Hughenden Irrigation Project Detailed Business Case

Geotechnical Investigation Interpretive Report

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Executive Summary

Jacobs has undertaken on behalf of the Hughenden Irrigation Project Corporation, a Detailed Business Case (DBC) for the proposed Hughenden Irrigation Project and associated distribution network. As part of the scope of works for the DBC, Jacobs has carried out a relevant geotechnical investigation and associated "Reference' design for the dam, appurtenant structures and the Distribution Network.

This geotechnical report provides a summary of the available relevant published sources and previous investigations, remote sensing of aerial imagery interpretation and LiDAR imagery, and project specific helicopter and walkover surveys, which were undertaken in order to design a project specific geotechnical investigation and derivation of accurate ground models.

The project specific geotechnical investigations were procured and supervised by Jacobs and included both an initial geophysical survey, followed by drilling and associated insitu testing and test pitting for the proposed scheme elements. The intended test pitting campaign for the Distribution Network, was ultimately cancelled due to access and COVID-19 challenges. In addition, hand specimens were also taken for potential construction material sources within the approximately a 50km radius from the project site. An associated suite of both soil and groundwater testing was undertaken at NATA accredited laboratories in Townsville and Brisbane.

The findings of the desk study, mapping/walkover surveys/ground truthing exercises, as well as the project specific ground investigations have been used to derive a geological model of the major components of the dam, as well as generic models for the Distribution Network.

The following sections provide high level summaries of the project specific interpreted ground conditions and the associated geotechnical considerations for the design and construction of the project scheme elements.

Ground Conditions

The site bedrock geology generally comprises interbedded Siltstone and Sandstone of the Wallumbilla Formation overlain by a variable thickness and composition of Holocene and Pleistocene Alluvium. The alluvium is cohesively dominated, with the occurring granular stratigraphy potentially linked to former channels. Subsurface conditions have been significantly influenced by palaeochannels which eroded into the underlying Wallumbilla Formation. Changes in depositional environments were believed to have occurred as a result of a period of sea level regression, which was subsequently infilled with Holocene and Pleistocene deposits as the relative sea level rose. The alluvium is deepest/thickest close to the present perennial surface water courses of the Flinders River, Stewart Creek and Back Valley Creek.

Reactive soils were also noted to be present across the site being typically less than 1.5m thick. These soils are believed to have been derived from the original basaltic residual soils that either were transported from colluvial slopes of the previously mentioned mesas in the area, &/or from former lava flows that have now been eroded by the River and creeks meandering across the inherent alluvial floodplains.

The slopes of the basalt mesas are draped in basalt talus and scree generally consisting of gravel to boulder size clasts.

During the investigation traces of gypsum were encountered throughout the soil horizons often occurring in concentrated masses. The gypsum may have formed as evaporite beds during significant climate change through the Quaternary during periods of higher rainfall (where gypsiferous laden water infiltrated through the soil profile – eluviation) followed by subsequent drought.

No surface or subsurface expression of evidence of major geological structural features (such as faults) has been found within or in close proximity to the project site.

The competent bedrock sequence is typically dominated by massive to very thickly bedded sub horizontal to horizontal bedding planes, however within the vicinity of the proposed Spillway discontinuity spacings reduce to very thinly bedded to closely jointed.

Seismic Considerations

No significant geological structural features are in close proximity to the project site. A project specific detailed seismic hazard assessment undertaken has calculated Peak Ground Acceleration (PGA) levels for the proposed HIPCo Saego Dam site for a 475-year return period (10% probability of exceedance in 50 years) of 0.0026 g and for a 10,000 year return period is 0.032 g, when considering earthquakes of magnitude 5.0 or greater with varying Vs30 values. The conclusion is that the risk of seismic events on the proposed works is considered to be very low.

Dam Piping Failure

As the main dam is to be founded at shallow depths, and the potential existence of permeable layers within the alluvial sequence, it is suggested that a grout curtain or slurry trench should be installed through the alluvium and be seated into the less weathered rockmass sequence over the dam Ch 2300-7825m (covering the Type B embankment).

Dam Stability

Based on the defined two typical sections (Residual Soil and deep Alluvium founding) for the main Saego Dam alignment both temporary and permanent slope stability FoS have been satisfied for a variety of dam levels on the upstream face and flood levels on the downstream side. Liquefaction analysis for the same typical embankment designs indicate high FoS (at least 2) against liquefaction at this site.

Spillway Considerations

With the presumption that the Spillway structure will be located on competent rock (defined here as moderately weathered and of low to medium strength or better), required excavation could be in the order of a maximum of 10 mbgl, or RL 254 to 257 mAHD.

The potential excavations required will be through typically very stiff to dense alluvial sediments and very low to low strength weathered rock horizons, and as such should be effectively excavated by the use of free-digging techniques by a suitable large excavator.

The Spillway and apron will be designed to withstand uplift pressures, being reservoir pressure at the upstream face varying linearly to tailwater pressure at the end of the apron. The critical feasible kinematic failure case is deemed to be sliding (rather than overturning) given the low strength of the siltstone.

Analysis indicates the probability of a kinematic feasible sliding failure mechanism below the Spillway is very low.

The Spillway stilling basin/apron and return channel (draining to Back Valley Creek) will potentially be in erodible materials and overtime could significantly deepen with the possibility of undermining the associated downstream cuttings and the Spillway downstream apron, as well as causing detrimental environmental effects (increasing sediment load in Back Valley Creek). With this in mind the Spillway return channel cuttings will be laid back on 1v:3h with a protective covering of locally won available rock armour (from the proposed basalt quarry) sized to suit the critical flow case (perhaps in the order of D50=750mm). This would extend over the full extent of the Spillway return channel to Back Valley Creek.

Weir Considerations

The proposed gated concrete Weir will require excavations in the riverbed of approximately 9.00 and 12.5m to found on competent rock.

In order to install the weir foundation a significant temporary works excavation will be required in both cohesive and potentially highly permeable granular alluvium perhaps in the order of 4-10m thick, being well below the interpreted winter water table level (approximately 1m below existing riverbed level).

To provide a stable and dry excavation it is likely that the majority of the excavation would need to be dewatered probably by a mixer of well points (at the top) and deep wells. The excavation dimensions would also have to be relatively conservative in design and probably require a two batter arrangement with a wide intermediate bench.

