 3-3.5 m.

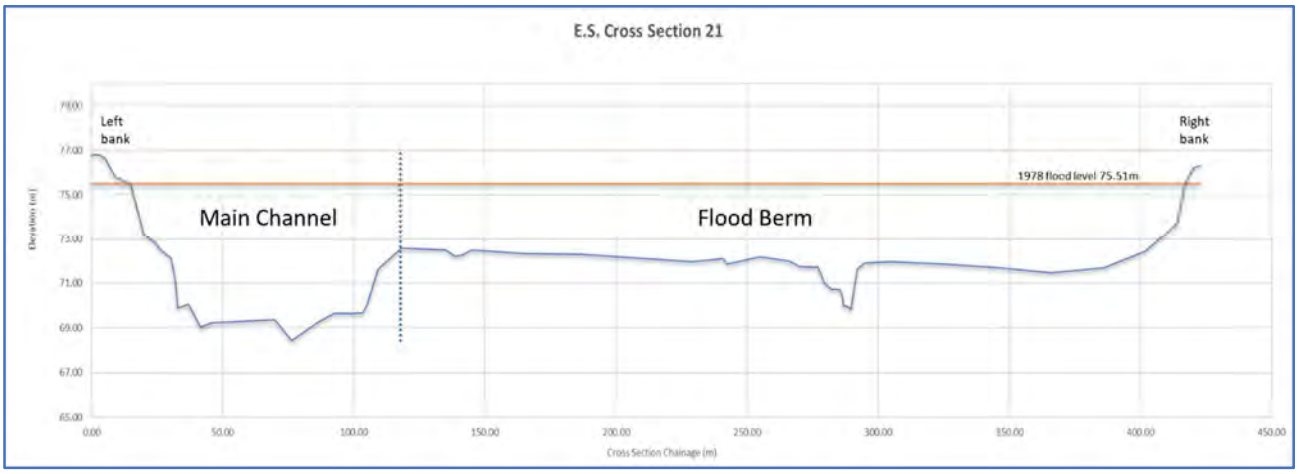


Figure 5: *Environment Southland Cross-section 21 looking downstream showing 1978 flood level. For the analysis, the channel is split at the blue dashed line into Main Channel and Flood Berm.*

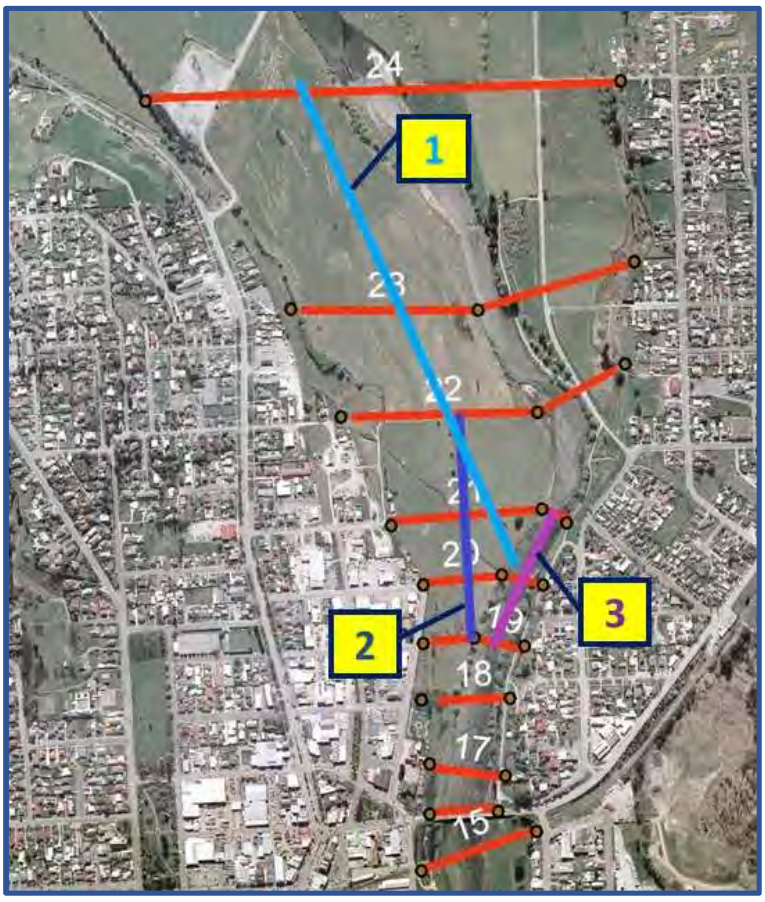


Figure 6: *Environment Southland cross-section locations and long-sections used for calculating channel slope.*

As shown in Figure 5, the cross-section was split into two sections: the main channel and the flood (zone) berm. This has been done to account for the clearly different dynamics in each section, especially the difference in velocities as demonstrated in Figure 4. For the analysis, the split cross-sections have been imported into the Hilltop software package, which has been used to calculate the area (*A*) and the hydraulic radius (*R*). For the area calculations, the 1978 peak flood level of 75.51m was used. This level was calculated by ES from their backwater curve for this location. The cross-section attributes are detailed in Table 2.

Table 2: Cross-section geometry attributes

Attribute	Main channel	Flood berm	Total channel
Flow Area (*A*), m^2	529.952	1040.263	1570.215
Proportion of total area	0.34	0.66	1
Wetted Perimeter, m	106.052	303.214	405.566
Hydraulic Radius (*R*), m	4.997	3.431	3.904
Manning's roughness, (*n*) *	0.03	0.035	0.0325
Average Channel Slope (*S*), m/m	0.0024	0.0017	0.0019

* Manning's Roughness (n) recommended by ES

Manning Roughness Coefficient (*n*)

The Manning Roughness Coefficient was based on recommendations given by Colin Young at ES (Young, 2019). A value of 0.03 was used for the main channel and 0.035 for the flood berm (Table 2). These values are consistent with what would be expected in other New Zealand rivers of this nature.

Channel Slope (*S*)

The Channel slope has been calculated from the cross-section data, using the 12D software package. As mentioned above the channel has been split into two parts: the main channel and the flood berm. Three long sections are presented in Figure 6, these roughly represent the reaches for which the channel slope is needed to undertake the Mannings analysis. In the 12D model, several bearing vectors were plotted between cross-sections for each of the long-sections in question.



Figure 7: *Example of channel slope analysis using 12D.*

Figure 7 shows an example of this for long section 1, with the green line representing the bearing vector between two points on cross-sections 21 and 24. 12D calculates the slope of the straight line between the two points, in this example, the slope is 0.225% that is 0.00225 metres of fall per metre of horizontal (m/m). For each of the long-sections in Figure 6, several vectors were plotted to get an idea of the range of possible slopes between the cross-sections. The results for the slope are quite consistent, but there are some outlier values that have not been used in the averaging for further analysis. The slope analysis results are shown in Table 3 below:

Table 3: Long-section slope analysis results from 12D

	Long-section 1 (berm)			Long-section 2 (berm)			Long-section 3 (main channel)		
Slope sample	**Grade (%)**	**Vertical fall in horizontal**	**Slope (*S*), m/m**	**Grade (%)**	**Vertical fall in horizontal**	**Slope (*S*), m/m**	**Grade (%)**	**Vertical fall in horizontal**	**Slope (*S*), m/m**
1	0.205	488.441	0.00205	0.348	287.283	0.00348	0.026*	3849.633*	0.00026*
2	0.181	552.916	0.00181	0.345	290.128	0.00345	0.217	461.405	0.00217
3	0.176	568.137	0.00176	0.484	206.787	0.00484	0.731*	136.796*	0.00731*
4	0.150	667.439	0.00150	0.376	265.97	0.00376	0.111	897.227	0.00112
5	0.225	444.680	0.00225	0.392	255.367	0.00392	0.266	375.744	0.00266
6	0.080*	1257.640*	0.00080*	0.440	227.295	0.00440	0.099	1007.526	0.00099
Average	0.170	663.209	**0.00151**	0.398	255.472	**0.00397**	0.173	685.476	**0.00173**

* Outlier not used in averaging

4. Results

The velocities for the two sections of the channel have been calculated using the two methods described above: that is back-calculation using measured flow, and the Manning method.

Back calculation from the measured flow

The 1978 flood used for this analysis was recorded at the Mataura River at the Gore flow monitoring site, approximately 650m downstream of the proposed bridge location. It can be assumed that there are no flow inputs of any significance between the two locations, and therefore, the flow at the bridge site is the same as for the monitoring site. Based on this assumption, and the fact that we can calculate the wetted area of flow in the channel at the bridge site we can use the relationship:

$$V = Q/A$$

where: V = Average velocity (m/s) Q = Flow (m^3/s) A = Cross-section area (m^2)

This then allows for the calculation of the average velocity in the channel. This calculation is very much reliant on the apportionment of flow to the different sections of the channel, which may not be a completely linear relationship. That is to say, that even though the main channel may be a lesser proportion of the total area of the cross-section, it may actually carry the majority of the flow. To account for this skewed flow, three methods of apportioning flow to the cross-section were applied and the results collated in Table 4.

Table 4: Back calculated average velocities using different flow apportioning approaches.

Flow portions	Main channel			Flood berm			Total channel		
	Flow (*Q*), m^3/s	Flow Area (*A*), m^2	Average Velocity (*V = Q/A*), m/s	Flow (*Q*), m^3/s	Flow Area (*A*), m^2	Average Velocity (*V = Q/A*), m/s	Flow (*Q*), m^3/s	Flow Area (*A*), m^2	Average Velocity (*V = Q/A*), m/s
Total width							2297	1570	**1.46**
34% main channel/66% flood berm*	775	530	**1.46**	1522	1040	**1.46**			
50% main channel/50% flood berm	1149	530	**2.17**	1149	1040	**1.10**			
75% main channel/25% flood berm	1723	530	**3.25**	574	1040	**0.55**			

* Directly proportional to the total area

The results above, show that using this method the flow velocities across the flood berm average between 0.55 m/s and 1.46 m/s. The value of 0.55 m/s seems to be on the low side, but the corresponding main channel average of 3.35 m/s seems to be reasonable, and as expected. Conversely, the other flow apportionments result in berm velocities that are more like that expected, but the main channel velocities again seem too low. Advice from ES (Young, 2019) on this matter, was that velocities of 2-3 m/s could be expected across the flood berm. The berm is around 300m wide and is not homogenous all the way across its profile, with various terrain changes, fences and vegetation. These will result in varying velocities across the width. So, although the average berm velocity using this method was estimated as 0.55 m/s, it is possible that in parts of the flood berm, the velocities could reach 2-3 m/s and should be designed for.

Velocity calculated from geometry using the Manning equation

The Manning formula is an empirical formula widely used for estimating the average velocity of a liquid flowing in an open channel. It is largely based on channel geometry and bed material characteristics. One advantage of using the Manning formula method is that it is not reliant on a judgement of how the flow is apportioned across the cross-section, as has been done above. The Manning formula for estimating velocity is:

$$V = R^{2/3}S^{1/2}/n$$

Where: V = Average velocity (m/s) R = Hydraulic Radius (m)
S = Channel slope (m/m) n = Manning's roughness coefficient

The various parameters required to calculate velocity using the Manning equation have been evaluated from the available data as described in Section 0 of this report. Full results are available in the attached spreadsheet and are summarised in Table 5 below.

Table 5: *Summary of velocity calculations using Manning formula*

Slope Sample	Main channel				Flood berm				Total channel			
	Hydraulic radius (*R*) m	Slope (*S*) m/m	Roughness (*n*)	Average Velocity (*V*) m/s	Hydraulic radius (*R*) m	Slope (*S*) m/m	Roughness (*n*)	Average Velocity (*V*) m/s	Hydraulic radius (*R*) m	Slope (*S*) m/m	Roughness (*n*)	Average Velocity (*V*) m/s
1	4.997	0.00026*	0.03	1.57*	3.431	0.00205	0.035	2.94	3.904	0.00253	0.0325 **	3.83
2		0.00217		4.54		0.00181		2.76				
3		0.00731*		8.33*		0.00176		2.73				
4		0.00112		3.26		0.00150		2.52				
5		0.00266		5.03		0.00225		3.08				
6		0.00099		3.07		0.00080*		1.83*				
Ave.	4.997	0.00173	0.03	**4.06**	3.431	0.00187	0.035	**2.81**	3.904	0.00253	0.0325	**3.83**

* Outlier not used in averaging

**Average of main channel and berm

The velocity as calculated using the Manning formula is proving sensitive to the channel slope. Channel slope is a difficult parameter to evaluate as a single number based on cross-section information alone. However, when the slope is averaged across the samples, the velocity values calculated appear to be sensible. Results in Table 5 show that the average velocity in the main channel is 4.06 m/s and across the berm is 2.81 m/s. The main channel value of 4.06 m/s is consistent with the sort of velocities that would induce the standing waves, observed in the channel during floods (Figure 4). The average velocity calculated across the berm at 2.82 m/s is also consistent with the advice received from ES (Young, 2019), and the observations in aerial flood photos. These velocities again appear to be sensible. However, when the calculated velocity is used to calculate the theoretical flow for the whole channel, then the resultant flow is 6,021 m^3/s, which is much higher than the actual measured flow. This discrepancy highlights the uncertainty around this type of analysis.

5. Conclusions

It is difficult to say with any high level of certainty, what the velocities around the proposed location of the right bank end of the bridge structure will be in any flood event. The best estimate is that velocities will be between 1 m/s and 3 m/s during a flood event similar in magnitude to the 1978 flood. These are not inconsiderable velocities, and when combined with a potential water depth of 3 – 3.5 m, the force that could be applied to a structure by the flow is considerable.

6. References

Environment Southland. (2019, August 7th). *Beacon - Natural Hazards*. Retrieved from Beacon: http://gis.es.govt.nz/index.aspx?app=natural-hazards

Environment Southland. (2019, August 7th). *Mataura River at Gore*. Retrieved from Beacon: http://envdata.es.govt.nz/index.aspx?c=flow&tab=hydro

Young, C. (2019, July 12th). Environment Southland, Technical Services Engineer.

7. Quality Assurance

This report prepared, checked and approved by the following people:

Activity	Person	Position	Signed
Prepared by	Nick Boyens	Hydrologist	
Checked by	Martin Cockitt	Engineer	
Approved for release by	Hamish Weir	Project Manager	

Versions

Version	Stage	Date
1.	Draft for comment	11/8/2019
2.	Final	14/8/2019

APPENDIX B

Hydraulic Analysis

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Meeting Minutes – Longford Shared Path Proposal

Present: ✓	Apology: ✓	Copy: ✓	Name:	Company:
✓			Eli Maynard (EGM)	Riley Consultants Ltd
✓			Hamish Weir (HW)	Landpro
✓			Colin Young (CY)	Environment Southland
		✓	Dan Crocker	DC Structures Studio
		✓	Tim Muller	Landpro
		✓	Hashem Ramezan-zadeh	Gore District Council
Recorded by:		Eli Maynard		
Date:		27 November 2019 10:50am	**Pages:**	2
Subject:		**Hydraulic inputs for proposed Longford Shared Path**		
Project Reference:		**2019/11**		

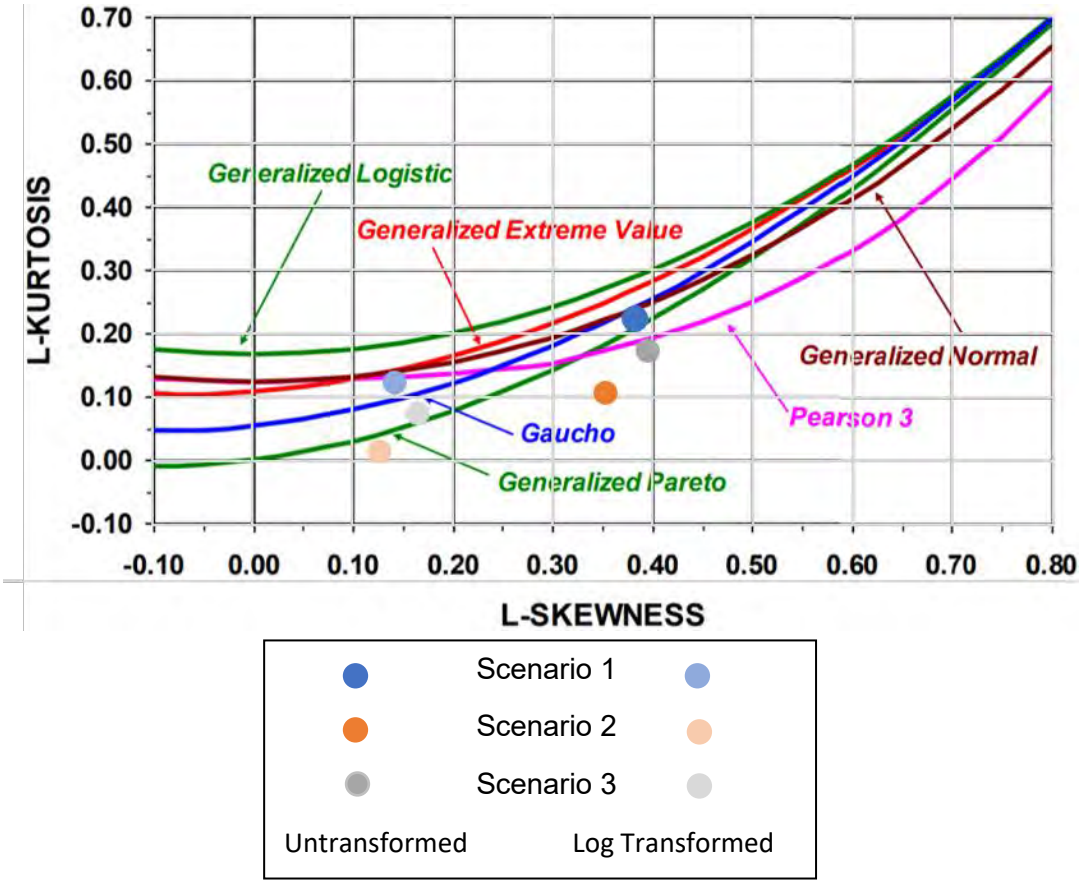
Summary of Discussion	
1	EGM outlined the purpose of the meeting, being to discuss the bridge preliminary design and hydraulic modelling work to date. He presented the preliminary design drawings on screen and spoke to the advantages of utilising a cable-stayed bridge from a hydraulic perspective in that no piers would be required within the main channel. CY agreed, commenting that a structure of this kind and the shape of the embankment earthworks indicated on the drawings was the preferred option to minimise disruption in the channel. CY indicated that the proposed location appeared to be the most sensible of the three originally considered.
2	EGM presented example 2D hydraulic modelling animations on screen along with corresponding water levels. He indicated that design flood events up to a 500-year flood were considered in accordance with the NZTA Bridge Manual, including allowances for climate change. He spoke to key outputs from the hydraulic model, including: 2.1 The observed variation in flow distribution and velocities across the river, with the main channel carrying proportionally more, and faster, flow compared to the floodplain (although this effect lessens at greater floods). CY advised that the modelled outputs were in line with his expectations, based on his knowledge and experience with the Mataura River at this location. He discussed the historic observations of standing waves within the main channel, and turbulence around obstructions on the floodplain in major floods. 2.2 Comparison of estimated flood levels with and without the bridge in place indicates very little, if any, changes in the river's flow regime as a result of the proposed bridge. EGM discussed that the right abutment earthworks made up only a small proportion (<5%) of the floodplain area, and it tended to direct a slightly greater (few %) proportion of flow towards the main channel with a corresponding decrease in overall flood levels – albeit a barely measurable affect overall. CY suggested that the estimated changes in levels appeared small and are probably within the error of the modelling calculations. EGM agreed. 2.3 Flood depths and velocities were slightly above those previously measured but were still within CY's expectations. 2.4 Stopbanks on both sides of the river are overtopped for floods in excess of the 100-year. EGM advised that in his view the stopbanks would be unlikely to tolerate sustained overtopping, particularly during a 500-year flood (ULS flood event considered), and hence the stopbank crest provided a reasonable upper level unless they were to be raised in future. He proposed that the bridge soffit level should be set at slightly above the existing floodbank crest level, but not considerably more as this may negatively impact the approach earthworks. CY agreed that that would be reasonable and provided useful background information regarding the stopbanks in question (summarised below).