Analysis indicates the probability of a potential kinematic sliding failure mechanism below the Weir is very low, however anchors may still be required to resist water uplift pressures.

Diversion Channel

The maximum depth of cut required to form the Diversion Channel is approximately 12.6 mbgl, and a small section of fill will be required close to the interface with the main Sego Dam wall.

From the formulated ground model it is interpreted that the majority of the proposed cuttings can be effectively excavated by the use of standard plant (dozzers, excavators and graders), with only the potential requirement for D10 &/or D11 with tyne rippers or excavators with hydraulic breakers for approximately 1.5 km of the alignment.

Long term stability cut slope batter dimensions will be dependent on the encountered geology, and preliminary minimum cut slope designs have been provided, which take into consideration the potential erodibility of the Wallumbilla Formation sequence when exposed.

Excavated materials could be used for both required areas of fill and for the proposed trailing embankment.

Due to the very high probability of degradation/erosion of exposed both soils and rock sequences, for stability, to maintain full diversion dimensions and to minimise operational phase maintenance requirements, all cut and fill slopes will more than likely be required to be lined.

Construction Materials

Based on a variety of local interviews with Council, quarry operators and earthworks contractors, plus remote sensing, walkover surveys and specific sampling and associated laboratory testing, the following conclusions on construction material sourcing are made:

- Sufficient sand and gravel deposits for project construction needs have a good likelihood to be available within the region at probable economic processing and haulage costs; coarse filter materials (sandy gravels) could also be generated from specific hard rock crushing/processing circuits;

- The local basalt can be used for a variety of aggregate sources including rockfill, rip-rap, drainage, gabion, road base and concrete, although it may be variable in composition and thickness over the district; there are no issues with the volume available of these possible aggregates in terms of proximity to any potential scheme options, but this will require the establishment of project specific quarries;

- The location of suitable quality and quantity of low plasticity clay deposits for possible dam clay core sources within the widespread alluvial deposits, may be problematic to identify. Upper Holocene and

Lower Pleistocene clays are both reactive, dispersive and sodic in nature. The weathered basal sedimentary sequence of siltstones/sandstones/shales could be a possible source of clay for dam cores if processed, however their sodic & dispersive nature does not lend it to advantageous clay core sources. To alleviate these presently interpreted poor clay core sources may require appropriate amelioration with lime, use of more substantial dam filters, blending with inert local materials, and/or consideration of concrete or grout enriched dam upstream faces; in addition, careful moisture control of placed core materials will be required; the present base case for design and associated CAPEX estimation is based on the use of untreated near surface alluvial and residual soil clays as dam clay core (Type1A) & Earthfill (Type 1B) materials, with wider dam filters being employed; and

- Local concrete batching is available but may not meet a D & C contract needs.

Distribution Network

From the interpreted ground conditions, all pipelines are expected to be installed by free digging with excavators in trenches, with the exception of the proposed siphon crossing of the Flinders Rivers which will be presently installed by HDD.

In the alluvial floodplains, perched water tables could require battering back or the use of sheet piling and dewatering for trench stability. In these areas buoyancy and scour may be required to be dealt with by the use of sand bagging and rip-rap backfill for the majority of the trench over the pipe crown.

Reactive soil excessive swelling under the pipe invert may also need to be catered for by again similar measures for pipe buoyancy. Sodic soils could also be encountered in the pipeline trench alignments and a variety of control measures could be applied to deal with associated risks, including amelioration, the use of sand blocks or differing backfill compaction specifications.

The majority of trench excavated materials can be used as backfill over/around the pipelines, although as above they may need different compaction regimes and/or additives.

Due to the probability of encountering relatively deep rockhead at both of the proposed pumping station locations and the likelihood of improving density/shear strength of both alluvial and weathered rock sequences with depth (as indicated by the project specific investigations), 900mm diameter concrete piles driven provisionally to 15mbgl, have been considered as appropriate for both acceptable load and associated settlement criteria.

Important note about your report

The sole purpose of this Geotechnical Report is to present the findings of a geotechnical investigation carried out by Jacobs for the Hughenden Irrigation Project Corporation Pty Ltd ('the Client') in connection with the Hughenden Irrigation Project ("the site"). This report was produced in accordance with and is limited to the scope of services set out in the contract between Jacobs and the Client.

This report is based on assumptions that the site conditions as revealed through sampling and interpretation of other data sources, are indicative of conditions throughout the site. The findings are the result of standard assessment techniques used in accordance with normal practices and standards, and (to the best of Jacobs' knowledge) they represent a reasonable interpretation of the current conditions on the site.

Sampling techniques, by definition, cannot determine the conditions between the sample points and so this report cannot be taken to be a full representation of the sub-surface conditions. This report only provides an indication of the likely sub surface conditions.

Conditions encountered when site work commences may be different from those inferred in this report, for the reasons explained in this limitation statement. If site conditions encountered during site works are different from those encountered during Jacobs' site investigation, Jacobs reserves the right to revise any of the findings, observations and conclusions expressed in this report.

The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report.

In preparing this report, Jacobs has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by the Client and from other sources. Except as otherwise stated in the report, Jacobs has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

Jacobs has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose described above and by reference to applicable standards, guidelines, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report, to the extent permitted by law.

This report does not address environmental or geo-environmental issues including the presence of any contaminants or hazardous materials at the site.

This report should be read in full, and no excerpts are to be taken as representative of the findings. No responsibility is accepted by Jacobs for use of any part of this report in any other context.

This report has been prepared on behalf of, and for the exclusive use of, the Client, and is subject to, and issued in accordance with, the provisions of the contract between Jacobs and the Client. Jacobs accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party.

1. Introduction

The Hughenden region has long been a vibrant and integral part of the north Queensland, Queensland and Australian economies. It has been suffering long-term economic decline for several decades.

The region has abundant irrigable cropping lands but lacks a large-scale reliable water source. With the support of the Australian Government, and the vision and drive of local farmers and irrigators, the Hughenden Irrigation Project (HIP) has been developed to a stage where it demonstrates potential to harvest environmentally sustainable water from the Flinders catchment and to store and deliver this water for new irrigation enterprises to transform currently underutilised grazing land into economically productive irrigated agriculture.

The proposed HIP could be transformational for the Hughenden Community and the broader region, by potentially reinvigorating local and regional economies and improve social disadvantages.