	Summary of Discussion
3	EGM indicated that scour protection had been considered for the western approach embankment to accommodate inundation and the high velocities estimated. Large diameter riprap, in the order of D^{50}~0.9m had been preliminarily sized, which would need to be locally sourced for this option to be economic. EGM indicated that this size of riprap was fairly typical in major rivers nearby, to which CY agreed. The approach earthworks itself would probably be constructed of imported granular fill rather than site-won material, which would require new consents for gravel abstraction which might be problematic to obtain.
4	CY indicated that the stopbanks protect up to the 1978 flood level (flood of record) with minimal freeboard. He advised that a portion of the left stopbank between Huron and Oxford, which is coincident with the proposed bridge's left abutment, was reconstructed and raised in c.1986 to remove several tile drains and improve performance generally. The 1978 flood plus 0.5m freeboard was chosen as the basis for the raising design, putting its crest ~0.5m+ higher than on the true right. CY advised that any future raising of the stopbanks (on either side) would likely be considered on the basis of asset condition and a risk-based approach. There is a reasonable chance that this would be undertaken in the next 20 to 30 years, i.e. within the design life of the bridge. EGM then advised that some reasonable allowance for future stopbank raising may be prudent, or the bridge be designed in such a way that it could accommodate future raising, both of which will be considered as part of detailed design.
5	CY advised that Environment Southland had historically considered options to improve the flood conveyance of the Mataura River through Gore, including: deepening the main river channel, raising the stopbanks or providing a spillway/bypass possible near Hamilton and Church Street.
6	EGM discussed the basis for adopted design floods (in accordance with the NZTA Bridge Manual) and that Environment Southland had provided updated flood estimations based on review of the full flood record. CY advised caution on apply the latest values, which are slightly less than those historically reported. This may be due to the cyclic pattern to historic flooding, which tended to occur in 30-year cycles, and the fact that several major floods occurred during the late 70s and early 80s (e.g. 1977, 78, 84 ,87, 99) but only a single event since (in 2010). Because of this, recent revisions to flood frequency analysis may have downgraded the 'true' flood risk. He also provided a brief overview of other hydrological considerations involving the southern oscillation index and El Niño.
7	EGM queried whether CY had observed any debris being carried down the Mataura in historic floods, i.e. such as that occurring Pyramid Bridge in early 2018. CY advised that debris was often carried through this section of the river, including full willow trees. EGM advised that this will be considered as part of detailed design, and that the higher freeboard requirement will likely be adopted.
8	EGM and CY discussed suitable peer reviewers for the hydraulic design work including WSP, DHI and GeoSolve.
9	EGM advised that the 50-year flood, without allowance for climate change, was selected as the basis for the construction flood. CY indicated that >24 hours warning of an impending flood would be reasonable, based on Metservice warning. ES does not provide early warning for specific projects, and it would be the Contractor's responsibility to monitor Metservice for any impending floods. A trigger level of 50mm to 70mm falling at Fairlight might serve as a reasonable basis.
10	HW outlined the expectation that bylaw approval would be sought due to the right abutment earthworks being within the floodway. CY advised that current Bylaw has expired but that a new one is being ushered in and we would need to seek approval under this.
11	HW questioned whether Environment Southland would have any expectations on surfacing to the embankment and/or access track. CY advised that options such as asphalt or compacted gravel would likely be sufficient. CY also advised that his expectation would be that the path would be flush to the existing surface so as not to form an impediment to flood water. HW advised that locally sourced dunite running course has performed well elsewhere and could be used here.
12	HW advised that some minor drainage to the connecting pathway would be considered in the design. Discussions about the pathways crossing a small drainage ditch lead CY to suggest that a culvert here should to be designed as a submerged weir to accommodate flow from both directions.

Notes:

L moment Analysis Results

	Moment	Scenario 1	Scenario 2	Scenario 3
Untransformed	L1	645.081	791.785	676.704
	L2	202.645	296.74	240.772
	t3	0.382	0.352	0.395
	t4	0.224	0.107	0.175
Log-transformed	L1	6.319	6.465	6.326
	L2	0.296	0.374	0.340
	t3	0.142	0.126	0.165
	t4	0.123	0.015	0.075



Flood Frequency Analysis Results

Scenario 1: Generalised Extreme Value 1960-2018

Hilltop hydro version 5.78					
Maxima at Mataura at Gore					
From 5-Aug-1960 23:25:00 to 21-Nov-2018 20:05:00					
GEV exponent -0.30574 has magnitude greater than					
0.06586 and so GEV fits better than Gumbel					
Location = 443. Scale = 201. Shape =-0.306		100yr/2.33yr = 4.327			
Data selected from months 6 to 5 inclusive					
			-- GEV Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			5213.11	0.001	1000
			4176.13	0.002	500
			3102.14	0.005	200
			2466.91	0.01	100
1-Jun-78	14/10/1978 22:30	2108.49	2108.49	0.016	62.7
			1951.69	0.02	50
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.022	45
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.026	38.5
1-Jun-79	17/01/1980 21:25	1592.56	1592.56	0.036	27.9
			1414.99	0.05	20
1-Jun-67	14/04/1968 12:10	1295.09	1295.09	0.064	15.7
1-Jun-72	26552	1184.1	1184.1	0.081	12.3
			1093.32	0.1	10
1-Jun-76	4/05/1977 8:55	1071.19	1071.19	0.105	9.5
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.139	7.2
1-Jun-77	31/10/1977 2:55	958.976	958.976	0.139	7.2
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.144	7
1-Jun-13	3/06/2013 22:30	929.601	929.601	0.151	6.6
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.151	6.6
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.183	5.5
			825.418	0.2	5
1-Jun-15	24/05/2016 10:40	801.624	801.624	0.214	4.7
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.235	4.2
1-Jun-05	24/12/2005 4:20	712.854	712.854	0.277	3.6
1-Jun-66	27/04/1967 23:45	712.077	712.077	0.278	3.6
1-Jun-18	21/11/2018 20:05	710.093	710.093	0.28	3.6
1-Jun-69	27/12/1969 11:45	697.96	697.96	0.29	3.4
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.3	3.3
1-Jun-16	24/01/2017 4:20	646.013	646.013	0.34	2.9
1-Jun-11	24/02/2012 22:15	644.877	644.877	0.341	2.9
1-Jun-10	8/02/2011 10:30	578.999	578.999	0.418	2.4
1-Jun-68	24/10/1968 9:15	575.713	575.713	0.422	2.4
1-Jun-04	23/12/2004 7:55	570.2	570.2	0.429	2.3
			570.066	0.43	2.33
1-Jun-12	11/01/2013 5:45	566.205	566.205	0.435	2.3
1-Jun-62	30/07/1962 5:50	556.247	556.247	0.448	2.2
1-Jun-75	2/08/1975 20:10	546.906	546.906	0.462	2.2
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.542	1.8
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.559	1.8
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.573	1.7
1-Jun-09	27/04/2010 17:35	475.447	475.447	0.575	1.7
1-Jun-60	5/08/1960 23:25	472.036	472.036	0.581	1.7
1-Jun-17	3/02/2018 0:25	470.207	470.207	0.584	1.7
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.607	1.6
1-Jun-71	9/06/1971 5:20	456.484	456.484	0.608	1.6
1-Jun-63	24/06/1963 7:50	433.968	433.968	0.649	1.5
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.65	1.5
1-Jun-65	24/06/1965 16:50	426.446	426.446	0.663	1.5
1-Jun-14	3/08/2014 22:00	418.734	418.734	0.678	1.5
1-Jun-61	30/11/1961 10:05	398.167	398.167	0.717	1.4
1-Jun-08	14/05/2009 0:10	390.372	390.372	0.732	1.4
1-Jun-70	18/09/1970 0:45	384.278	384.278	0.744	1.3
1-Jun-00	12/10/2000 20:10	378.976	378.976	0.754	1.3
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.759	1.3
1-Jun-01	26/08/2001 23:40	372.437	372.437	0.766	1.3
1-Jun-07	17/10/2007 21:50	364.05	364.05	0.782	1.3
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.785	1.3
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.823	1.2
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.837	1.2
1-Jun-06	1/12/2006 9:50	333.519	333.519	0.838	1.2
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.839	1.2
1-Jun-02	21/09/2002 1:50	330.548	330.548	0.843	1.2
1-Jun-03	6/05/2004 9:20	311.384	311.384	0.875	1.1
1-Jun-64	6/03/1965 5:55	297.191	297.191	0.897	1.1
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.924	1.1
1-Jun-97	11/07/1997 3:40	254.147	254.147	0.952	1.1
1-Jun-74	28/02/1975 16:00	246.447	246.447	0.96	1
1-Jun-73	7/09/1973 6:50	200.468	199.786	0.989	1
		Mean = 645.081			

Scenario 1: Log Pearson Type 3 1960-2018

Hilltop Hydro 5.76					
Volume (m3/day) at Mataura at Gore					
From 5-Aug-1960 23:25:00 to 21-Nov-2018 20:05:00					
Location = 6.32 Scale = 0.538 Shape = 0.866 100yr/2.33yr = 4.769					
Data selected from months 6 to 5 inclusive					
			-- PE3 Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			5731.6	0.001	1000.00
			4595.2	0.002	500.00
			3403.5	0.005	200.00
			2691.4	0.01	100.00
			2110.1	0.02	50.00
1-Jun-78	14/10/1978 22:30	2108	2108	0.02	50.00
1-Jun-86	11/03/1987 6:55	1882	1882	0.027	37.04
1-Jun-99	18/11/1999 8:20	1783	1783	0.032	31.25
1-Jun-79	17/01/1980 21:25	1592	1592	0.043	23.26
			1502.6	0.05	20.00
1-Jun-67	14/04/1968 12:10	1295	1295	0.073	13.70
1-Jun-72	26552	1184	1184	0.091	10.99
			1139.5	0.1	10.00
1-Jun-76	4/05/1977 8:55	1071	1071	0.116	8.62
1-Jun-80	6/06/1980 6:30	959	959	0.15	6.67
1-Jun-77	31/10/1977 2:55	958	958	0.15	6.67
1-Jun-82	11/03/1983 8:15	947	947	0.154	6.49
1-Jun-13	3/06/2013 22:30	929	929	0.161	6.21
1-Jun-93	21/02/1994 13:55	928	928	0.161	6.21
1-Jun-96	13/01/1997 9:55	857	857	0.192	5.21
			840.5	0.2	5.00
1-Jun-15	24/05/2016 10:40	801	801	0.221	4.52
1-Jun-83	28/01/1984 14:45	768	768	0.242	4.13
1-Jun-05	24/12/2005 4:20	712	712	0.281	3.56
1-Jun-66	27/04/1967 23:45	712	712	0.281	3.56
1-Jun-18	21/11/2018 20:05	710	710	0.283	3.53
1-Jun-69	27/12/1969 11:45	697	697	0.293	3.41
1-Jun-95	3/09/1995 0:40	686	686	0.302	3.31
1-Jun-16	24/01/2017 4:20	646	646	0.339	2.95
1-Jun-11	24/02/2012 22:15	644	644	0.341	2.93
1-Jun-10	8/02/2011 10:30	578	578	0.413	2.42
1-Jun-68	24/10/1968 9:15	575	575	0.416	2.40
1-Jun-04	23/12/2004 7:55	570	570	0.423	2.36
1-Jun-12	11/01/2013 5:45	566	566	0.428	2.34
			564.4	0.43	2.33
1-Jun-62	30/07/1962 5:50	556	556	0.44	2.27
1-Jun-75	2/08/1975 20:10	546	546	0.454	2.20
1-Jun-87	20/01/1988 18:40	494	494	0.53	1.89
1-Jun-94	4/07/1994 7:10	484	484	0.546	1.83
1-Jun-91	17/08/1991 17:00	476	476	0.56	1.79
1-Jun-09	27/04/2010 17:35	475	475	0.561	1.78
1-Jun-60	5/08/1960 23:25	472	472	0.566	1.77
1-Jun-17	3/02/2018 0:25	470	470	0.57	1.75
1-Jun-92	6/07/1992 8:25	457	457	0.592	1.69
1-Jun-71	9/06/1971 5:20	456	456	0.594	1.68
1-Jun-63	24/06/1963 7:50	433	433	0.636	1.57
1-Jun-88	14/09/1988 2:55	433	433	0.636	1.57
1-Jun-65	24/06/1965 16:50	426	426	0.649	1.54
1-Jun-14	3/08/2014 22:00	418	418	0.664	1.51
1-Jun-61	30/11/1961 10:05	398	398	0.703	1.42
1-Jun-08	14/05/2009 0:10	390	390	0.719	1.39
1-Jun-70	18/09/1970 0:45	384	384	0.731	1.37
1-Jun-00	12/10/2000 20:10	378	378	0.743	1.35
1-Jun-89	22/06/1989 20:00	376	376	0.747	1.34
1-Jun-01	26/08/2001 23:40	372	372	0.755	1.32
1-Jun-07	17/10/2007 21:50	364	364	0.772	1.30
1-Jun-90	19/02/1991 13:00	362	362	0.776	1.29
1-Jun-81	1/08/1981 7:40	341	341	0.818	1.22
1-Jun-06	1/12/2006 9:50	333	333	0.834	1.20
1-Jun-85	31/05/1986 19:40	333	333	0.834	1.20
1-Jun-84	18/10/1984 2:45	332	332	0.836	1.20
1-Jun-02	21/09/2002 1:50	330	330	0.84	1.19
1-Jun-03	6/05/2004 9:20	311	311	0.877	1.14
1-Jun-64	6/03/1965 5:55	297	297	0.902	1.11
1-Jun-98	7/08/1998 9:20	277	277	0.934	1.07
1-Jun-97	11/07/1997 3:40	254	254	0.965	1.04
1-Jun-74	28/02/1975 16:00	246	246	0.973	1.03
1-Jun-73	7/09/1973 6:50	200	200	0.998	1.00
		Mean = 644.6			

Scenario 2: Generalised Parento Distribution 1977-2000

Hilltop Hydro 5.76					
Maxima at Mataura at Gore					
From 1-Jan-1977 23:25:00 to 31-Dec-1999 20:05:00					
Location = 211. Scale = 557. Shape =-0.414E-01 100yr/2.33yr = 4.404					
Data selected from months 6 to 5 inclusive					
			-- GPA Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			4665.91	0.001	1000.00
			4159.18	0.002	500.00
			3511.25	0.005	200.00
			3037.19	0.01	100.00
			2576.54	0.02	50.00
1-Jun-78	14/10/1978 22:30	2108.49	2108.49	0.041	24.39
			1987.53	0.05	20.00
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.059	16.95
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.069	14.49
1-Jun-79	17/01/1980 21:25	1592.56	1592.56	0.094	10.64
			1556.57	0.1	10.00
			1137.81	0.2	5.00
1-Jun-77	4/05/1977 8:55	1071.19	1071.19	0.224	4.46
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.27	3.70
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.276	3.62
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.285	3.51
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.322	3.11
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.375	2.67
			689.609	0.43	2.33
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.432	2.31
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.604	1.66
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.615	1.63
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.623	1.61
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.645	1.55
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.672	1.49
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.744	1.34
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.763	1.31
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.791	1.26
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.802	1.25
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.804	1.24
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.886	1.13
1-Jun-97	11/07/1997 3:40	254.147	225.781	0.973	1.03
		Mean = 791.785			

Scenario 2: Log Generalised Parento Distribution 1977-2000

Hilltop Hydro 5.76					
Maxima at Mataura at Gore					
From 1-Jan-1977 23:25:00 to 31-Dec-1999 20:05:00					
Location = 5.51 Scale = 1.48 Shape = 0.553 100yr/2.33yr = 4.360					
Data selected from months 6 to 5 inclusive					
			-- GPA Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			3413.8	0.001	1000.00
			3321.01	0.002	500.00
			3136.73	0.005	200.00
			2933.27	0.01	100.00
			2658.49	0.02	50.00
			2168.44	0.05	20.00
1-Jun-78	14/10/1978 22:30	2108.49	2108.49	0.055	18.18
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.078	12.82
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.09	11.11
			1706.87	0.1	10.00
1-Jun-79	17/01/1980 21:25	1592.56	1592.56	0.117	8.55
			1201.56	0.2	5.00
1-Jun-77	4/05/1977 8:55	1071.19	1071.19	0.239	4.18
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.28	3.57
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.285	3.51
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.292	3.42
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.324	3.09
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.37	2.70
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.42	2.38
			672.737	0.43	2.33
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.582	1.72
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.593	1.69
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.602	1.66
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.624	1.60
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.654	1.53
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.735	1.36
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.757	1.32
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.792	1.26
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.806	1.24
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.809	1.24
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.922	1.08
1-Jun-97	11/07/1997 3:40	254.147	254.147	0.973	1.03
		Mean = 791.785			

Scenario 3: Generalised Parento Distribution 1968-2010

Hilltop Hydro 5.76					
Maxima at Mataura at Gore					
From 1-Jan-1968 23:25:00 to 31-Dec-2010 20:05:00					
Location = 227. Scale = 390. Shape =-0.133 100yr/2.33yr = 4.698					
Data selected from months 6 to 5 inclusive					
			-- GPA Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			4644.16	0.001	1000.0
			3996.33	0.002	500.0
			3226.88	0.005	200.0
			2704.17	0.01	100.0
			2227.61	0.02	50.0
1-Jun-78	14/10/1978 22:30	2108.49	2108.49	0.024	41.6
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.035	29.0
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.041	24.6
			1661.59	0.05	20.0
1-Jun-79	17/01/1980 21:25	1592.56	1592.56	0.056	17.7
1-Jun-68	14/04/1968 12:10	1295.09	1295.09	0.097	10.3
			1277.07	0.1	10.0
1-Jun-72	26552	1184.1	1184.1	0.119	8.4
1-Jun-76	4/05/1977 8:55	1071.19	1071.19	0.149	6.7
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.187	5.4
1-Jun-77	31/10/1977 2:55	958.976	958.976	0.187	5.3
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.192	5.2
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.199	5.0
			926.505	0.2	5.0
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.231	4.3
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.28	3.6
1-Jun-05	24/12/2005 4:20	712.854	712.854	0.316	3.2
1-Jun-69	27/12/1969 11:45	697.96	697.96	0.326	3.1
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.335	3.0
			575.591	0.43	2.3
1-Jun-04	23/12/2004 7:55	570.2	570.2	0.435	2.3
1-Jun-75	2/08/1975 20:10	546.906	546.906	0.459	2.2
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.518	1.9
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.531	1.9
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.541	1.8
1-Jun-09	27/04/2010 17:35	475.447	475.447	0.543	1.8
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.566	1.8
1-Jun-71	9/06/1971 5:20	456.484	456.484	0.567	1.8
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.599	1.7
1-Jun-08	14/05/2009 0:10	390.372	390.372	0.665	1.5
1-Jun-70	18/09/1970 0:45	384.278	384.278	0.675	1.5
1-Jun-00	12/10/2000 20:10	378.976	378.976	0.684	1.5
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.689	1.5
1-Jun-01	26/08/2001 23:40	372.437	372.437	0.695	1.4
1-Jun-07	17/10/2007 21:50	364.05	364.05	0.709	1.4
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.713	1.4
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.749	1.3
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.764	1.3
1-Jun-06	1/12/2006 9:50	333.519	333.519	0.765	1.3
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.767	1.3
1-Jun-02	21/09/2002 1:50	330.548	330.548	0.771	1.3
1-Jun-03	6/05/2004 9:20	311.384	311.384	0.808	1.2
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.879	1.1
1-Jun-97	11/07/1997 3:40	254.147	254.147	0.934	1.1
1-Jun-74	28/02/1975 16:00	246.447	246.447	0.952	1.1
1-Jun-73	7/09/1973 6:50	200.468	233.075	0.985	1.0
		Mean = 676.704			