Through specific government allocated funds a Preliminary Business Case (PBC) for the HIP was undertaken by Engeny Water Management (Engeny) in 2019-2020, for the Client Hughenden Irrigation Project Corporation Pty Ltd (HIPCO). The PBC considered various site and layout options for the scheme, each with differing water storage volumes and yields. Figure 3-6 presents the preferred PBC option of Saego Dam.

Essentially the Saego Dam option is an off-stream storage facility diverting water from the Flinders River, by a weir and significant channel arrangement, to a considerable holding basin retained by a long earth embankment with a central spillway. Water diversion from the Flinders River will only occur during seasonal high flow/flood events. The maximum storage for this scheme based on the Engeny February 2020 PBC Study Report was 190 GL. The stored water was to be delivered by a pipeline arrangement to identified agriculture land to the south of the Flinders River.

In December 2020, HIPCO awarded the Detailed Business Case (DBC) for the HIP to Jacobs. Through the project life cycle a number of iterations of scheme type, location and components have been studied by Jacobs, including previous options explored during the PBC, “in-River” dam options, and various off stream storage options and supplementation by potential well fields. A Multicriteria Analysis (MCA) of a number of scheme options has been undertaken, which has led to the present base case project design.

This report provides an assessment of the ground engineering aspects of the proposed base case HIPCO scheme, as set out in Drawing No. IS362200-0000-CI-DG-0002 in Appendix A.

The common features of the proposed HIPCO scheme are set out in Table 1-1.

Table 1-1: Key project design characteristics

Characteristic	Project configuration
Dam	
Location	Flinders River (@Spillway) Easting (m): 795644 Northing (m): 7715639
Dam name	Saego Dam
Dam status	Proposed dam

Characteristic	Project configuration
Purpose of storage	Irrigation and water supply
Dam type	Earth embankment incorporating a fixed crest spillway
Catchment area:	7,559 km^2
Full supply level (FSL)	265.1 mAHD
Storage at FSL	160,982 ML
Storage volume at crest level	406,550 ML
Surface area at FSL	3,676 ha
Surface area at crest level	7,360 ha
Dead storage level	256 mAHD
Hazard category	High B
Acceptable flood capacity (AFC)	Minimum of 1 in 10,000 Annual Exceedance probability (AEP)
Failure impact rating	Category 1
Main embankment and spillway	
Dam type	Zoned earth embankment with rockfill
Dam crest elevation	270.5 mAHD
Dam crest width	7.1 m
Dam crest length	7,650 m
Maximum embankment height	20.5 m
Spillway type	Ungated mass concrete ogee crest with smooth concrete chute and Type II stilling basin
Spillway crest elevation	265.1 mAHD
Spillway crest length	200 m
Spillway capacity	1 in 10,000 AEP event or 2,853 m^3/s outflow
Maximum height	6 m
Weir & Trailing Embankment	
Weir type	Mass concrete gravity structure with combination of lay-flat and vertical lift gates
Weir FSL	265.1 m AHD
Weir Dead Storage Level	259.1m AHD
Weir crest width	10 m
Weir crest length	95 m
Maximum height	10.6 m
Consequence Category	Non-Referable structure
Diversion Channel & Embankment	
Channel length	5.069 km
Slope	Due to site geometric constraints no slope has been provided
Channel width	50 to 100m
Channel bed invert level	260 mAHD
Channel FSL	265.1 mAHD

Characteristic	Project configuration
Channel depth @ FSL	5.1 m (increases to 7.1 m @ Back Valley Creek crossing)
Maximum ¢ cutting height	12.6 m
Maximum channel flow capacity	15,000 ML/d
Road crossings	Steel bridge
Channel flow regulating structure	Outlet conduits (16 No. 3 x3m) with flaps & vertical lift gates (3 x 3m) close to NE dam abutment
Embankment type	Zoned earth embankment
Embankment length	3.21 km
Embankment crest elevation	267 m AHD
Embankment crest width	7.1 m
Maximum height	9 m
Consequence Category	Non-Referable structure
Diversion Channel Embankment Emergency Overflow	
Type	Rock mattress overflow structure
Design flood AEP	1:100 AEP (265 m^3/s / 22,900 ML/day)
Overflow sill level	265.5 mAHD
Overflow width	80 m
Design life	100 years
Outlet works	Inlet structure – wet well intake tower; conduit 2000mm diameter MSCL concrete encased pipe
Fish passage	Weir - single fish lock 5m high US, 4m high DS; upstream operating levels 260-266 mAHD
Mechanical asset design life	Nominal 35 years
Electrical asset design life	Nominal 25 years
Distribution Network	
Overall length	60 km
Outlet Pump Station Pipeline	a) 1981 OD MSCL suction b) 1981 OD MSCL delivery up to Flinders River crossing c) 2 x DN1200 PE and 1 x DN 1200 PE within distribution network
Siphon Crossing pipeline	3 No DN1200 Polyethylene pipes
Booster pump station pipeline	a) 1200 Ø MSCL suction and delivery within pump station building b) 1 x DN1200 PE rising main
Outlet Saego Dam pump station	
Total pumping head	Up to 50 m TDH
Pumping capacity	Horizontal split case 2 x 20, 2 x 40 & 3 x 100 ML/day
Peak power	3,100 kW
Booster pump station	
Total pumping head	Up to 40 m TDH
Pumping capacity	Horizontal split case 2 x 10, 2 x 20 & 3 x 50 ML/day

Characteristic	Project configuration
Peak power	800 kW
Main pipeline from Siphon south	HDPE, DN1200 PN6.3
General Distribution Network	HPDE, varying diameter DN1200 to DN630 min. PN6.3
Civil asset design life	50–100 years
Mechanical asset design life	Nominal 35 years
Electrical asset design life	Nominal 25 years

This Geotechnical Investigation Report provides:

- A summary and review of the available published information and previous investigations.
- Details of the project specific geotechnical investigations.
- Definition of appropriate ground models and design parameters for the proposed main scheme elements.
- Interpretation and associated Reference Design for both dam and appurtenant structures, as well as the Distribution Network; and
- Identification of key project geotechnical risks and potential mitigation measures.

2. Geological and Geomorphological Setting

To gain an appreciation of the topography and geomorphology of the Hughenden area, both the Hughenden and Richmond sheets of the 1:250,000 Geological Series were considered (see Figure 2-1). These are listed in Table 2-1.