Scenario 3: Log Generalised Parento Distribution 1968-2010

Hilltop Hydro 5.76					
Maxima at Mataura at Gore					
From 1-Jan-1968 23:25:00 to 31-Dec-2010 20:05:00					
Location = 5.50 Scale = 1.19 Shape = 0.435 100yr/2.33yr = 4.588					
Data selected from months 6 to 5 inclusive					
			-- GPA Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			3278.35	0.001	1000
			3125.73	0.002	500
			2857.63	0.005	200
			2596.09	0.01	100
			2280.18	0.02	50
1-Jun-78	14/10/1978 22:30	2108.49	2108.49	0.028	35.8
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.042	23.6
			1786.36	0.05	20
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.05	19.9
1-Jun-79	17/01/1980 21:25	1592.56	1592.56	0.07	14.4
			1375.64	0.1	10
1-Jun-68	14/04/1968 12:10	1295.09	1295.09	0.114	8.7
1-Jun-72	26552	1184.1	1184.1	0.138	7.3
1-Jun-76	4/05/1977 8:55	1071.19	1071.19	0.167	6
			966.341	0.2	5
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.202	4.9
1-Jun-77	31/10/1977 2:55	958.976	958.976	0.203	4.9
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.207	4.8
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.214	4.7
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.243	4.1
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.286	3.5
1-Jun-05	24/12/2005 4:20	712.854	712.854	0.318	3.1
1-Jun-69	27/12/1969 11:45	697.96	697.96	0.328	3.1
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.335	3
1-Jun-04	23/12/2004 7:55	570.2	570.2	0.426	2.3
			565.796	0.43	2.33
1-Jun-75	2/08/1975 20:10	546.906	546.906	0.448	2.2
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.503	2
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.515	1.9
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.524	1.9
1-Jun-09	27/04/2010 17:35	475.447	475.447	0.526	1.9
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.549	1.8
1-Jun-71	9/06/1971 5:20	456.484	456.484	0.55	1.8
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.582	1.7
1-Jun-08	14/05/2009 0:10	390.372	390.372	0.649	1.5
1-Jun-70	18/09/1970 0:45	384.278	384.278	0.659	1.5
1-Jun-00	12/10/2000 20:10	378.976	378.976	0.668	1.5
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.673	1.5
1-Jun-01	26/08/2001 23:40	372.437	372.437	0.68	1.5
1-Jun-07	17/10/2007 21:50	364.05	364.05	0.695	1.4
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.699	1.4
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.739	1.4
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.756	1.3
1-Jun-06	1/12/2006 9:50	333.519	333.519	0.757	1.3
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.759	1.3
1-Jun-02	21/09/2002 1:50	330.548	330.548	0.763	1.3
1-Jun-03	6/05/2004 9:20	311.384	311.384	0.807	1.2
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.894	1.1
1-Jun-97	11/07/1997 3:40	254.147	254.147	0.967	1
1-Jun-74	28/02/1975 16:00	246.447	246.447	0.992	1
1-Jun-73	7/09/1973 6:50	200.468	200.468	0.985	1
		Mean = 676.704			

Generalised Extreme Value 1960-1977

Hilltop hydro version 5.78					
Maxima at Mataura at Gore					
From 5-Jan-1960 23:25:00 to 21-Nov-1977 20:05:00					
GEV exponent -0.13813 has magnitude greater than					
0.11923 and so GEV fits better than Gumbel					
Location = 442. Scale = 224. Shape =-0.138 100yr/2.33yr = 3.262					
Data selected from months 6 to 5 inclusive					
			-- GEV Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			3028.36	0.001	1000
			2644.02	0.002	500
			2188.92	0.005	200
			1880.43	0.01	100
			1599.23	0.02	50
1-Jun-67	14/04/1968 12:10	1295.09	1295.09	0.046	21.9
			1263.82	0.05	20
1-Jun-72	26552	1184.1	1184.1	0.063	15.8
1-Jun-76	4/05/1977 8:55	1071.19	1071.19	0.089	11.3
			1032.7	0.1	10
1-Jun-77	31/10/1977 2:55	958.976	958.976	0.126	7.9
			815.004	0.2	5
1-Jun-66	27/04/1967 23:45	712.077	712.077	0.279	3.6
1-Jun-69	27/12/1969 11:45	697.96	697.96	0.292	3.4
			576.487	0.43	2.33
1-Jun-68	24/10/1968 9:15	575.713	575.713	0.431	2.3
1-Jun-62	30/07/1962 5:50	556.247	556.247	0.457	2.2
1-Jun-75	2/08/1975 20:10	546.906	546.906	0.47	2.1
1-Jun-60	5/08/1960 23:25	472.036	472.036	0.583	1.7
1-Jun-71	9/06/1971 5:20	456.484	456.484	0.608	1.6
1-Jun-63	24/06/1963 7:50	433.968	433.968	0.645	1.5
1-Jun-65	24/06/1965 16:50	426.446	426.446	0.658	1.5
1-Jun-61	30/11/1961 10:05	398.167	398.167	0.705	1.4
1-Jun-70	18/09/1970 0:45	384.278	384.278	0.728	1.4
1-Jun-64	6/03/1965 5:55	297.191	297.191	0.861	1.2
1-Jun-74	28/02/1975 16:00	246.447	246.447	0.921	1.1
1-Jun-73	7/09/1973 6:50	200.468	191.485	0.966	1
		Mean = 606.319			

Generalised Extreme Value 1968-2010

Hilltop hydro version 5.78					
Maxima at Mataura at Gore					
From 1-Jan-1968 23:25:00 to 1-Jan-2010 20:05:00					
GEV exponent -0.31642 has magnitude greater than					
0.07900 and so GEV fits better than Gumbel					
Location = 437. Scale = 238. Shape =-0.316 100yr/2.33yr = 4.951					
Data selected from months 6 to 5 inclusive					
			-- GEV Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			6379.78	0.001	1000
			5060.35	0.002	500
			3705.41	0.005	200
			2910.96	0.01	100
			2271.4	0.02	50
1-Jun-78	14/10/1978 22:30	2108.49	2108.49	0.025	40.8
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.033	30.1
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.038	26.1
			1610.87	0.05	20
1-Jun-79	17/01/1980 21:25	1592.56	1592.56	0.051	19.4
1-Jun-68	14/04/1968 12:10	1295.09	1295.09	0.086	11.6
			1218.49	0.1	10
1-Jun-72	26552	1184.1	1184.1	0.107	9.3
1-Jun-76	4/05/1977 8:55	1071.19	1071.19	0.135	7.4
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.172	5.8
1-Jun-77	31/10/1977 2:55	958.976	958.976	0.172	5.8
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.177	5.6
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.185	5.4
			894.266	0.2	5
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.218	4.6
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.271	3.7
1-Jun-05	24/12/2005 4:20	712.854	712.854	0.311	3.2
1-Jun-69	27/12/1969 11:45	697.96	697.96	0.323	3.1
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.333	3
			587.95	0.43	2.33
1-Jun-04	23/12/2004 7:55	570.2	570.2	0.45	2.2
1-Jun-75	2/08/1975 20:10	546.906	546.906	0.478	2.1
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.547	1.8
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.562	1.8
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.573	1.7
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.601	1.7
1-Jun-71	9/06/1971 5:20	456.484	456.484	0.603	1.7
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.638	1.6
1-Jun-08	14/05/2009 0:10	390.372	390.372	0.706	1.4
1-Jun-70	18/09/1970 0:45	384.278	384.278	0.716	1.4
1-Jun-00	12/10/2000 20:10	378.976	378.976	0.725	1.4
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.729	1.4
1-Jun-01	26/08/2001 23:40	372.437	372.437	0.735	1.4
1-Jun-07	17/10/2007 21:50	364.05	364.05	0.749	1.3
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.752	1.3
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.784	1.3
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.797	1.3
1-Jun-06	1/12/2006 9:50	333.519	333.519	0.798	1.3
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.799	1.3
1-Jun-02	21/09/2002 1:50	330.548	330.548	0.802	1.2
1-Jun-03	6/05/2004 9:20	311.384	311.384	0.832	1.2
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.88	1.1
1-Jun-97	11/07/1997 3:40	254.147	254.147	0.91	1.1
1-Jun-74	28/02/1975 16:00	246.447	246.447	0.919	1.1
1-Jun-73	7/09/1973 6:50	200.468	162.829	0.985	1
		Mean = 681.612			

Generalised Extreme Value 1979-2000

Hilltop hydro version 5.78					
Maxima at Mataura at Gore					
From 1-Jan-1979 23:25:00 to 1-Jan-2000 20:05:00					
GEV exponent -0.35025 has magnitude greater than					
0.11311 and so GEV fits better than Gumbel					
Location = 441. Scale = 238. Shape =-0.350 100yr/2.33yr = 5.337					
Data selected from months 6 to 5 inclusive					
			-- GEV Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			7408.07	0.001	1000
			5758.64	0.002	500
			4109.8	0.005	200
			3169.38	0.01	100
			2429.94	0.02	50
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.038	26.2
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.044	22.9
			1686.76	0.05	20
1-Jun-80	17/01/1980 21:25	1592.56	1592.56	0.057	17.4
			1257.59	0.1	10
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.185	5.4
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.193	5.2
			911.654	0.2	5
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.226	4.4
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.278	3.6
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.34	2.9
			593.865	0.43	2.33
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.554	1.8
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.568	1.8
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.579	1.7
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.608	1.6
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.644	1.6
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.736	1.4
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.759	1.3
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.792	1.3
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.805	1.2
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.807	1.2
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.888	1.1
1-Jun-97	11/07/1997 3:40	254.147	200.668	0.969	1
		Mean = 703.57			

Generalised Extreme Value 2000-2018

Hilltop hydro version 5.78					
Maxima at Mataura at Gore					
From 5-Jan-2000 23:25:00 to 21-Nov-2018 20:05:00					
GEV exponent -0.02318 has magnitude less than 0.11605					
and so Gumbel fits better than GEV					
Location = 440. Scale = 144. Shape =-0.232E-01 100yr/2.33yr = 2.176					
Data selected from months 6 to 5 inclusive					
			-- GEV Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			1519.7	0.001	1000
			1403.3	0.002	500
			1252.15	0.005	200
			1139.7	0.01	100
			1028.65	0.02	50
1-Jun-13	3/06/2013 22:30	929.601	929.601	0.037	26.8
			883.228	0.05	20
1-Jun-15	24/05/2016 10:40	801.624	801.624	0.084	12
			773.012	0.1	10
1-Jun-05	24/12/2005 4:20	712.854	712.854	0.145	6.9
1-Jun-18	21/11/2018 20:05	710.093	710.093	0.148	6.8
			660.051	0.2	5
1-Jun-16	24/01/2017 4:20	646.013	646.013	0.217	4.6
1-Jun-11	24/02/2012 22:15	644.877	644.877	0.219	4.6
1-Jun-10	8/02/2011 10:30	578.999	578.999	0.32	3.1
1-Jun-04	23/12/2004 7:55	570.2	570.2	0.336	3
1-Jun-12	11/01/2013 5:45	566.205	566.205	0.343	2.9
			523.822	0.43	2.33
1-Jun-09	27/04/2010 17:35	475.447	475.447	0.543	1.8
1-Jun-17	3/02/2018 0:25	470.207	470.207	0.556	1.8
1-Jun-14	3/08/2014 22:00	418.734	418.734	0.686	1.5
1-Jun-08	14/05/2009 0:10	390.372	390.372	0.757	1.3
1-Jun-00	12/10/2000 20:10	378.976	378.976	0.784	1.3
1-Jun-01	26/08/2001 23:40	372.437	372.437	0.799	1.3
1-Jun-07	17/10/2007 21:50	364.05	364.05	0.817	1.2
1-Jun-06	1/12/2006 9:50	333.519	333.519	0.879	1.1
1-Jun-02	21/09/2002 1:50	330.548	330.548	0.884	1.1
1-Jun-03	6/05/2004 9:20	311.384	265.046	0.968	1
		Mean = 526.639			

Generalised Extreme Value 1960-2000

Hilltop hydro version 5.78					
Maxima at Mataura at Gore					
From 5-Jun-1960 23:25:00 to 30-May-2000 20:05:00					
GEV exponent -0.28498 has magnitude greater than					
0.07998 and so GEV fits better than Gumbel					
Location = 460. Scale = 251. Shape =-0.285 100yr/2.33yr = 4.607					
Data selected from months 6 to 5 inclusive					
			-- GEV Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			5875.01	0.001	1000
			4745.99	0.002	500
			3557.16	0.005	200
			2842.01	0.01	100
			2253.52	0.02	50
1-Jun-78	14/10/1978 22:30	2108.49	2108.49	0.024	41.2
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.034	29.8
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.039	25.6
			1630.17	0.05	20
1-Jun-79	17/01/1980 21:25	1592.56	1592.56	0.053	18.8
1-Jun-67	14/04/1968 12:10	1295.09	1295.09	0.091	10.9
			1250.02	0.1	10
1-Jun-72	26552	1184.1	1184.1	0.114	8.8
1-Jun-76	4/05/1977 8:55	1071.19	1071.19	0.145	6.9
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.186	5.4
1-Jun-77	31/10/1977 2:55	958.976	958.976	0.186	5.4
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.192	5.2
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.2	5
			928.573	0.2	5
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.237	4.2
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.294	3.4
1-Jun-66	27/04/1967 23:45	712.077	712.077	0.338	3
1-Jun-69	27/12/1969 11:45	697.96	697.96	0.35	2.9
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.361	2.8
			616.857	0.43	2.33
1-Jun-68	24/10/1968 9:15	575.713	575.713	0.477	2.1
1-Jun-62	30/07/1962 5:50	556.247	556.247	0.5	2
1-Jun-75	2/08/1975 20:10	546.906	546.906	0.512	2
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.582	1.7
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.596	1.7
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.607	1.6
1-Jun-60	5/08/1960 23:25	472.036	472.036	0.614	1.6
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.636	1.6
1-Jun-71	9/06/1971 5:20	456.484	456.484	0.637	1.6
1-Jun-63	24/06/1963 7:50	433.968	433.968	0.67	1.5
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.671	1.5
1-Jun-65	24/06/1965 16:50	426.446	426.446	0.682	1.5
1-Jun-61	30/11/1961 10:05	398.167	398.167	0.725	1.4
1-Jun-70	18/09/1970 0:45	384.278	384.278	0.746	1.3
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.758	1.3
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.779	1.3
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.809	1.2
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.821	1.2
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.823	1.2
1-Jun-64	6/03/1965 5:55	297.191	297.191	0.871	1.1
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.895	1.1
1-Jun-97	11/07/1997 3:40	254.147	254.147	0.922	1.1
1-Jun-74	28/02/1975 16:00	246.447	246.447	0.929	1.1
1-Jun-73	7/09/1973 6:50	200.468	165.939	0.984	1
		Mean = 701.34			