Table 2-1: Desktop Sources Reviewed

Title	Date	Author
1:250,000 Geological Series – Explanatory Notes: Richmond, Queensland	1970	Bureau of Mineral Resources, Geology and Geophysics. Compiled by R. R. Vine
1:250,000 Geological Series – Explanatory Notes: Hughenden, Queensland	1974	Bureau of Mineral Resources, Geology and

A topographic map of the project area, showing 10m contours is presented in Figure 2-2, and in Appendix A. The town of Hughenden is at approximate elevation 320m AHD, dropping gradually to the south and west over the Lolworth Range plateau, and rising in the east and north towards the Great Dividing Range. The source of the Flinders River is on the western side of the Great Dividing Range, approximately 110km north-east of Hughenden. The Flinders River and its tributaries drain west / north-west towards the Gulf of Carpentaria (see Figure IS362200_P_009 in Appendix A).

The project site is located approximately 45km north west of Hughenden in an area comprised of alluvial plains of the Flinders river in the central and southern sections of the proposed Saego Dam location with basalt plateaus forming the northern boundary of the site. The alluvial plains are generally flat or gently sloping to the south to southwest towards the Flinders River.

Basalt that once flowed down broad, shallow valleys, now forms the protective caps to the Dutton and most of the Grampian Plateau. The plateaus rise typically steeply above the alluvial plains gaining typically 60-70m in height over the plains (see Photograph 2-1).

Based on the published geology there are no identified major faults or other major geological structures within the region (see Figure IS3262200_G014, Appendix A).

2.1 Dam Inundation Area Geology, Soils and Geomorphology from Published Sources and Mapping

The alluvial sediments of the Flinders River are comprised of clay, silt and sand with minor gravel layers, which are underlain by the sedimentary rocks of the Wallumbilla Formation. Figure 2-2 provides the main published soil classifications for the dam and inundation area indicating that that it is composed of Vertosols or reactive soils having probably been derived from eroded flood basalts.

The Wallumbilla Formation is the most widely distributed marine unit within the Great Australian Superbasin, extending through the Euromanga, Surat, Carpentaria and Laura basins (P. A. Jell, 2013) and comprises of early cretaceous nearshore to shallow to basinal marine sediments (Vine, Bastian, & Casey, 1970). At the project site, the Wallumbilla Formation is divided into three members: the Ranmoor Member, Jones Valley Member and the Doncaster Member. The Wallumbilla Members general descriptions are shown in Figure 2-1 and are typically comprised of Mudstone, Siltstone and labile and glauconitic Sandstone (see Photograph 2-2).

Photograph 2-1: Typical steep sided slopes of basalt capped mesa rising above the alluvial floodplains

Photograph 2-2: Typical interbedded sandstones and shales/mudstones of the Wallumbilla Formation

The basalt, described as olivine basalt of the Sturgeon Basalt field (Vine, Bastian, & Casey, 1970), is generally 30m thick and prevents erosion of the underlying cretaceous sediments resulting in the development of the

typical mesa formations of the region (see Photograph 2-3). The flat top plateaus are generally 50-70m in height above the alluvial floodplains, reaching elevations of typically RL 370 mAHD to the north east of the dam site and approximately RL325 mAHD adjacent to the western dam abutment. The slopes of the plateaus are dominated by talus comprised of basalt boulders overlying sediments of the Wallumbilla Formation.

Photograph 2-3 : Basalt outcrop forming the uppermost stratigraphy of the north side of the Alstonvale Valley

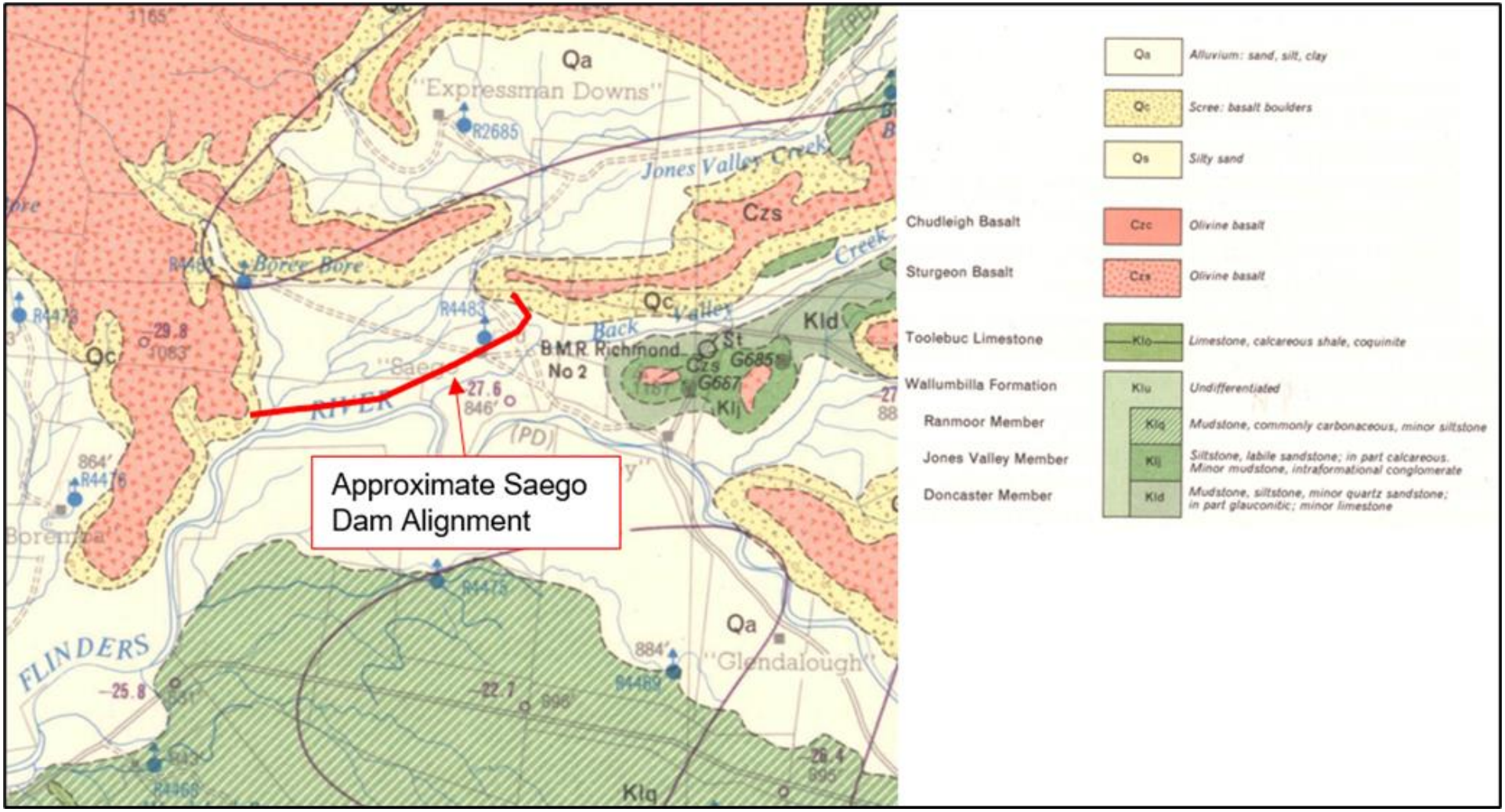


Figure 2-1: Regional geological setting of Saego Dam

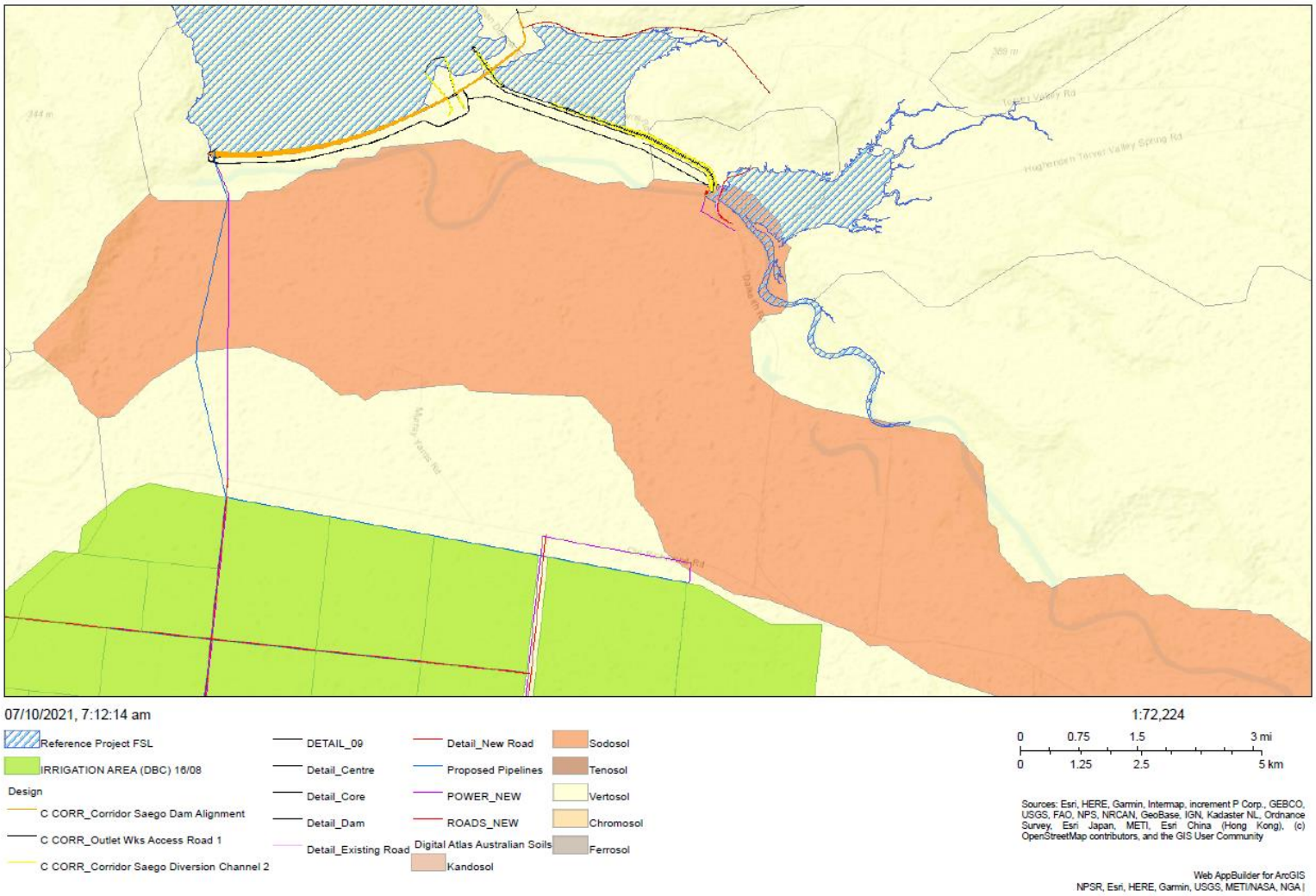


Figure 2-2: Published Soils of the HIPCO project area

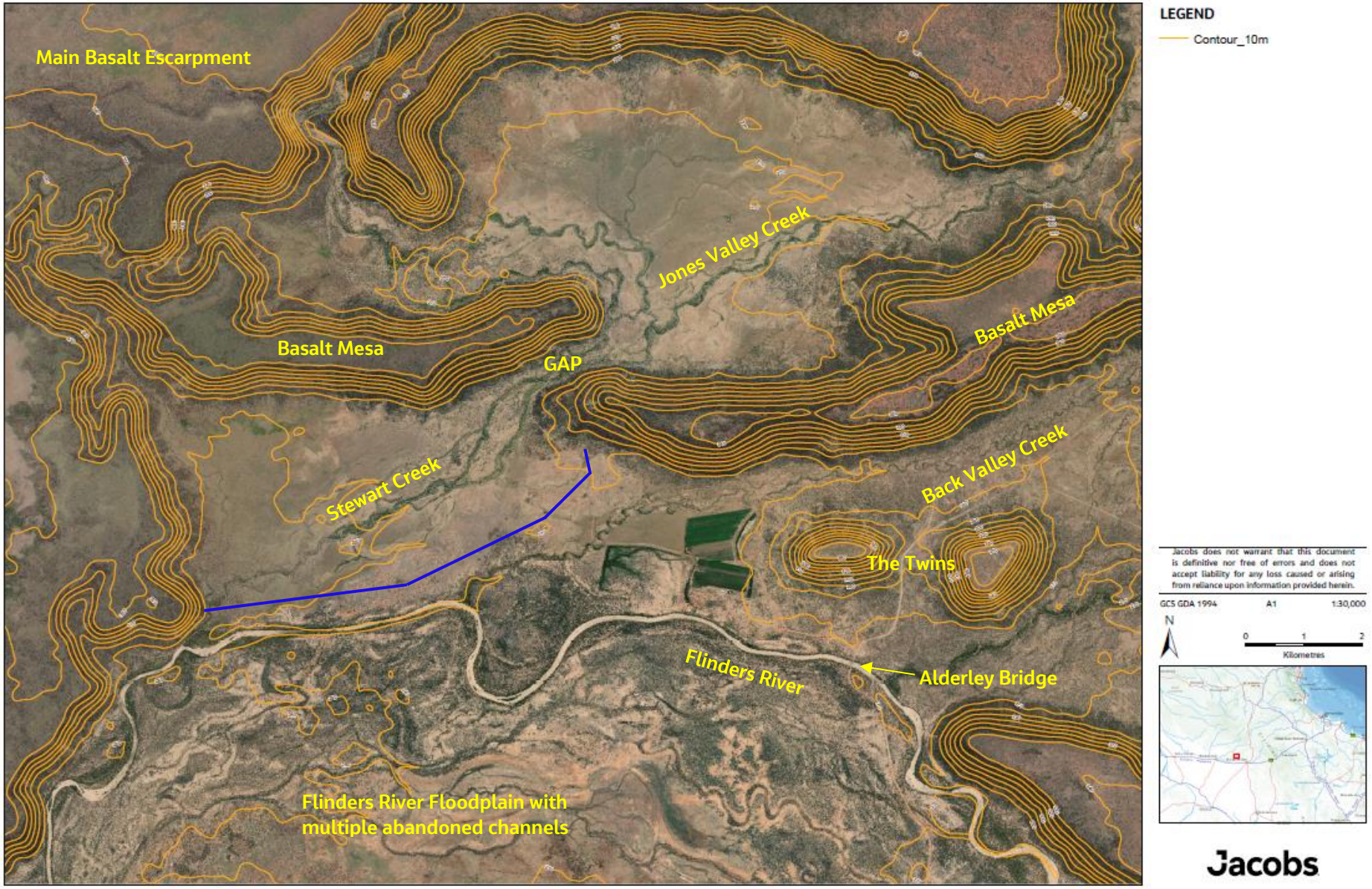


Figure 2-3: Main topographical & geomorphological characteristics of the HIPCO project area

Aerial Photo Interpretation (API) and hillshade returns (from the most recent LiDAR survey – see Figure IS362200_G008, Appendix A) shows abandoned meanders (oxbows) and abandoned or high flow only channels on a braided river system, with most of the channels and braids towards the southern margin of the main Flinders River channel. These abandoned channels suggest a northward migration of the main river channel during the late Quaternary. Stewart Creek drains into the main Flinders River close to the proposed western dam abutment (see Photograph 2-4). The Creek runs through from the northeast GAP in a southwest draining direction and will be inundated by the proposed dam. The Creek itself has probably meandered across its associated floodplain.

Photograph 2-4: Stewart Creek draining into the Flinders River close to the dam western abutment

South of the proposed HIP Dam site, the Flinders River Floodplain extends between 2km to 4 km away from the current river channel with again a number of distinct abandoned channels (see Figure 2-3).

2.2 Distribution Network Geology, Soils and Geomorphology

The Distribution Network, located to the south of the proposed dam site and on the southern margin of the Flinders River, is situated in an area where the Ranmoor Member of the Wallumbilla Formation is shown to outcrop. The southern edge of the Distribution Network is noted to extend into the Toolebuc Limestone (Vine, Bastian & Casey, 1970).

The northern part of the Distribution Network is underlain by quaternary sediments of the Flinders River comprised mostly by floodplain alluvium with abandoned meanders and braided channels that flow only during flood events (see Figure 2-2 & 2-3). Approximately 50% of the proposed main pipeline feed alignment from the Flinders River siphon to the Irrigation Area is indicated to be composed of Sodosols with a potential sodic nature. The remaining section of this pipeline and the entire Distribution Network is shown in Figure 2-2 to be composed of Vertosols with a potential reactive nature.