Generalised Extreme Value 1980-2018

Hilltop hydro version 5.78					
Maxima at Mataura at Gore					
From 5-Jun-1980 23:25:00 to 30-May-2019 20:05:00					
GEV exponent -0.30974 has magnitude greater than					
0.08100 and so GEV fits better than Gumbel					
Location = 430. Scale = 169. Shape =-0.310 100yr/2.33yr = 4.008					
Data selected from months 6 to 5 inclusive					
			-- GEV Distribution --		
12 mth Partition starts	Value Recorded at	Measured maximum		Prob. per year	Return Period
			4516.98	0.001	1000
			3621.39	0.002	500
			2696.79	0.005	200
			2151.7	0.01	100
1-Jun-86	11/03/1987 6:55	1882.01	1882.01	0.015	66.6
1-Jun-99	18/11/1999 8:20	1783.35	1783.35	0.018	56.6
			1710.83	0.02	50
			1253.06	0.05	20
			979.604	0.1	10
1-Jun-80	6/06/1980 6:30	959.968	959.968	0.106	9.5
1-Jun-82	11/03/1983 8:15	947.096	947.096	0.11	9.1
1-Jun-13	3/06/2013 22:30	929.601	929.601	0.115	8.7
1-Jun-93	21/02/1994 13:55	928.716	928.716	0.116	8.6
1-Jun-96	13/01/1997 9:55	857.293	857.293	0.143	7
1-Jun-15	24/05/2016 10:40	801.624	801.624	0.17	5.9
1-Jun-83	28/01/1984 14:45	768.336	768.336	0.19	5.3
			752.534	0.2	5
1-Jun-05	24/12/2005 4:20	712.854	712.854	0.229	4.4
1-Jun-18	21/11/2018 20:05	710.093	710.093	0.231	4.3
1-Jun-95	3/09/1995 0:40	686.231	686.231	0.251	4
1-Jun-16	24/01/2017 4:20	646.013	646.013	0.289	3.5
1-Jun-11	24/02/2012 22:15	644.877	644.877	0.29	3.5
1-Jun-10	8/02/2011 10:30	578.999	578.999	0.368	2.7
1-Jun-04	23/12/2004 7:55	570.2	570.2	0.38	2.6
1-Jun-12	11/01/2013 5:45	566.205	566.205	0.386	2.6
			536.816	0.43	2.33
1-Jun-87	20/01/1988 18:40	494.683	494.683	0.502	2
1-Jun-94	4/07/1994 7:10	484.532	484.532	0.521	1.9
1-Jun-91	17/08/1991 17:00	476.779	476.779	0.536	1.9
1-Jun-09	27/04/2010 17:35	475.447	475.447	0.538	1.9
1-Jun-17	3/02/2018 0:25	470.207	470.207	0.548	1.8
1-Jun-92	6/07/1992 8:25	457.277	457.277	0.575	1.7
1-Jun-88	14/09/1988 2:55	433.532	433.532	0.625	1.6
1-Jun-14	3/08/2014 22:00	418.734	418.734	0.657	1.5
1-Jun-08	14/05/2009 0:10	390.372	390.372	0.721	1.4
1-Jun-00	12/10/2000 20:10	378.976	378.976	0.747	1.3
1-Jun-89	22/06/1989 20:00	376.184	376.184	0.753	1.3
1-Jun-01	26/08/2001 23:40	372.437	372.437	0.762	1.3
1-Jun-07	17/10/2007 21:50	364.05	364.05	0.781	1.3
1-Jun-90	19/02/1991 13:00	362.217	362.217	0.785	1.3
1-Jun-81	1/08/1981 7:40	341.981	341.981	0.829	1.2
1-Jun-85	31/05/1986 19:40	333.903	333.903	0.846	1.2
1-Jun-06	1/12/2006 9:50	333.519	333.519	0.847	1.2
1-Jun-84	18/10/1984 2:45	332.596	332.596	0.849	1.2
1-Jun-02	21/09/2002 1:50	330.548	330.548	0.853	1.2
1-Jun-03	6/05/2004 9:20	311.384	311.384	0.89	1.1
1-Jun-98	7/08/1998 9:20	277.98	277.98	0.944	1.1
1-Jun-97	11/07/1997 3:40	254.147	235.945	0.984	1
		Mean = 601.153			

~~~ Hilltop Hydro ~~~ Version 6.65.07 8-Jul-2019
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Source is G:\Hydrology\Official Hydrological Calcs\Hilltop Files for Analysis\Analysis 2018.hts
Flow (Cumecs) at Mataura River at Gore High Flow
From 29-Jul-1960 00:00:00 to 1-Jun-2019 00:00:00

Moments: L1 = 645.081 L2 = 202.645 T3 = 0.382 T4 = 0.224
Location = 443.316 Scale = 200.783 Shape = -0.306 100yr/2.33yr = 4.327
GEV exponent -0.30574 has magnitude greater than 0.06586 and so GEV fits better than Gumbel

Data selected from months 6 to 5 inclusive

12 mth partition starts	5.00 Minutes mean starts	maximum measured		-- G.E.V. Distribution -- 1.96 std. dev.	ann. prob. (1/y)	return period (y)
				5213.113	0.001	1000
				4176.137	0.002	500
				3102.140	0.005	200
				2466.907	0.010	100
1-Jun-1978	14-Oct-1978 22:30:00	2108.490	A	2108.490	0.016	62.7
				1951.692	0.020	50
1-Jun-1986	11-Mar-1987 06:55:00	1882.011	B	1882.011	0.022	45.0
1-Jun-1999	18-Nov-1999 08:20:00	1783.354	C	1783.354	0.026	38.5
1-Jun-1979	17-Jan-1980 21:25:00	1592.563	D	1592.563	0.036	27.9
				1414.995	0.050	20
1-Jun-1967	14-Apr-1968 12:10:00	1295.091	E	1295.091	0.064	15.7
1-Jun-1972	10-Sep-1972 00:00:00	1184.101	F	1184.101	0.081	12.3
				1093.316	0.100	10
1-Jun-1976	04-May-1977 08:55:00	1071.191	G	1071.191	0.105	9.5
1-Jun-1980	06-Jun-1980 06:30:00	959.968	H	959.968	0.139	7.2
1-Jun-1977	31-Oct-1977 02:55:00	958.977	I	958.977	0.139	7.2
1-Jun-1982	11-Mar-1983 08:15:00	947.097	J	947.097	0.144	7.0
1-Jun-2013	03-Jun-2013 22:30:00	929.601	K	929.601	0.151	6.6
1-Jun-1993	21-Feb-1994 13:55:00	928.716	L	928.716	0.151	6.6
1-Jun-1996	13-Jan-1997 09:55:00	857.293	M	857.293	0.183	5.5
				825.418	0.200	5
1-Jun-2015	24-May-2016 10:40:00	801.624	N	801.624	0.214	4.7
1-Jun-1983	28-Jan-1984 14:45:00	768.337	O	768.337	0.235	4.2
1-Jun-2005	24-Dec-2005 04:20:00	712.854	P	712.854	0.277	3.6
1-Jun-1966	27-Apr-1967 23:45:00	712.077	Q	712.077	0.278	3.6
1-Jun-2018	21-Nov-2018 20:05:00	710.093	R	710.093	0.280	3.6
1-Jun-1969	27-Dec-1969 11:45:00	697.961	S	697.961	0.290	3.4
1-Jun-1995	03-Sep-1995 00:40:00	686.232	T	686.232	0.300	3.3
1-Jun-2016	24-Jan-2017 04:20:00	646.014	U	646.014	0.340	2.9
1-Jun-2011	24-Feb-2012 22:15:00	644.878	V	644.878	0.341	2.9
1-Jun-2010	08-Feb-2011 10:30:00	578.999	W	578.999	0.418	2.4
1-Jun-1968	24-Oct-1968 09:15:00	575.714	X	575.714	0.422	2.4
1-Jun-2004	23-Dec-2004 07:55:00	570.200	Y	570.200	0.429	2.3
				570.066	0.430	2.33
1-Jun-2012	11-Jan-2013 05:45:00	566.206	Z	566.206	0.435	2.3
1-Jun-1962	30-Jul-1962 05:50:00	556.248	a	556.248	0.448	2.2
1-Jun-1975	02-Aug-1975 20:10:00	546.906	b	546.906	0.462	2.2
1-Jun-1987	20-Jan-1988 18:40:00	494.683	c	494.683	0.542	1.8

1-Jun-1987	20-Jan-1988 18:40:00	494.683	c	494.683	0.542	1.8
1-Jun-1994	04-Jul-1994 07:10:00	484.532	d	484.532	0.559	1.8
1-Jun-1991	17-Aug-1991 17:00:00	476.780	e	476.780	0.573	1.7
1-Jun-2009	27-Apr-2010 17:35:00	475.448	f	475.448	0.575	1.7
1-Jun-1960	05-Aug-1960 23:25:00	472.037	g	472.037	0.581	1.7
1-Jun-2017	03-Feb-2018 00:25:00	470.207	h	470.207	0.584	1.7
1-Jun-1992	06-Jul-1992 08:25:00	457.277	i	457.277	0.607	1.6
1-Jun-1971	09-Jun-1971 05:20:00	456.484	j	456.484	0.608	1.6
1-Jun-1963	24-Jun-1963 07:50:00	433.969	k	433.969	0.649	1.5
1-Jun-1988	14-Sep-1988 02:55:00	433.533	l	433.533	0.650	1.5
1-Jun-1965	24-Jun-1965 16:50:00	426.446	m	426.446	0.663	1.5
1-Jun-2014	03-Aug-2014 22:00:00	418.735	n	418.735	0.678	1.5
1-Jun-1961	30-Nov-1961 10:05:00	398.168	o	398.168	0.717	1.4
1-Jun-2008	14-May-2009 00:10:00	390.372	p	390.372	0.732	1.4
1-Jun-1970	18-Sep-1970 00:45:00	384.279	q	384.279	0.744	1.3
1-Jun-2000	12-Oct-2000 20:10:00	378.976	r	378.976	0.754	1.3
1-Jun-1989	22-Jun-1989 20:00:00	376.185	s	376.185	0.759	1.3
1-Jun-2001	26-Aug-2001 23:40:00	372.438	t	372.438	0.766	1.3
1-Jun-2007	17-Oct-2007 21:50:00	364.051	u	364.051	0.782	1.3
1-Jun-1990	19-Feb-1991 13:00:00	362.218	v	362.218	0.785	1.3
1-Jun-1981	01-Aug-1981 07:40:00	341.982	w	341.982	0.823	1.2
1-Jun-1985	31-May-1986 19:40:00	333.904	x	333.904	0.837	1.2
1-Jun-2006	01-Dec-2006 09:50:00	333.520	y	333.520	0.838	1.2
1-Jun-1984	18-Oct-1984 02:45:00	332.597	z	332.597	0.839	1.2
1-Jun-2002	21-Sep-2002 01:50:00	330.548	A	330.548	0.843	1.2
1-Jun-2003	06-May-2004 09:20:00	311.384	B	311.384	0.875	1.1
1-Jun-1964	06-Mar-1965 05:55:00	297.192	C	297.192	0.897	1.1
1-Jun-1998	07-Aug-1998 09:20:00	277.981	D	277.981	0.924	1.1
1-Jun-1997	11-Jul-1997 03:40:00	254.148	E	254.148	0.952	1.1
1-Jun-1974	28-Feb-1975 16:00:00	246.448	F	246.448	0.960	1.0
1-Jun-1973	07-Sep-1973 06:50:00	200.469	G	200.469	0.989	1.0
	Mean =	645.081				

Mataura River at Gore – High Flow Analysis Post February 2020 Flood

~~~ Hilltop Hydro ~~~ Version 6.65.18 12-May-2020
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Source is \\esfile\src_groups\Environmental_Info_Data\MAIN ARCHIVE\Environmental Data Archive.hts
Flow (m3/sec) at Mataura River at Gore
From 1-Jan-1960 12:00:00 to 31-Mar-2020 00:00:00

Moments: L1 = 663.708 L2 = 222.113 T3 = 0.404 T4 = 0.266
Location = 440.111 Scale = 209.813 Shape = -0.335 100yr/2.33yr = 4.772
GEV exponent -0.33487 has magnitude greater than 0.06477 and so GEV fits better than Gumbel

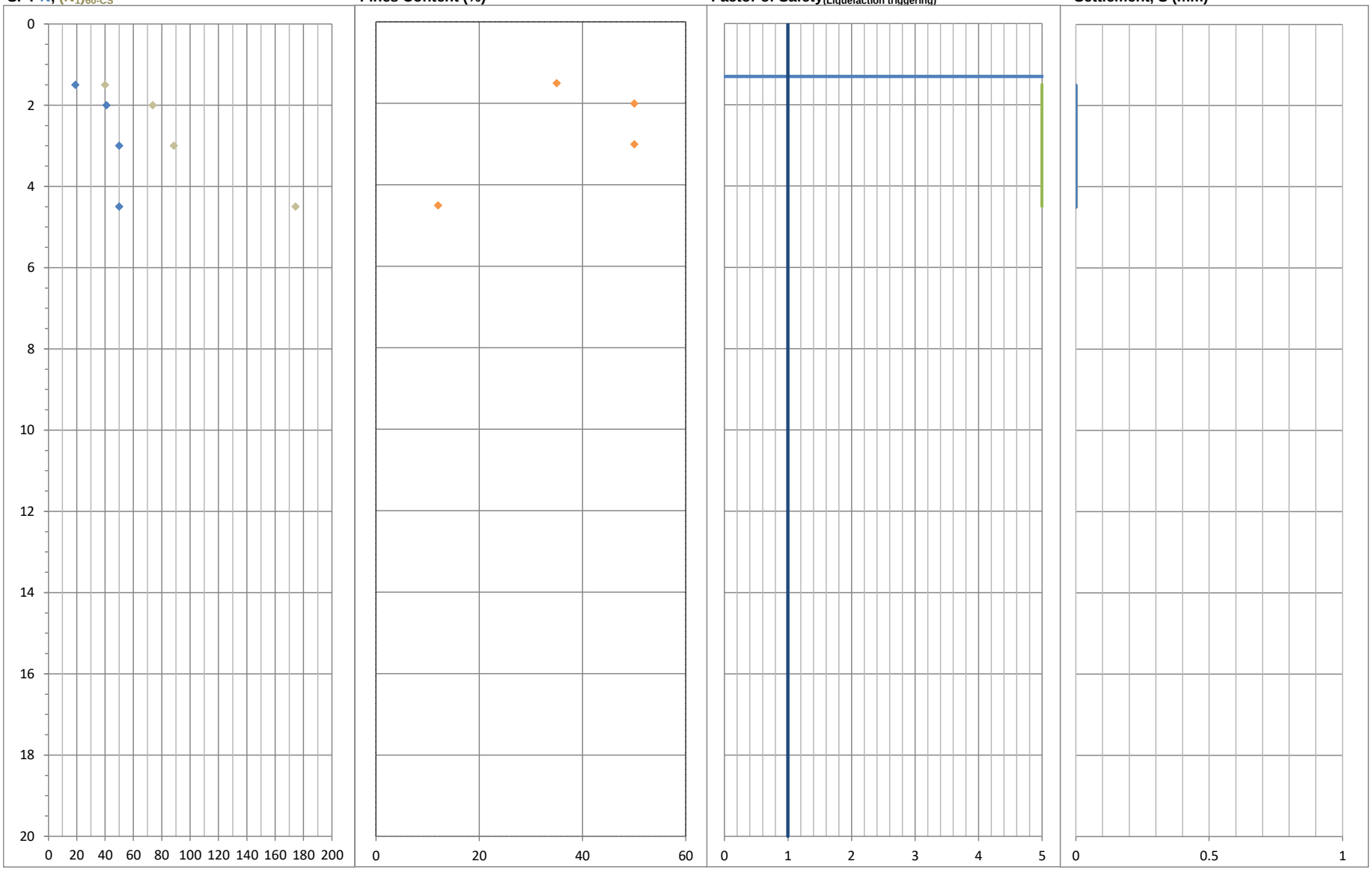
12 mth partition starts	5.00 Minutes mean starts	maximum measured		-- G.E.V. Distribution -- 1.96 std. dev.	ann. prob. (1/y)	return period (y)
				6144.930	0.001	1000
				4832.573	0.002	500
				3504.531	0.005	200
				2737.499	0.010	100
1-Jan-2020	05-Feb-2020 10:45:00	2398.649	A	2398.649	0.014	69.4
				2127.891	0.020	50
1-Jan-1978	14-Oct-1978 22:30:00	2108.490	B	2108.490	0.021	48.8
1-Jan-1987	11-Mar-1987 06:55:00	1888.837	C	1888.837	0.028	36.2
1-Jan-1999	18-Nov-1999 08:20:00	1783.354	D	1783.354	0.032	31.1
1-Jan-1980	17-Jan-1980 21:25:00	1592.563	E	1592.563	0.043	23.1
				1507.583	0.050	20
1-Jan-1968	14-Apr-1968 12:10:00	1295.091	F	1295.091	0.074	13.6
1-Jan-1972	10-Sep-1972 00:00:00	1184.101	G	1184.101	0.092	10.9
				1144.727	0.100	10
1-Jan-1977	04-May-1977 08:55:00	1071.191	H	1071.191	0.117	8.5
1-Jan-1983	11-Mar-1983 08:40:00	947.097	I	947.097	0.157	6.4
1-Jan-2013	03-Jun-2013 22:30:00	929.601	J	929.601	0.163	6.1
1-Jan-1994	21-Feb-1994 13:55:00	928.716	K	928.716	0.164	6.1
1-Jan-1997	13-Jan-1997 09:55:00	857.293	L	857.293	0.196	5.1
1-Jan-1993	23-Dec-1993 09:10:00	853.929	M	853.929	0.197	5.1
				848.931	0.200	5
1-Jan-2016	24-May-2016 10:40:00	801.624	N	801.624	0.226	4.4
1-Jan-1984	28-Jan-1984 15:40:00	768.337	O	768.337	0.247	4.0
1-Jan-2015	04-Jun-2015 21:35:00	729.856	P	729.856	0.275	3.6
1-Jan-2005	24-Dec-2005 04:25:00	712.854	Q	712.854	0.288	3.5
1-Jan-1967	27-Apr-1967 23:45:00	712.077	R	712.077	0.289	3.5
1-Jan-2018	21-Nov-2018 20:05:00	710.093	S	710.093	0.290	3.4
1-Jan-1969	27-Dec-1969 11:45:00	697.961	T	697.961	0.300	3.3
1-Jan-1995	03-Sep-1995 00:40:00	686.232	U	686.232	0.311	3.2
1-Jan-2017	24-Jan-2017 04:20:00	646.014	V	646.014	0.348	2.9
1-Jan-2012	24-Feb-2012 22:15:00	644.879	W	644.879	0.349	2.9
1-Jan-2011	19-Oct-2011 20:10:00	607.709	X	607.709	0.389	2.6
1-Jan-1982	27-Oct-1982 22:30:00	597.072	Y	597.072	0.401	2.5
				573.715	0.430	2.33
1-Jan-2004	23-Dec-2004 07:55:00	570.200	Z	570.200	0.434	2.3
1-Jan-1962	30-Jul-1962 05:50:00	556.248	a	556.248	0.452	2.2
1-Jan-1975	02-Aug-1975 20:10:00	546.906	b	546.906	0.465	2.2
1-Jan-1979	16-May-1979 23:45:00	521.179	c	521.179	0.501	2.0
1-Jan-2000	01-Feb-2000 09:50:00	502.154	d	502.154	0.530	1.9
1-Jan-1988	20-Jan-1988 18:40:00	494.683	e	494.683	0.541	1.8
1-Jan-1996	14-Oct-1996 03:55:00	490.876	f	490.876	0.547	1.8
1-Jan-1991	17-Aug-1991 17:00:00	476.780	g	476.780	0.570	1.8
1-Jan-1986	26-Nov-1986 06:15:00	476.507	h	476.507	0.570	1.8
1-Jan-2010	27-Apr-2010 17:35:00	475.448	i	475.448	0.572	1.7
1-Jan-1960	05-Aug-1960 23:25:00	472.037	j	472.037	0.578	1.7
1-Jan-1992	06-Jul-1992 08:25:00	457.277	k	457.277	0.602	1.7
1-Jan-1971	09-Jun-1971 05:20:00	456.484	l	456.484	0.604	1.7

1-Jan-1963	24-Jun-1963 07:50:00	433.969	m	433.969	0.643	1.6
1-Jan-1965	24-Jun-1965 16:50:00	426.446	n	426.446	0.656	1.5
1-Jan-1966	03-Jan-1966 09:30:00	426.408	o	426.408	0.656	1.5
1-Jan-2006	27-Apr-2006 09:50:00	420.824	p	420.824	0.666	1.5
1-Jan-2014	03-Aug-2014 22:00:00	418.735	q	418.735	0.670	1.5
1-Jan-1961	30-Nov-1961 10:05:00	398.168	r	398.168	0.708	1.4
1-Jan-2009	14-May-2009 00:10:00	390.372	s	390.372	0.722	1.4
1-Jan-1970	18-Sep-1970 00:45:00	384.279	t	384.279	0.733	1.4
1-Jan-1989	22-Jun-1989 20:00:00	376.185	u	376.185	0.748	1.3
1-Jan-2001	26-Aug-2001 23:40:00	372.438	v	372.438	0.755	1.3
1-Jan-1976	15-Dec-1976 19:30:00	370.450	w	370.450	0.759	1.3
1-Jan-2007	17-Oct-2007 21:55:00	364.051	x	364.051	0.770	1.3
1-Jan-2002	29-May-2002 12:15:00	352.123	y	352.123	0.792	1.3
1-Jan-1981	01-Aug-1981 07:40:00	341.982	z	341.982	0.810	1.2
1-Jan-2019	21-Dec-2019 12:30:00	324.569	A	324.569	0.841	1.2
1-Jan-2003	17-Jul-2003 09:20:00	307.376	B	307.376	0.869	1.2
1-Jan-1998	07-Aug-1998 09:20:00	277.981	C	277.981	0.913	1.1
1-Jan-1964	16-May-1964 09:50:00	277.438	D	277.438	0.914	1.1
1-Jan-1990	14-Oct-1990 18:40:00	262.557	E	262.557	0.933	1.1
1-Jan-2008	25-Sep-2008 00:25:00	261.487	F	261.487	0.934	1.1
1-Jan-1985	02-Dec-1985 21:55:00	224.213	G	224.213	0.971	1.0
1-Jan-1974	01-Aug-1974 01:00:00	217.486	H	217.486	0.976	1.0
1-Jan-1973	08-May-1973 00:55:00	206.250	I	206.250	0.982	1.0
	Mean =	663.708				

APPENDIX C

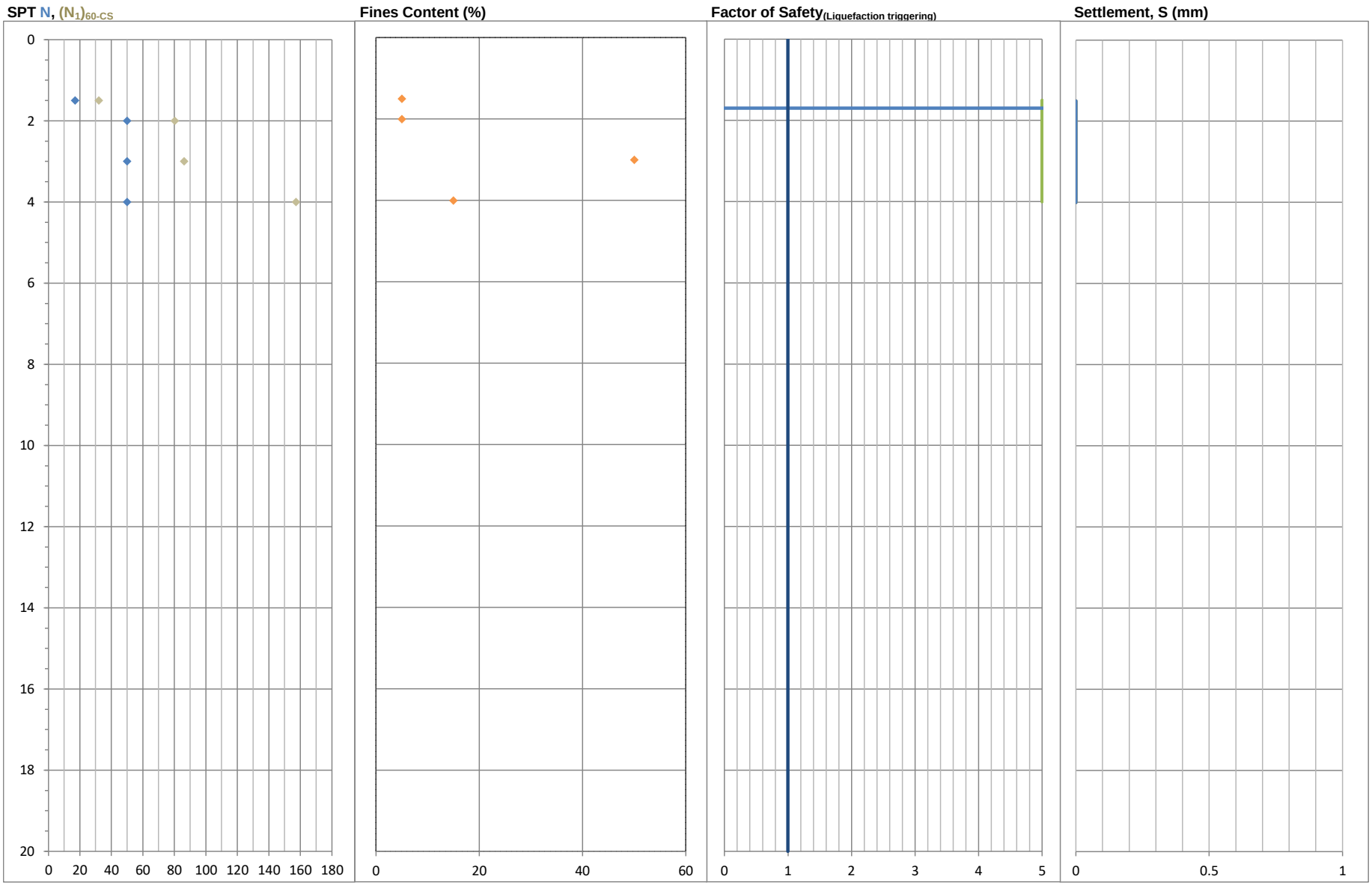
Geotechnical Analysis

RILEY CONSULTANTS	Test:	***BH1***	File:	190376	***SLS1***	Mw	6	PGA	0.06	g	S	0	mm	S_{Index}	0	mm	LSN	0
	Project:	Longford Shared Path																
	Water table depth:		1.3	m	***ULS***	Mw	6.1	PGA	0.26	g	S	0	mm	S_{Index}	0	mm	LSN	0



RILEY CONSULTANTS

Test:	***BH2***	File:	190376	***SLS1***	Mw	6	PGA	0.06	g	S	0	mm	S_{Index}	0	mm	LSN	0
Project:	Longford Shared Path														
Water table depth:		1.7	m	***ULS***	Mw	6.1	PGA	0.26	g	S	0	mm	S_{Index}	0	mm	LSN	0



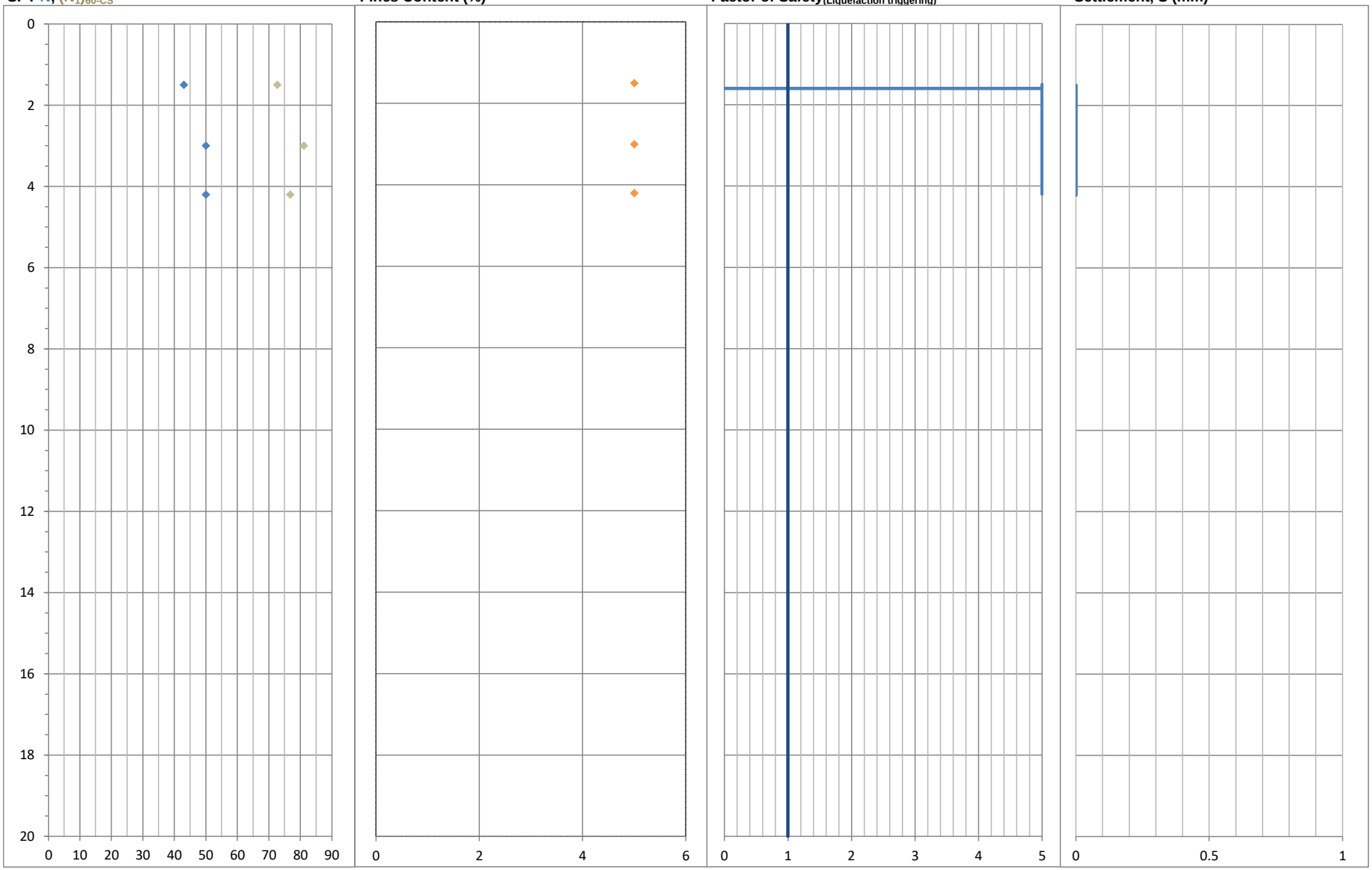
RILEY CONSULTANTS

Test: ***BH3*** File: 190376

Project: Longford Shared Path

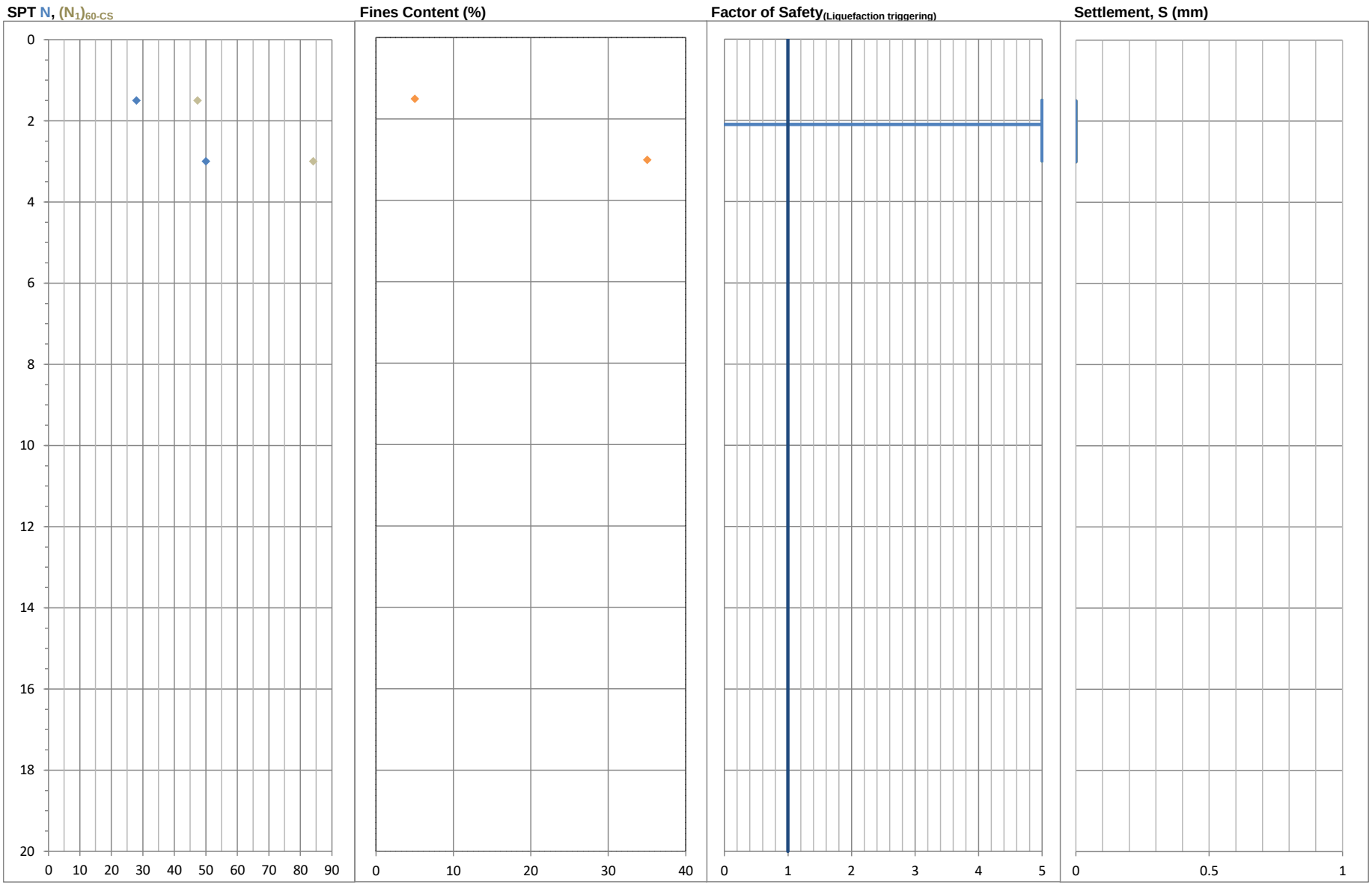
Water table depth: 1.6 m

SLS1	Mw	6	PGA	0.06	g	S	0	mm	S_{Index}	0	mm	LSN 0
ULS	Mw	6.1	PGA	0.26	g	S	0	mm	S_{Index}	0	mm	LSN 0



RILEY CONSULTANTS

Test:	***BH4***	File:	190376	***SLS1***	Mw	6	PGA	0.06	g	S	0	mm	S_{Index}	0	mm	LSN	0
Project:	Longford Shared Path														
Water table depth:		2.1	m	***ULS***	Mw	6.1	PGA	0.26	g	S	0	mm	S_{Index}	0	mm	LSN	0