The central and southern part of the proposed Irrigation Area, is underlain by thin residual soils with a geomorphological expression of gentle rolling slopes up to 15 m in height, vegetated with grazing pastures.

2.3 Existing Water Wells and Groundwater Conditions

Records of historic groundwater wells obtained from the Queensland Government Groundwater Information database are presented on Figures IS362200_G011 & G015 (Appendix A), while the key details are summarised in Table 2-2. These provide indications of both regional groundwater and geological stratigraphy.

The groundwater information is concentrated locally around the Saego Dam site, with the wells typically installed within the Alluvial floodplain. Standing water levels vary from 2.2 to 20.4 mbgl, but typically 7 to 14mbgl. Pumping tests (generally undertaken on well first installation) indicated yields of between approximately 1 to 81 l/s, but are typically <10 l/s, with many of the wells reported to be artesian in nature. It should be note that yields for wells may reduce with time. The artesian flows experienced in the area have been substantiated by discussions with local landowners. Many of the old wells installed have now been abandoned or been destroyed.

The alluvial thickness typically varies from 9 to 25m and is described in the driller's logs as clay, sand, sandy clay, black soil, silt, sandy silt, silty clay and mud. At two (2) locations (4482 and 4483), shale was encountered from the surface (i.e. no alluvium).

The underlying rock is generally described as blue shale, shale and sand bands, yellow and grey shale, shale and quartz bands, mudstone, siltstone. The majority of bores terminated within this unit, at depths between 15-30m below ground level, however the shale extends to much greater depths. Below the shale, the 'Hooray Sandstone' unit was encountered, the top of this unit varies from 93 to 122m below ground level. The deepest bore was drilled to 290m below ground level.

Table 2-2 Summary of available groundwater well data from the Queensland Government GeoResGlobe service.