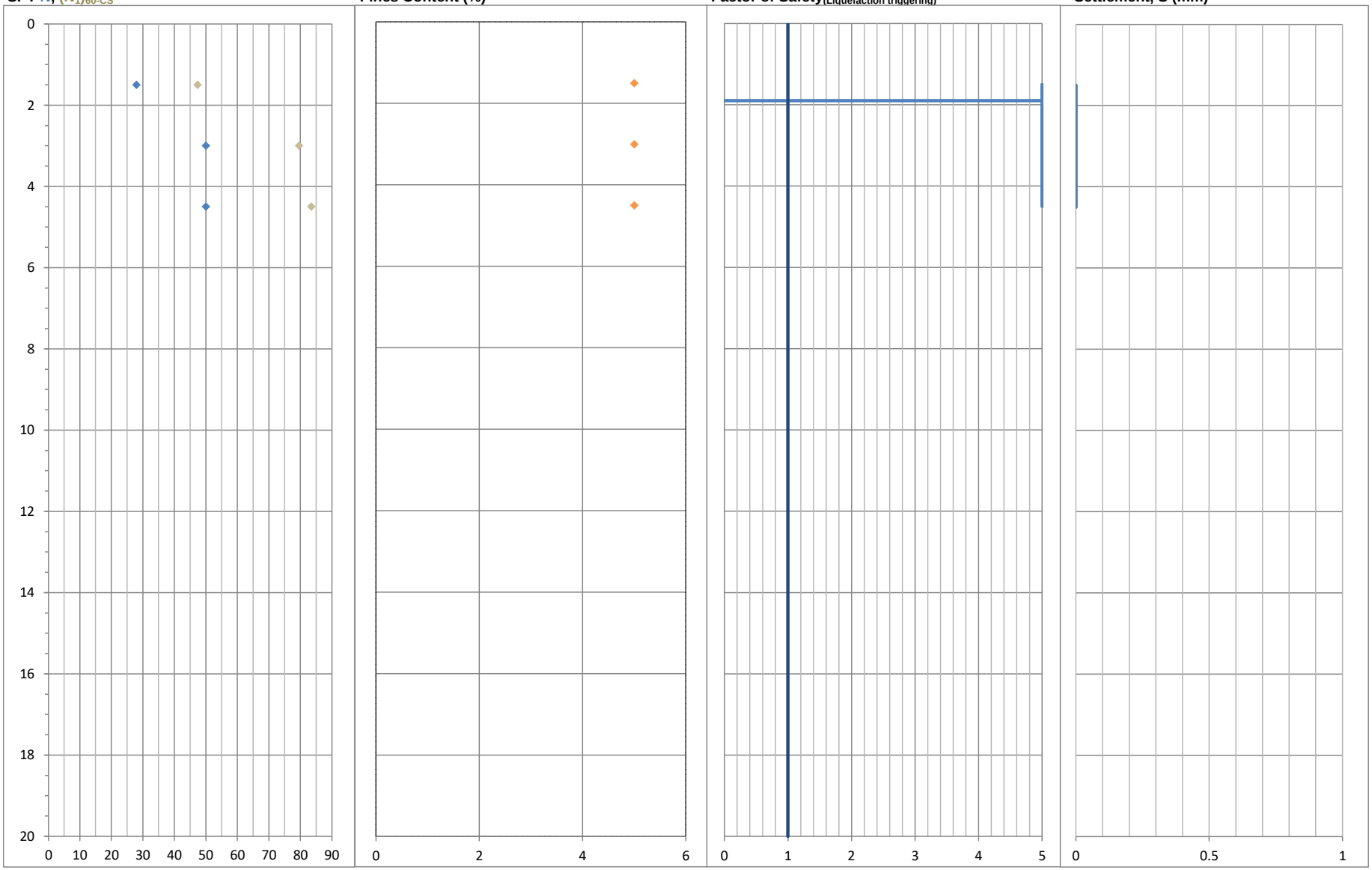
RILEY CONSULTANTS

Test: ***BH5*** File: 190376

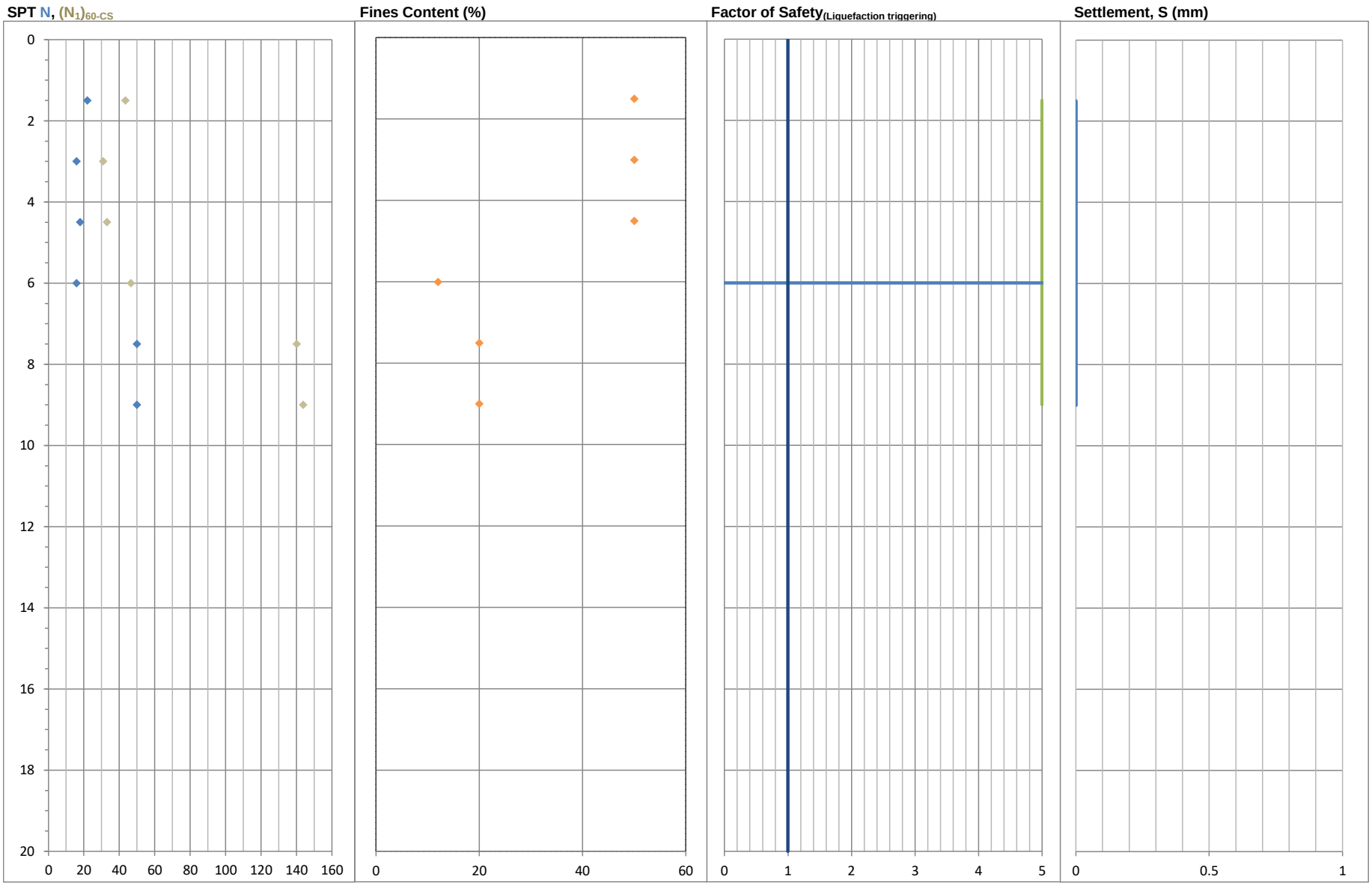
Project: Longford Shared Path

Water table depth: 1.9 m

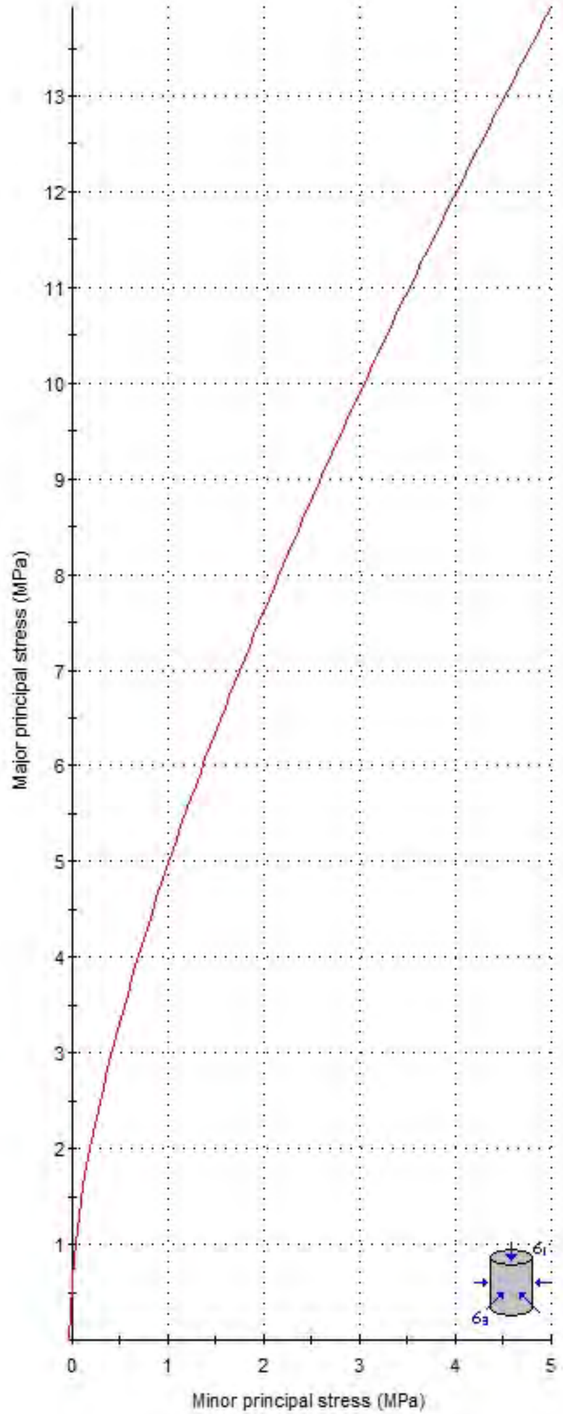
SLS1	Mw	6	PGA	0.06	g	S	0	mm	S_{Index}	0	mm	LSN	0
ULS	Mw	6.1	PGA	0.26	g	S	0	mm	S_{Index}	0	mm	LSN	0



Test:	***BH6***	File:	190376	***SLS1***	Mw	6	PGA	0.06	g	S	0	mm	S_{Index}	0	mm	LSN	0
Project:	Longford Shared Path														
Water table depth:	6	m		***ULS***	Mw	6.1	PGA	0.26	g	S	0	mm	S_{Index}	0	mm	LSN	0



Analysis of Rock Strength using RocLab



Hoek-Brown Classification

intact uniaxial comp. strength (sigci) = 20 MPa
GSI = 40 mi = 7 Disturbance factor (D) = 0
intact modulus (Ei) = 7500 MPa
modulus ratio (MR) = 375

Hoek-Brown Criterion

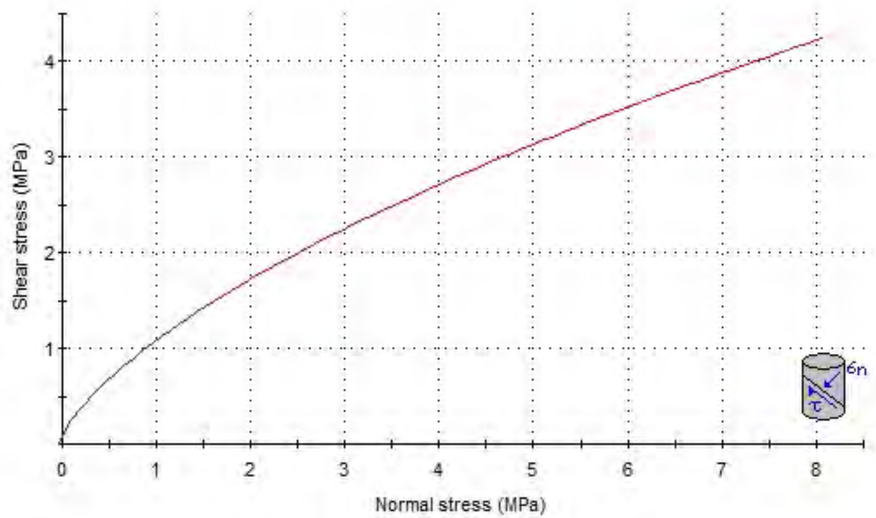
mb = 0.821 s = 0.0013 a = 0.511

Mohr-Coulomb Fit

cohesion = 0.751 MPa friction angle = 24.69 deg

Rock Mass Parameters

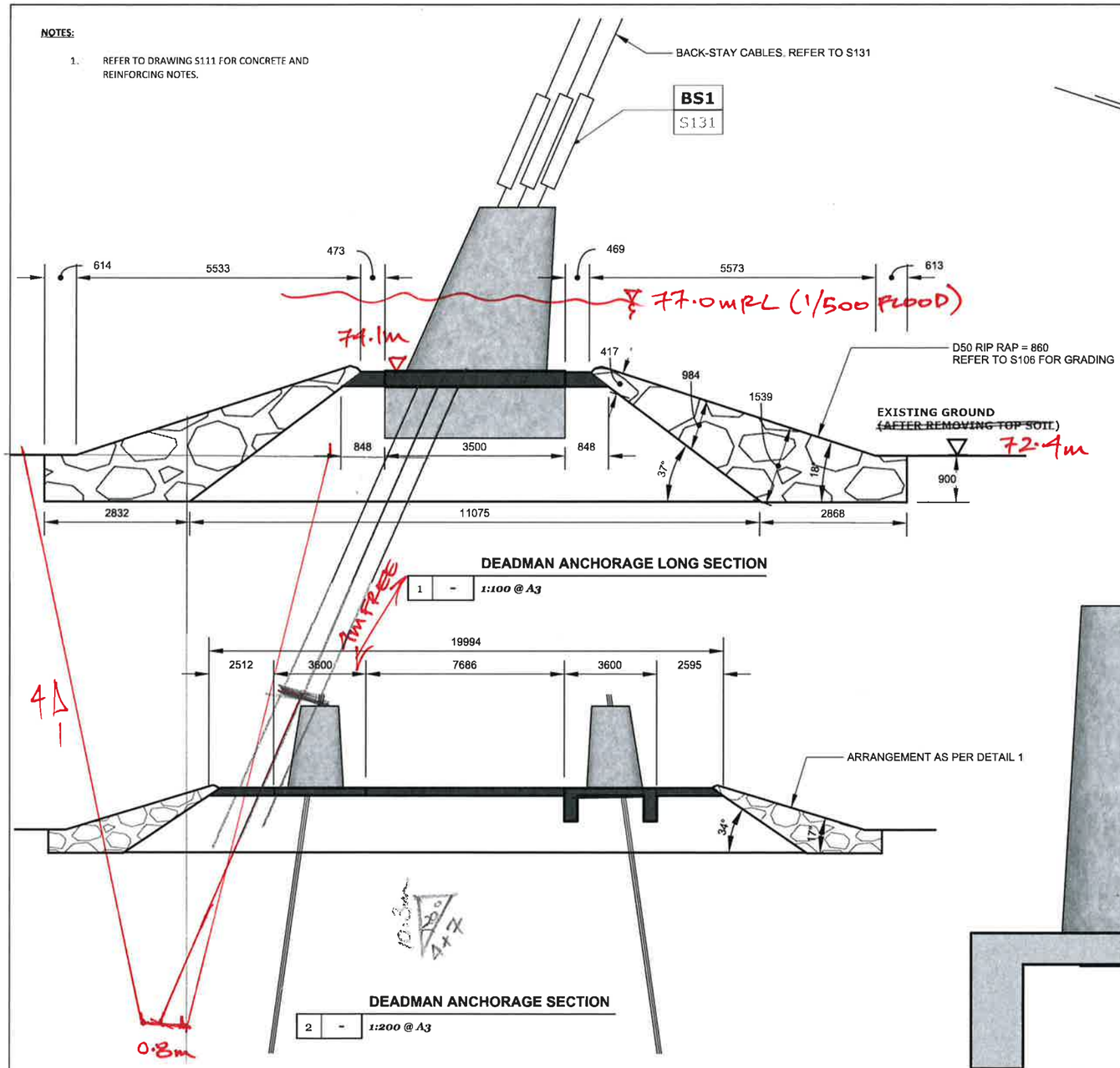
tensile strength = -0.031 MPa
uniaxial compressive strength = 0.661 MPa
global strength = 2.343 MPa
deformation modulus = 1197.39 MPa



PLOT DATE: May 13, 2020

NOTES:

1. REFER TO DRAWING S111 FOR CONCRETE AND REINFORCING NOTES.

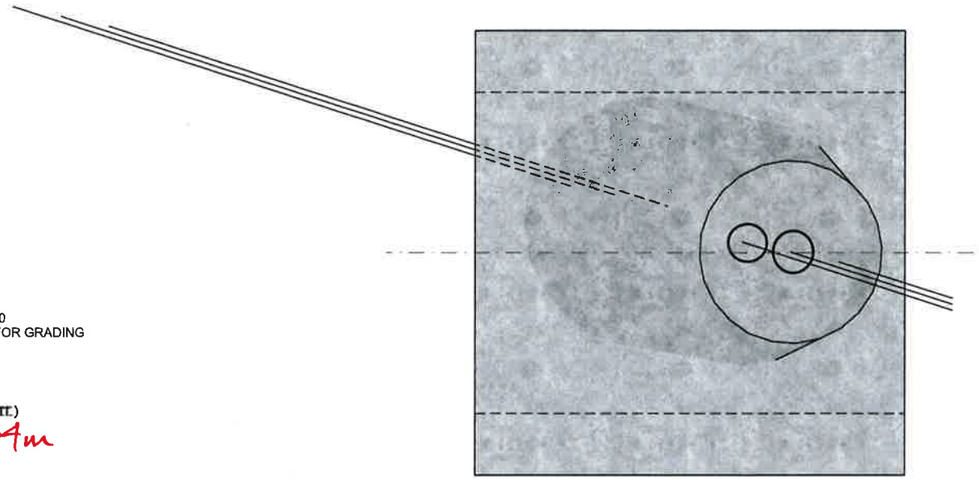


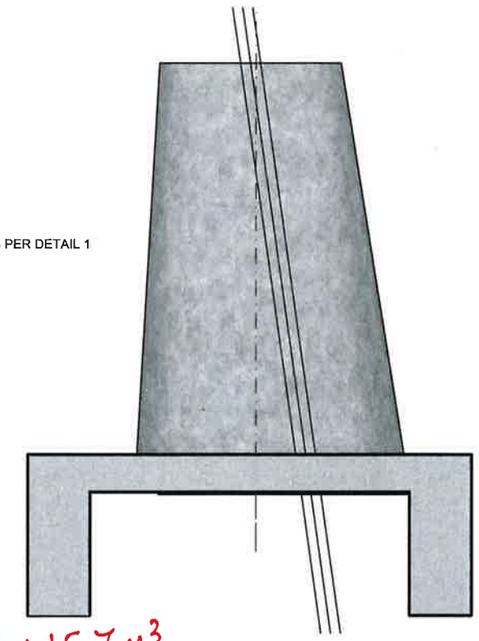
DEADMAN ANCHORAGE LONG SECTION

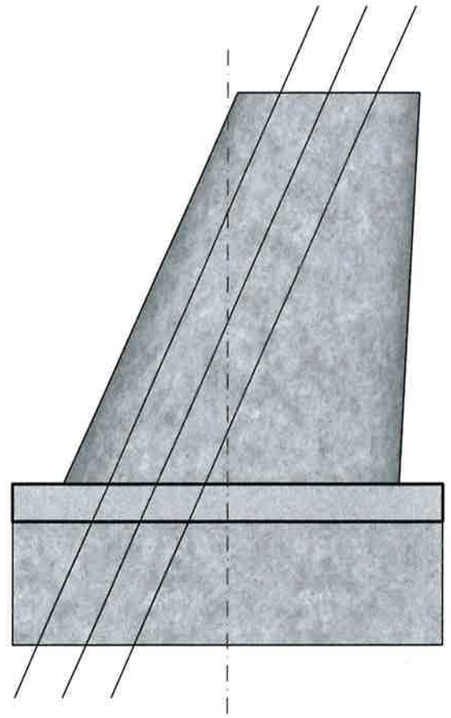
1 | - | *1:100 @ A3*

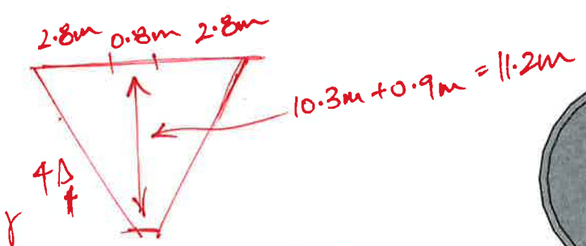
DEADMAN ANCHORAGE SECTION

2 | - | *1:200 @ A3*









$Q_u = V\gamma$

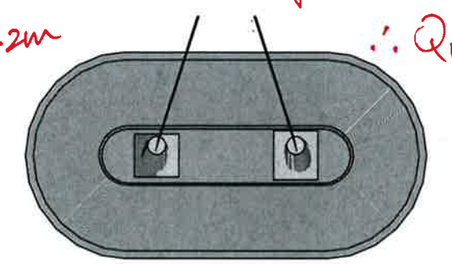
$V = \frac{1}{3} \times (A_{base} + A_{Top} + \sqrt{A_{base} \times A_{Top}}) \times D$

$A_{base} = B \times L$

$= 0.8m \times 0.15m = 0.12m^2$

$A_{TOP} = (B + \frac{D}{2})(L + D/2) = 6.4m \times 5.75$

$= 36.8m^2$



$V = 145.7m^3$

$\therefore Q_u = 145.7m^3 \times 20 kN/m^3$

$= 2,913.6kN$

$= 971.2 kN/Anchor$

$\gg 560 kN$ max Load

$\therefore$ OK

DO NOT SCALE. IF IN DOUBT ASK

DRAFT

13/05/2020 09:46:27

REV	DESCRIPTION	DATE	DESIGN	DRAWN	APPROVED
A	PRELIM DESIGN	NOV 19	DC	PS	DC
B	(NOT USED)				
C	FOR PEER REVIEW	MAY 20	DC	PS	DC

RILEY CONSULTANTS

DC STRUCTURES STUDIO

GO DISTRICT COUNCIL RE

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PROJECT
LONGFORD SHARED PATH
PROJECT NO.
2019/11

LONGFORD SHARED PATH
MATAURA CYCLEBRIDGE
BACKSTAY ANCHORAGE

DWG: GDC_2019/11_S115

S115
Rev C

Rock Socket Pile Design per Sinclair & Every

Side shear resistance		Case1	
Atmospheric pressure	Pa	100	kPa
Side roughness factor	b	1	From table
Rock quality designation (shaft)	RQD	0	(0 to 100)
UCS of intact rock (shaft)	qu	20	Mpa
Youngs modulus of intact rock (shaft)	Ei	7500	Mpa
Rock quality factor	beta	0.35	
Rock mass factor	j	0.010415669	
Side friction	fs	494.9747468	kPa
Youngs modulus of rock mass (shaft)	Er	78.11751379	Mpa
End bearing resistance			
Rock quality designation (base)	RQD	0	(0 to 100)
UCS of intact rock (base)	qu	20	Mpa
Youngs modulus of intact rock (base)	Ei	7500	
Rock quality factor	beta	0.35	
Rock mass factor	j	0.010415669	
Ultimate end bearing capacity	qmax	10889.44443	kPa
Youngs modulus of rock mass (base)	Eb	78.11751379	Mpa

Design philosophy

For SLS, use elasticity to distribute load between shaft and end resistance. Check no side slip, end be
For ULS, use ultimate side and end bearing incorporating strength reduction factors.