Well Ref.	Status	Type	Date Drilled	E (m)	N (m)	Depth (m)	Geology Noted During Installation	Yield (l/s)	pH	Standing Water level (mbgl)
2683	Existing	Artesian - Ceased	1/01/1914	805014	7721538	182.9	No data available	-	7.6 to 8.6	-
2685	Abandoned and Destroyed	Artesian - Uncontrolled Flow	1/09/1914	795879	7732154	152.0	0 to 21m Yellow Clay (Alluvium) 21-122m Blue Shale >122m Sandstone	-	-	-
4475	Abandoned and Destroyed	Artesian - Uncontrolled Flow	1/06/1910	794870	7709913	269.1	No data available	21.18 to 81.00	7.3 to 8.2	-
4482	Abandoned and Destroyed	Artesian - Uncontrolled Flow	28/03/1918	790305	7717772	141.4	0 to 15.24m Yellow Shale 15.24 to 118.87m Shale, sand bands >118.87m Hooray Sandstone	2.75 to 4.55	7.9 to 8.1	9.75
4483	Existing	Artesian - Uncontrolled Flow	29/08/1929	796108	7716081	91.4	Yellow & Grey Shale from surface	1.25 to 4.21	7.9 to 8.0	6.47
69423	Existing	Sub-Artesian Facility	1/12/1985	798842	7714374	3.0	River Bore 0-3m Sand Alluvium	-	-	-
69424	Existing	Artesian - Controlled Flow	5/02/1987	802764	7721670	151.0	0-2m Black Soil 2-13m Sand and Clay Alluvium 13-40m Grey & Yellow Clay 40-97m Shale >97m Hooray Sandstone	-	-	-
69433	Existing	Artesian - Controlled Flow	1/02/1987	793041	7715475	91.4	No data available	4.68	8.0 to 8.3	2.18 to 11.49
69591	Existing	Artesian	30/11/1990	799304	7714718	294.1	0-9m Yellow Brown Clay Alluvium 9-93m Shale 93-118m Hooray Sandstone 118-136m Shale & Quartz Bands 136-286m Hutton Sandstone	25	7.5	18.38

Well Ref.	Status	Type	Date Drilled	E (m)	N (m)	Depth (m)	Geology Noted During Installation	Yield (l/s)	pH	Standing Water level (mbgl)
93232	Existing	Artesian - Controlled Flow	14/07/1996	795728	7722129	153.2	Base of bore Hooray Sandstone	3.52 to 5.57	7.7 to 7.8	6.44 to 9.22
93416	Existing	Artesian - Controlled Flow	4/12/1997	795075	7710276	269.3	0-1m Topsoil 1-15m Clay 15-269.3 Mudstone/ Siltstone / Sandstone	29.46 to 32.16	7.4 to 8.1	17.61 to 20.43
118005	Existing	Sub-Artesian Facility	14/08/2002	804368	7708727	16.5	0-9.5m Brown Clay 9.5-12.2m Brown Sandy Clay 12.2-16.5m Sand Gravel Coarse	3.15	-	12.2
118483	Existing	Artesian	4/09/2004	799455	7716238	290.0	0-10m Yellow Brown Clay Alluvium 10-176.8m Shale, minor Sst bands	38	7.4 to 7.5	16
146490	Existing	Artesian - Controlled Flow	1/06/2012	790467	7718003	169.5	0-1.50m Black Soil & Boulders 1.5-18.0m Yellow Clay Alluvium 18.0-119m Shale 119-169.5 Hooray Sandstone	-	-	-
163562	Existing	Sub-Artesian Facility	24/08/2015	803125	7709215	18.0	0-4m Topsoil, Black 5-16.5m Loam, clay, sand 16.5-18m Mudstone, Grey	1.00	-	13.7
183511	Abandoned and Destroyed	Sub-Artesian Facility	7/05/2019	802804	7709445	18.5	0-18m Clay, silt, sand 18-18.5m Clay, Gravels	-	-	-
183512	Existing	Sub-Artesian Facility	8/05/2019	802742	7709200	19.4	0-18.3m Clay, sand 18.3-19.4m Clay / Shale	-	-	11.0
91500056	Abandoned and Destroyed	Sub-Artesian Facility	11/04/1967	802155	7709284	65.0?	No data available	-	-	-
91500057	Abandoned and Destroyed	Sub-Artesian Facility	11/04/1967	802243	7710062	19.8	0-18.2m Clay, sand 18.2-19.8m Shale	-	-	-
91500058	Abandoned and Destroyed	Sub-Artesian Facility	11/04/1967	802355	7710864	15.2	0-13.7m Sandy Silt, clay, sand 13.7-15.2m Shale	-	-	-

Well Ref.	Status	Type	Date Drilled	E (m)	N (m)	Depth (m)	Geology Noted During Installation	Yield (l/s)	pH	Standing Water level (mbgl)
91500059	Abandoned and Destroyed	Sub-Artesian Facility	7/11/1968	802672	7711260	20.4	0-.9m Topsoil 0.9-11.3m Sand, Gravel, Clay 11.3-20.4m Mudstone	-	-	-
91500060	Abandoned and Destroyed	Sub-Artesian Facility	11/04/1967	802357	7711615	50.0	0-13.71m Silt, Sand & Clay Alluvium >13.71m Shale	-	8.0	-
91500061	Abandoned and Destroyed	Sub-Artesian Facility	13/04/1967	798877	7720672	24.38	0 to 11.27m Clay Alluvium 11.27-24.38m Shale	-	-	-
91500062	Abandoned and Destroyed	Sub-Artesian Facility	13/04/1967	800713	7721597	19.91	0 to 18.9m Clay Alluvium 18.9-19.91m Shale	-	-	-
91500063	Abandoned and Destroyed	Sub-Artesian Facility	13/04/1967	798413	7714718	19.81	0-15.24m Silt, Clay & Sand Alluvium 15.24-19.81m Shale	-	-	-
91500064	Abandoned and Destroyed	Sub-Artesian Facility	13/04/1967	799595	7714246	19.81	0-15.24m Clay, Sand & Silt Alluvium 15.24-19.81m Shale	-	-	-
91500065	Abandoned and Destroyed	Sub-Artesian Facility	12/04/1967	789006	7709766	24.38	0-2.4m Silty Soil 2.4-22.9m Clay, Silty Clay 22.9-24.4m Shale	-	-	-
91500066	Abandoned but still useable	Sub-Artesian Facility	9/11/1968	788339	7709956	21.9	0-17.8m Silty Clay, sand, clay 17.8-21.9m Mudstone	-	7.3 to 7.4	6.96 to 8.7
91500067	Abandoned but still useable	Sub-Artesian Facility	12/04/1967	788683	7710138	24.4	0-21.3m Silt, sand, clay, gravel 21.3-24.4m Shale	-	-	-
91500068	Abandoned but still useable	Sub-Artesian Facility	13/04/1967	788444	7710453	30.5	0-25m Mud, sand, gravel, clay 25-30.5m Weathered shale (clay)	-	7.6 to 7.8	7.45 to 9.81
91500069	Abandoned but still useable	Sub-Artesian Facility	12/04/1967	788172	7710804	29.0	0-21.3m Silt, sand, mud, gravel 21.3-29m Shale	-	7.8 to 7.9	8.4 to 10.87
91500070	Abandoned but still useable	Sub-Artesian Facility	7/11/1968	788784	7711294	16.2	No data available	-	7.2 to 7.3	7.13 to 10.41
91500071	Abandoned but still useable	Sub-Artesian Facility	9/11/2968	788584	7711447	14.9	No data available	-	7.4	3.25 to 14.79