SLS load	SLS (Pt)	200	kN
ULS load - max per pile - Tower	ULS	1225	kN
Pile diameter	D	0.6	m
Pile circumference		1.884955592	m
Pile base area		0.282743339	m2
Pile material UCS	f'c	30	Mpa
Pile material Youngs Modulus	Ep=Ec	25084.38891	MPa
SLS elastic solution			
Socket length for SLS with zero bearing contrib	Lmax	0.214361028	m
L/D ratio for Figure 7	Lmax/D	0.357268379	
Eb/Er ratio for Figure 7	Eb/Er	1	
Ep/Er=Ec/Er ratio for Figure 7 and 8	Ep/Er=Ec/Er	321.110948	
L/D ratio from Figure 7	L/D	0.833333333	
Pb/Pt % from Figure 7	Pb/Pt	35	%
Total pile load	Pt	200	kN
Load on pile base	Pb	70	kN
Bearing pressure on pile base		247.5743559	kPa
Ratio of base bearing to UCS	base bearing /	1%	(check less
I delta from Figure 8	Idelda	0.31	
Settlement	sigma v	1.322793421	mm
Required pile socket length for SLS	Lsls	0.5	m
ULS factored ultimate capacity solution			
Geotech capacity reduction factor AS2159	Ø̸g	0.5	
Factored side resistance	fs factored	247.4873734	kPa
Penetration depth reduction factor		0.761666667	
Factored base resistance	qu factored	4147.063421	kPa
ULS load minus factored base resistance		52.44544217	kN
Socket length for ULS	Luls	0.112422589	m

Check SLS settlement for selected socket length

Selected rock socket length	L	0.5	m (must be
L/D ratio for Figure 7 and 8	L/D	0.833333333	
Pb/Pt % from Figure 7	Pb/Pt	40	%
Load on pile base	Pb	80	kN
Bearing pressure on pile base		282.9421211	kPa
Ratio of base bearing to UCS	base bearing /	1%	(check less
I delta from Figure 8	Idelda	0.5	
Settlement	sigma v	2.133537776	mm

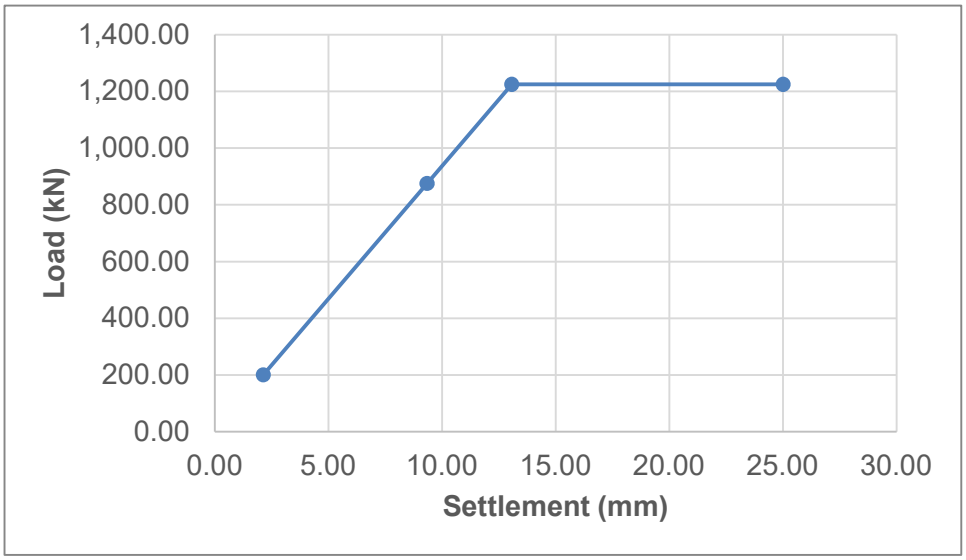
Min embedment	0.6 m
Neglect upper 1D	0.6 m
Min design embedment	1.1 m

7m long piles at Tower, 6m long at East Approach

		kN	(check in B1/VM4 with dense
Ultimate end bearing - factored	**1,539.46**		gravel properties is 1,225kN)
Ultimate Skin Friction - factored	**279.90**		
Factored Ultimate capacity	**1,819.36**	kN	per pile
Average Load per Tower Pile	1,099.00	kN	therefore OK
Average Load per east abutment Pile	295.00	kN	therefore OK
Check end bearing B1/VM4 in dense gravel	1,225.00	kN	similar to above
Uplift	0.60	m	
Ultimate capacity	1819.36	kN	per pile
	(2/3 skin + pile weight)*reduction factor		
Tensile capacity of piles	419.71	kN	
Factored tensile capacity	209.86	kN	

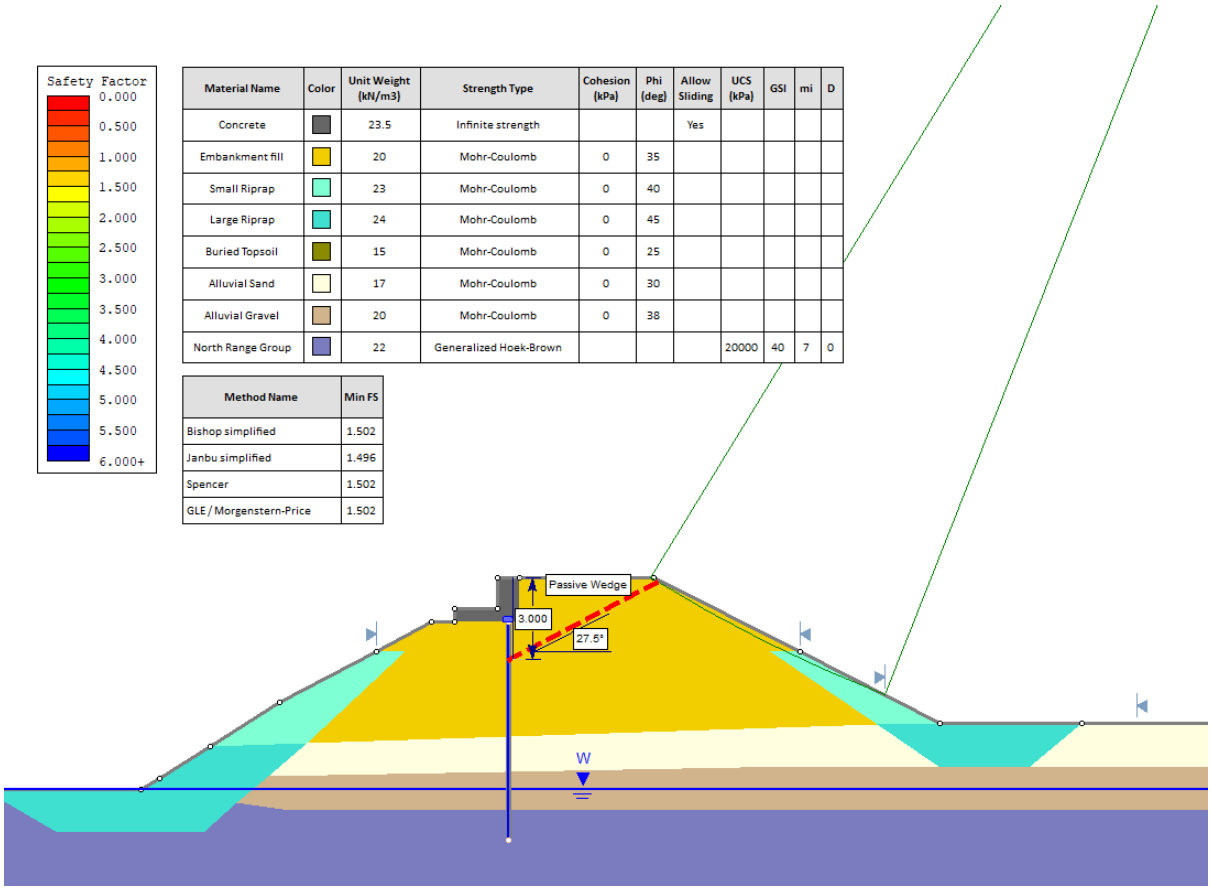
Load/ Settlement Relationship

Load	Settlement	
200.00	2.13	93,741.02
875.00	9.33	93,741.02
1,225.00	13.07	93,741.02
1,225.00	25.00	



Slope Stability Outputs – Selected Model Screen Shots

Figure A. West Embankment - Static Factor of Safety Showing Passive Wedge



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	UCS (kPa)	GSI	mi	D
Concrete		23.5	Infinite strength			Yes				
Embankment fill		20	Mohr-Coulomb	0	35					
Small Riprap		23	Mohr-Coulomb	0	40					
Large Riprap		24	Mohr-Coulomb	0	45					
Buried Topsoil		15	Mohr-Coulomb	0	25					
Alluvial Sand		17	Mohr-Coulomb	0	30					
Alluvial Gravel		20	Mohr-Coulomb	0	38					
North Range Group		22	Generalized Hoek-Brown				20000	40	7	0

Method Name	Min FS
Bishop simplified	1.502
Janbu simplified	1.496
Spencer	1.502
GLE / Morgenstern-Price	1.502

Figure B. West Embankment – Factor of Safety Under Rapid Drawdown Conditions

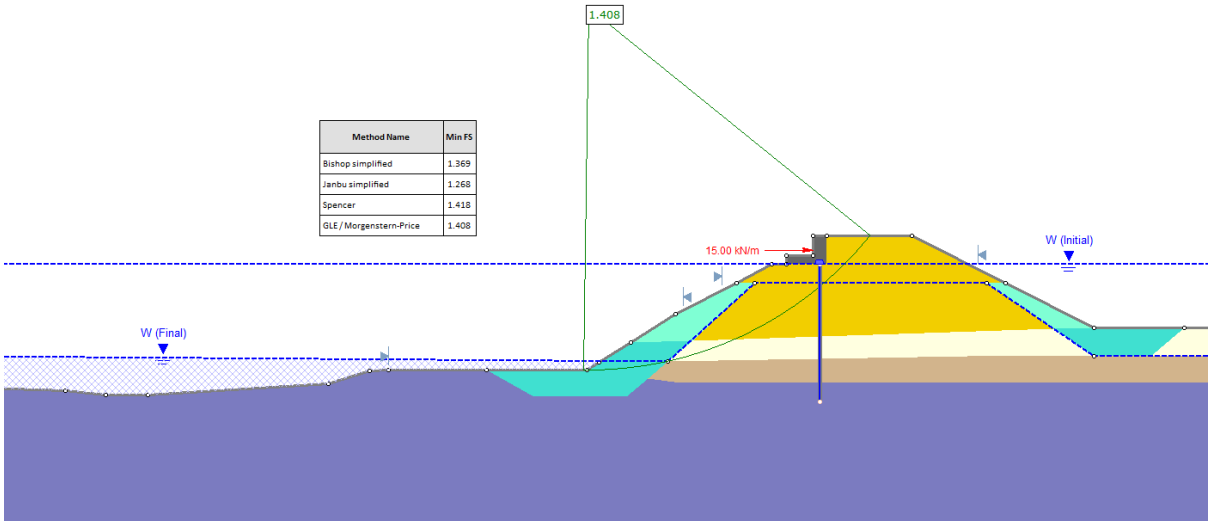


Figure C. West Embankment – ULS (0.26g) Factor of Safety Less than 1.0

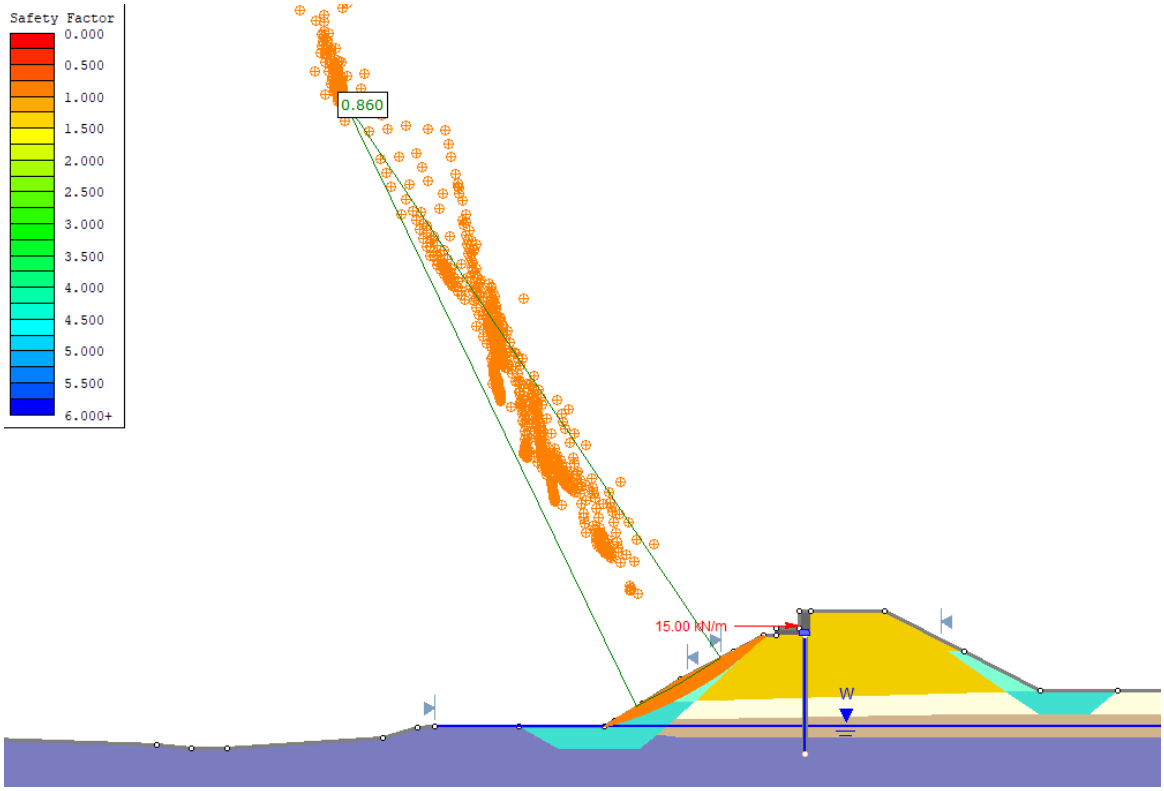


Figure D. West Embankment – Fully Liquefied Strength Parameters

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	UCS (kPa)	GSI	mi	D	Vertical Strength Ratio	Minimum Shear Strength (kPa)
Concrete		23.5	Infinite strength			Yes						
Embankment fill		20	Mohr-Coulomb	0	35							
Small Riprap		23	Mohr-Coulomb	0	40							
Large Riprap		24	Mohr-Coulomb	0	45							
Buried Topsoil		15	Mohr-Coulomb	0	25							
Alluvial Sand		17	Vertical Stress Ratio								0.1	5
Alluvial Gravel		20	Mohr-Coulomb	0	38							
North Range Group		22	Generalized Hoek-Brown				20000	40	7	0		

Method Name	Min FS
Bishop simplified	1.095
Janbu simplified	1.010
Spencer	1.183
GLE / Morgenstern-Price	1.112

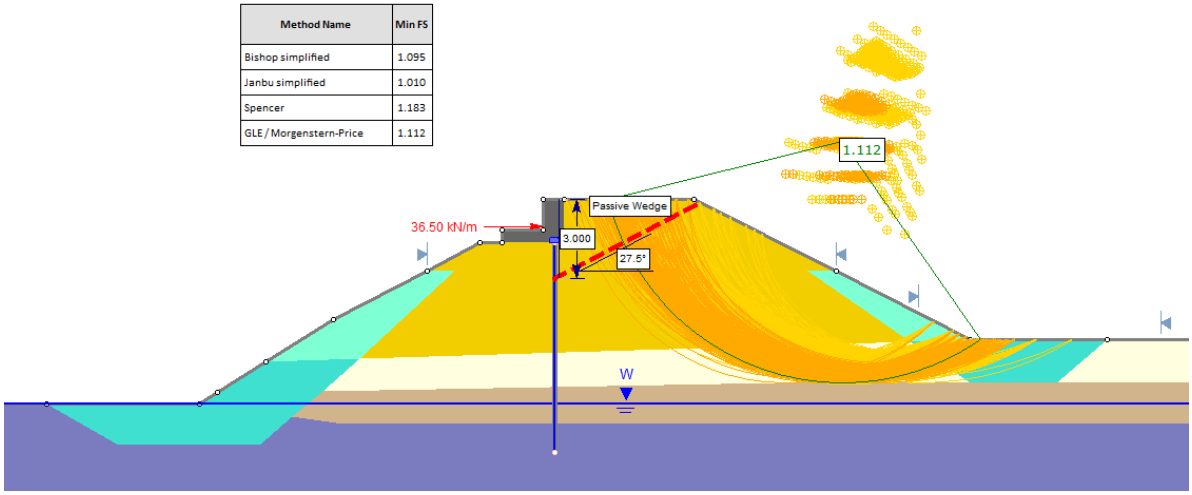


Figure E. East Abutment – Current Factor of Safety

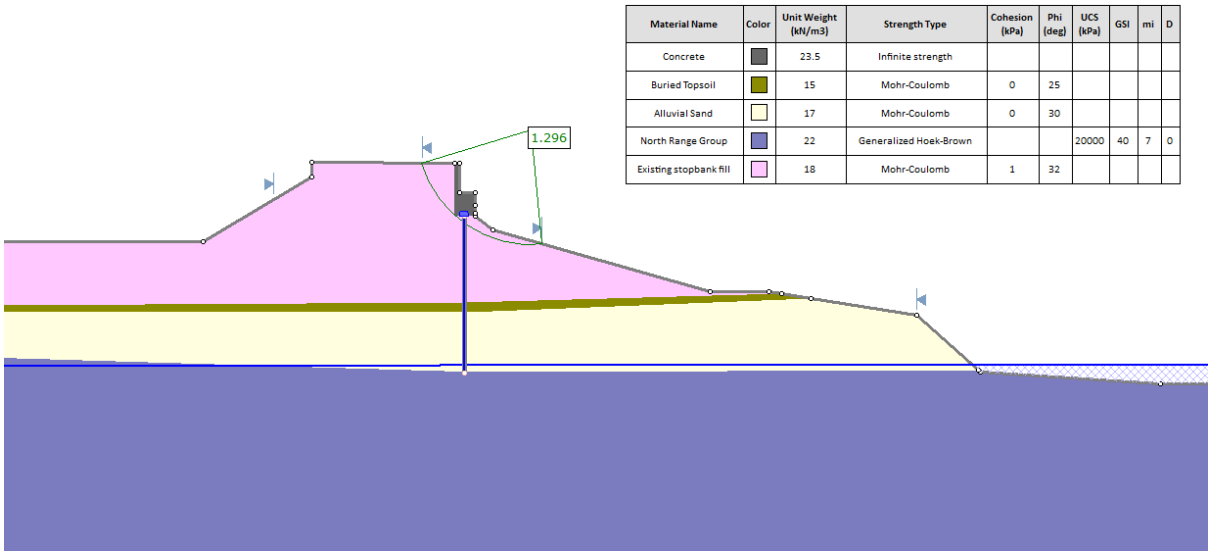


Figure F. East Abutment – Static Factor of Safety with Bridge Load

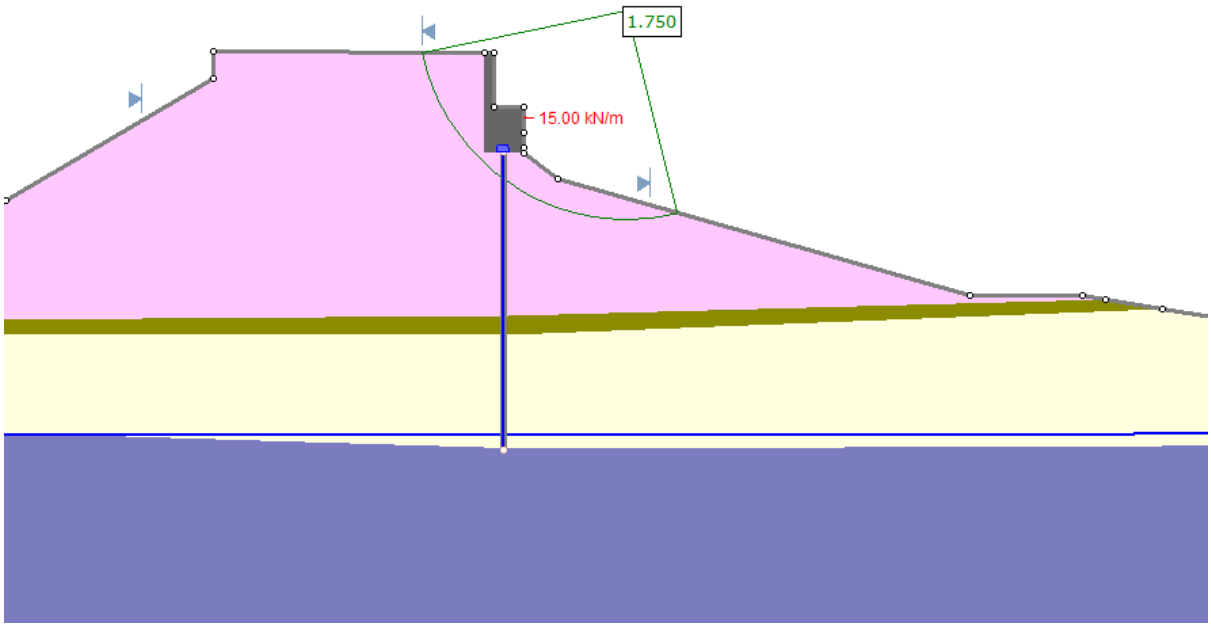


Figure G. East Abutment – Rapid-Drawdown Conditions

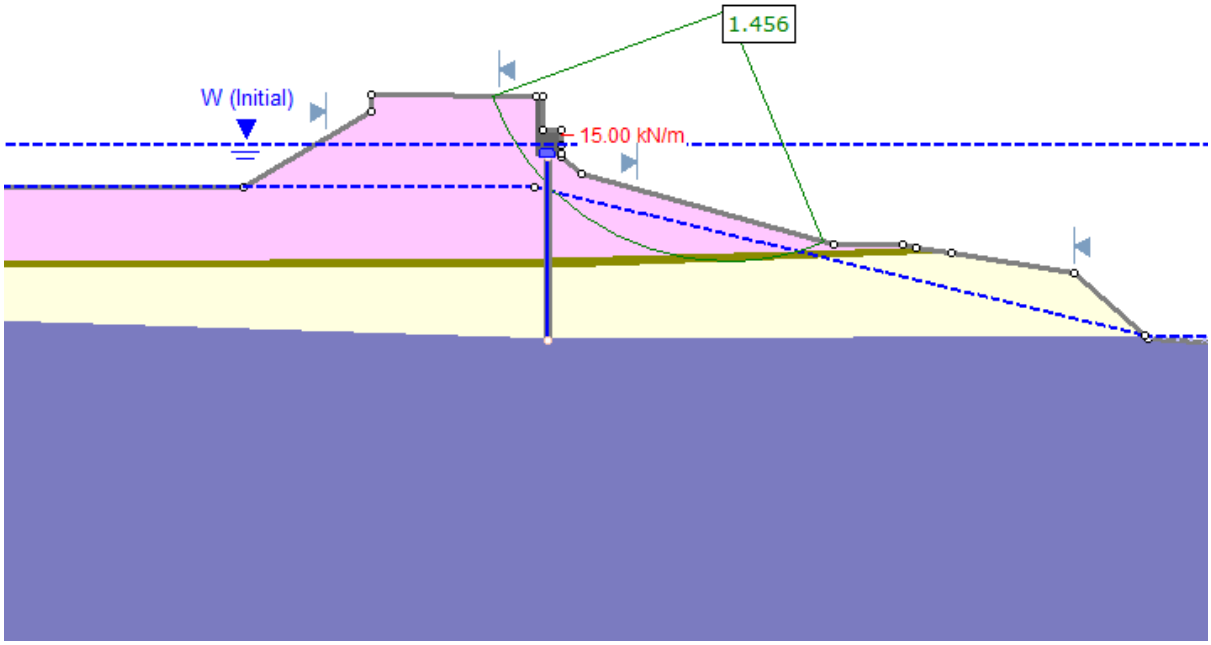


Figure H. East Abutment - SLS Factor of Safety

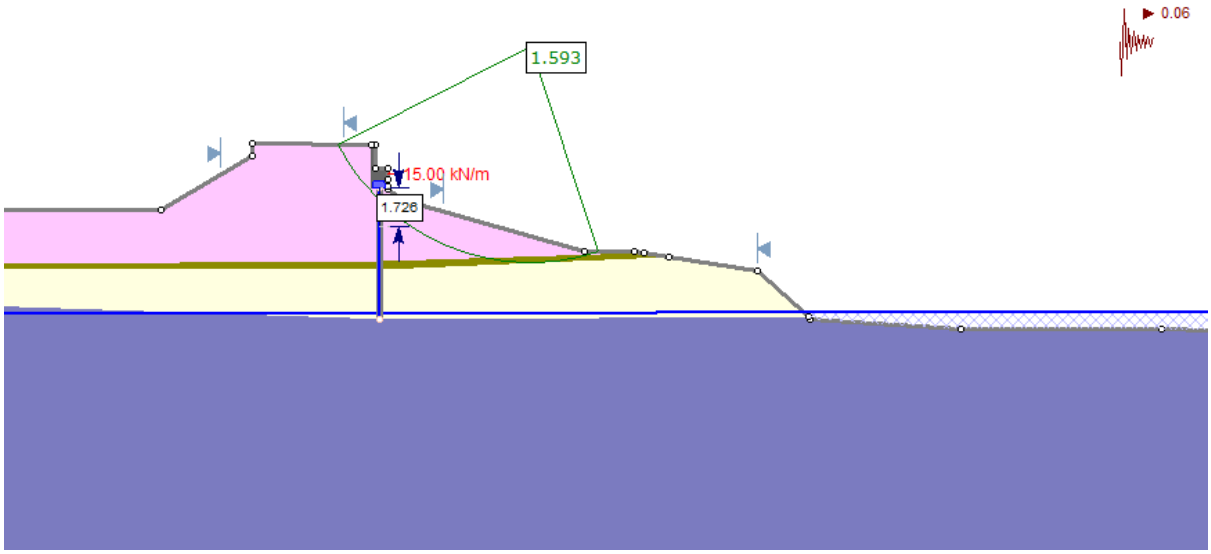
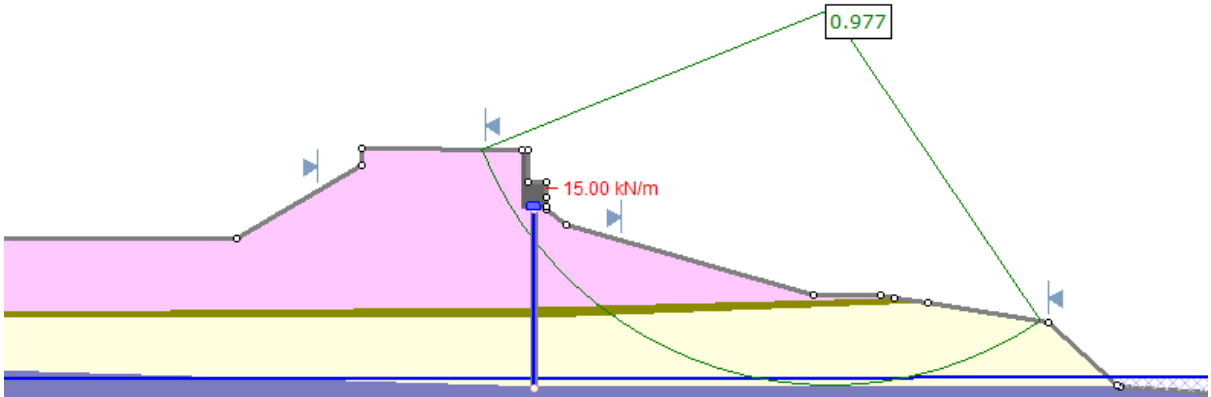


Figure I. East Abutment – Liquefied Strength



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	UCS (kPa)	GSI	mi	D	Vertical Strength Ratio	Minimum Shear Strength (kPa)
Concrete		23.5	Infinite strength			Yes						
Embankment fill		20	Mohr-Coulomb	0	35							
Small Riprap		23	Mohr-Coulomb	0	40							
Large Riprap		24	Mohr-Coulomb	0	45							
Buried Topsoil		15	Mohr-Coulomb	0	25							
Alluvial Sand		17	Vertical Stress Ratio								0.1	5
Alluvial Gravel		20	Mohr-Coulomb	0	38							
North Range Group		22	Generalized Hoek-Brown				20000	40	7	0		
Existing stopbank fill		18	Mohr-Coulomb	1	32							

Springs

Structure	Eastern Abutment Piles
Date	19/02/20

Pile Diam	0.6
Groundwater	5.7

INPUTS

Model

Initial parameters

Node n°	Level (m negative)	Soil description	Total stress	PWP	Effective Stress	Esoil (kPa)	γ kN.m^{-3}	ϕ' (^{o})	K_q^0	K_c^0	K^0	Nc	d_c^{inf}	K_c^{inf}	K_q^{inf}	α_q	α_c
1	1	FILL	17	0	21.6	9000	17	32	5.53806523	7.662064	0.47008	35.49026071	2.203563	78.20502	22.971867	0.09048	0.18999
2	2	FILL	34	0	47.1	9000	17	32	5.53806523	7.662064	0.47008	35.49026071	2.203563	78.20502	22.971867	0.09048	0.18999
3	3	SILT	50	0	50.0	4100	16	28	4.09431425	6.380809	0.53053	25.80334297	1.906904	49.20451	13.879951	0.12158	0.25544
4	4	SAND	68	0	68.0	14000	18	30	4.75305577	6.973587	0.5	30.13962779	2.034444	61.3174	17.700808	0.10597	0.22226
5	5	CW-HW Rock	90	9.81	80.2	78000	22	50	32.382122	26.41816	0.23396	266.8817627	9.830289	2623.525	731.48435	0.00883	0.01912
6	6	CW-HW Rock	112	19.62	92.4	78000	22	50	32.382122	26.41816	0.23396	266.8817627	9.830289	2623.525	731.48435	0.00883	0.01912
7	7	SW-MW Rock	134	29.43	104.6	78000	22	50	32.382122	26.41816	0.23396	266.8817627	9.830289	2623.525	731.48435	0.00883	0.01912
	8																

Model			Initial Parameters		K (kN/m^3)	Limiting Force (kN)	Terzaghi (1955)	Cubrinovski et al (2009b)					Average			Limiting Force (kN)
Node n°	Level (m negative)	Soil description	Kq	Kc				SPT N	Spacing [m]	Kc	Su	Pc-max (kN)	K (kN/m^3)	K/3 (kN/m^3)	K*3 (kN/m^3)	
1	1	FILL	**6.76034706**	**17.30439**	**7200**	**88**	**6667**	24	**1**	**19800**			**11200**	**3733**	**33600**	**88**
2	2	FILL	**8.75397641**	**30.37885**	**7200**	**248**	**6667**	24	**1**	**19800**			**11200**	**3733**	**33600**	**248**
3	3	SILT	**7.38461111**	**28.4599**	**3300**	**222**	**3037**	7	**1**	**5800**	**25**	**135**	**4000**	**1333**	**12000**	**135**
4	4	SAND	**9.69930698**	**37.65398**	**11200**	**396**	**10370**	23	**1**	**19000**			**13500**	**4500**	**40500**	**396**
5	5	CW-HW Rock	**75.8237868**	**352.0965**	**62400**	**3648**	**57778**	50	**1**	**41200**			**53700**	**17900**	**161100**	**3648**
6	6	CW-HW Rock	**84.7542976**	**413.6778**	**62400**	**4698**	**57778**	50	**1**	**41200**			**53700**	**17900**	**161100**	**4698**
7	7	SW-MW Rock	**93.4447992**	**472.0081**	**62400**	**5863**	**57778**	50	**1**	**41200**			**53700**	**17900**	**161100**	**5863**
	8															

Springs

Structure	**Tower Piles**
Date	**19/02/20**

Pile Diam	**0.6**
Groundwater	**5.7**

INPUTS

Model						Initial parameters											
Node n°	Level (m negative)	Soil description	Total stress	PWP	Effective Stress	Esoil (kPa)	γ kN.m^{-3}	ϕ' (°)	K_q^0	K_c^0	K^0	Nc	d_c^{inf}	K_c^{inf}	K_q^{inf}	α_q	α_c
1	1	FILL	20	0	21.6	25000	20	35	7.03201354	8.925567	0.42642	46.12359869	2.563174	118.2228	35.299558	0.0686	0.14487
2	2	FILL	40	0	47.1	25000	20	35	7.03201354	8.925567	0.42642	46.12359869	2.563174	118.2228	35.299558	0.0686	0.14487
3	3	FILL	60	0	60.0	25000	20	35	7.03201354	8.925567	0.42642	46.12359869	2.563174	118.2228	35.299558	0.0686	0.14487
4	4	FILL	80	0	80.0	25000	20	35	7.03201354	8.925567	0.42642	46.12359869	2.563174	118.2228	35.299558	0.0686	0.14487
5	5	Sandy SILT	96	0	96.0	4100	16	28	4.09431425	6.380809	0.53053	25.80334297	1.906904	49.20451	13.879951	0.12158	0.25544
6	6	Sandy GRAVEL	116	2.943	113.1	38000	20	38	9.06576827	10.56579	0.38434	61.3517657	3.103922	190.4311	57.182294	0.0496	0.1056
7	7	ROCK	138	12.753	125.2	78000	22	50	32.382122	26.41816	0.23396	266.8817627	9.830289	2623.525	731.48435	0.00883	0.01912
8	8	ROCK	160	22.563	137.4	78000	22	50	32.382122	26.41816	0.23396	266.8817627	9.830289	2623.525	731.48435	0.00883	0.01912
9	9	ROCK	182	32.373	149.6	78000	22	50	32.382122	26.41816	0.23396	266.8817627	9.830289	2623.525	731.48435	0.00883	0.01912
10	10	ROCK	204	42.183	161.8	78000	22	50	32.382122	26.41816	0.23396	266.8817627	9.830289	2623.525	731.48435	0.00883	0.01912
	11																

Model			Initial Parameters		K (kN/m^3)	Limiting Force (kN)	Terzaghi (1955)	Cubrinovski et al (2009b)					Average			Limiting Force (kN)
Node n°	Level (m negative)	Soil description	Kq	Kc				SPT N	Spacing [m]	Kc	Su	Pc-max (kN)	K (kN/m^3)	K/3 (kN/m^3)	K*3 (kN/m^3)	
1	1	FILL	8.56049484	20.69929	20000	111	18519	20	1	16500			18300	6100	54900	110
2	2	FILL	11.1699634	37.98586	20000	316	18519	20	1	16500			18300	6100	54900	315
3	3	FILL	13.3154001	50.06687	20000	479	18519	20	1	16500			18300	6100	54900	500
4	4	FILL	15.1104739	58.98596	20000	725	18519	20	1	16500			18300	6100	54900	725
5	5	Sandy SILT	8.7614391	34.51769	3300	505	3037	7	1	5800	25	135	4000	1300	12000	135
6	6	Sandy GRAVEL	24.1055195	99.03415	30400	1635	28148	34	1	28000			28800	9600	86400	1635
7	7	ROCK	93.4447992	472.0081	62400	7022	57778	50	1	41200			53700	17900	161100	7020
8	8	ROCK	101.904839	527.3383	62400	8403	57778	50	1	41200			53700	17900	161100	8400
9	9	ROCK	110.143463	579.894	62400	9888	57778	50	1	41200			53700	17900	161100	9880
10	10	ROCK	118.169253	629.8788	62400	11473	57778	50	1	41200			53700	17900	161100	11470
	11															

APPENDIX D

Geological Cross Section

LEGEND

BH1 — BOREHOLE LOCATIONS

POSSIBLE BURIED TOPSOIL

WEAKLY METAMORPHOSED SANDSTONE & SILTSTONE OF NORTH RANGE GROUP:

SLIGHTLY TO MODERATELY WEATHERED

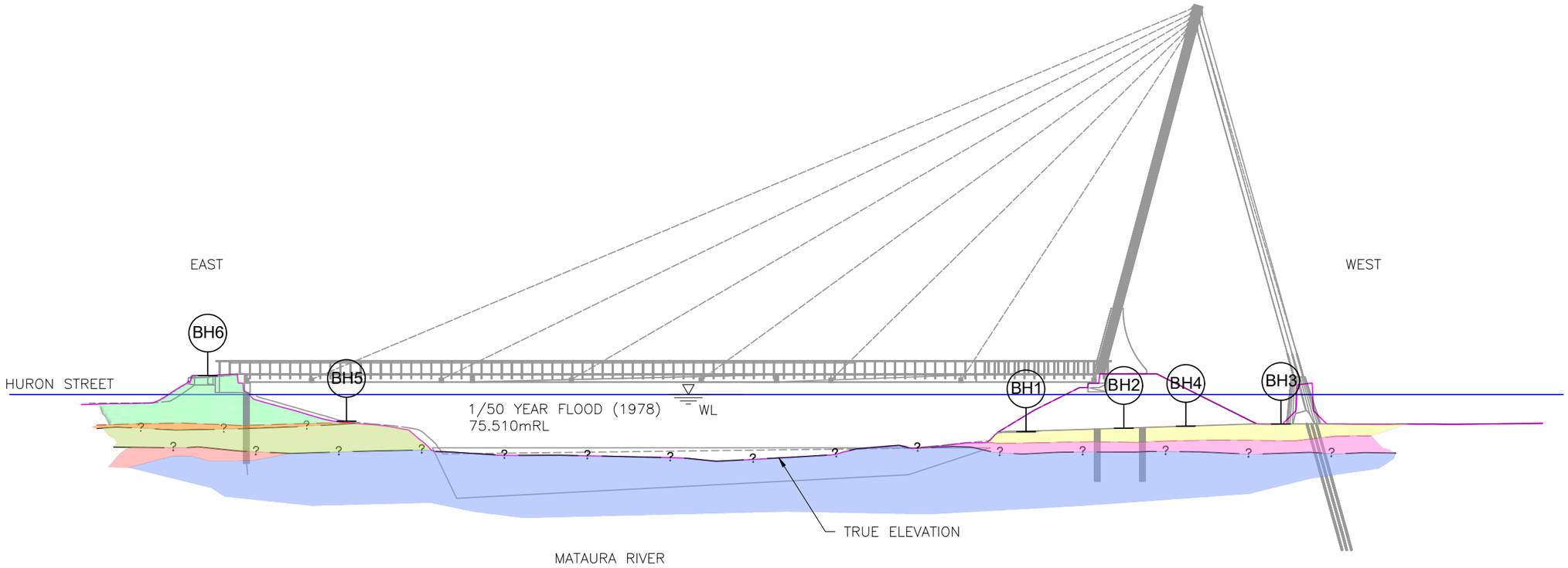
HIGHLY TO COMPLETELY WEATHERED

ALLUVIUM:

SILTY SAND & SANDY SILT

SANDY GRAVEL

SAND



5 0 10 20m

SCALE 1:500

FOR INFORMATION

REV	DATE	ISSUE	BY
1		ISSUED FOR INFORMATION	MD

DESIGN	DES CHECK	APPROVED FOR ISSUE
EM		DRAFT
DRAWN	CAD CHECK	
MD		
DATE DRAWN		ISSUE DATE
FEB 2020		11 / 03 / 19

RILEY CONSULTANTS
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CLIENT	GORE DISTRICT COUNCIL
ADDRESS	HURON STREET, GORE
PROJECT	LONGFORD SHARED PATH
SHEET TITLE	BRIDGE ELEVATION LOOKING DOWNSTREAM

ACENZ
GCS

CADFILE	190376-01.dwg
SCALE (A3)	ORIG. SHEET SIZE
1:500	A3
DRAWING No.	REV.
190376-01	1