2.4 Remote Sensing

As part of these preliminary geotechnical assessments, Jacobs has undertaken a review of available satellite imagery (Google Earth) and aerial photography as sourced from the Queensland Government QImagery website (https://qimagery.information.qld.gov.au/) covering both the PBC options and the general environs of the Hughenden district.

Table 2-3 sets out the aerial photography that has been reviewed as part of this report.

Table 2-3: Aerial Photography Imagery Reviewed

Date	Type	Image Reference	Scale/Flying Height
24/10/1960	Black & White	Q1080 – 61 &64	13,000 ft
09/11/1963	Black & White	Q1463 – 5026, 5028 to 30, 5032	3.658m
10/11/1963	Black & White	Q1466 – 5068, 5070 to 5073	4,328m
09/07/1966	Black & White	CAB302 – 178,182, 186 & 190	25,000 ft
24/02/1974	Black & White	Q2172-3	3,048m
01/08/1976	Black & White	QAP3177 – 5025	4,300m
01/09/1989	Black & White	QAP4840 – 122 to 124	6,900m
16/03/1998	Colour	QAP5392 – 53 to 60, 106, 110 & 111	7,540m
11/08/2001	Colour	QAP5893 – 52, 53, 60 & 61	6,496m
16/07/2002	Colour	QAP6025 – 89 to 97, 113, 114 to 123, 130 to 132	6,496m

Remote sensing was undertaken to gain a high-level knowledge of the attributes of the topography and geomorphology of the region of the project, as well as provide potential targets for later construction material sources through ground truthing and/or actual subsurface geotechnical investigations.

The main findings of the remote sensing exercise were:-

- No significant evidence of landslides or slope stability features associated with the prominent topographical high areas, ie., breaks in slopes of the flood basalt mesas – this will apply to rim stability assessments for the main Saego Dam inundation area.
- The flood basalt mesas have considerable colluvial &/or residual slopes extending most of or the entirety of the descending slopes to the river valley basins. They are generally covered by a basalt boulder ground surface.
- The Flinders River and the major creeks of the region show a considerable number of abandoned channels, previous meanders, shoots and cut offs, and possibly infilled ox-bow lakes. The nature of the fluvial environment suggest that these water courses have moved considerably within their catchments over time and therefore there is a high likelihood that the associated floodplain alluvial deposits have been reworked a number of times. This may mean that the alluvial soils could be thick and vary considerably in terms of consistency and composition over short distances within the floodplain. The present Flinders River channel in places has both sand banks/terraces and islands within its bank full limits.

- Three main vegetation cover/types are evident from the aerial photography, being generally grassland and low trees on the flood basalt mesas, mature trees immediately adjacent to river/creek, and grasslands within the valley floodplains.
- Existing agriculture is limited in extent occurring in and round the proposed Saego Dam site and consisting of irrigated strips & pivot irrigated circles, with water presumably being either directly drawn from the Flinders River catchment or major creeks draining into it.
- A review of the aerial imagery suggests multiple locations within the Flinders River floodplain where potential construction material sourcing locations may be apparent. These include both sand and gravel and clay. The dip or deposition slopes at the foot of the flood basalt mesas may also be potential sources of clay for earthen embankments, however these may also more likely to have a reactive nature; and
- Observations suggest that there is no evidence of major faults or other major geological structures within the region.

3. Previous Investigations

Limited historical geotechnical investigations have been undertaken in and around the Hughenden area, as set out in the following sections.

3.1 Alderley Bridge Crossing

Geotechnical investigations were carried out in 2019 on behalf of the Flinders Shire Council by Construction Sciences Pty Ltd for the new Alderley Bridge across the Flinders River. Nine boreholes and eight test pits were drilled / excavated within the bridge and abutments footprint. Boreholes were drilled into the bedrock and between 14.02m to 17.35m below the ground level. While test pits were excavated to 1.50 mbgl (see Figure 3-1). No surface elevation levels (RL) were provided in the investigation logs.

Table 3-1 provides a summary of the field work undertaken along the Alderley Bridge alignment including, investigation locations and termination depths. These locations are also shown on Figure IS362200_G012 in Appendix A.

Table 3-1 Summary of fieldwork locations and termination depth, Alderley Bridge Crossing

Test Reference	Easting (m)	Northing (m)	Termination Depth (m)	Remarks
BH01	801079.00	7713531.27	15.38	Eastern side of the old causeway
BH02	801090.47	7713543.85	15.92	
BH03	801088.06	7713559.95	15.00	
BH04	801100.53	7713572.00	15.60	
BH05	801097.13	7713588.63	17.35	
BH06	801109.56	7713600.18	14.30	
BH07	801106.19	7713617.31	14.40	
BH08	801118.62	7713628.85	14.02	
BH09	801115.25	7713645.99	14.40	
TP01	801045.00	7713382.00	1.5	South approach
TP02	801052.00	7713423.00	1.5	
TP03	801061.00	7713460.00	1.5	
TP04	801072.00	7713495.00	1.5	
TP05	801122.00	7713652.00	1.5	North approach
TP06	801134.00	7713690.00	1.5	
TP07	801147.00	7713728.00	1.5	
TP08	801160.00	7713766.00	1.5	

Table 3-2 and Table 3-3 below presents a summary of the ground conditions encountered during the investigations carried out by Construction Sciences along the bridge alignment.

Figure 3-1: Alderley Bridge Crossing Geotechnical Investigations Layout Plan

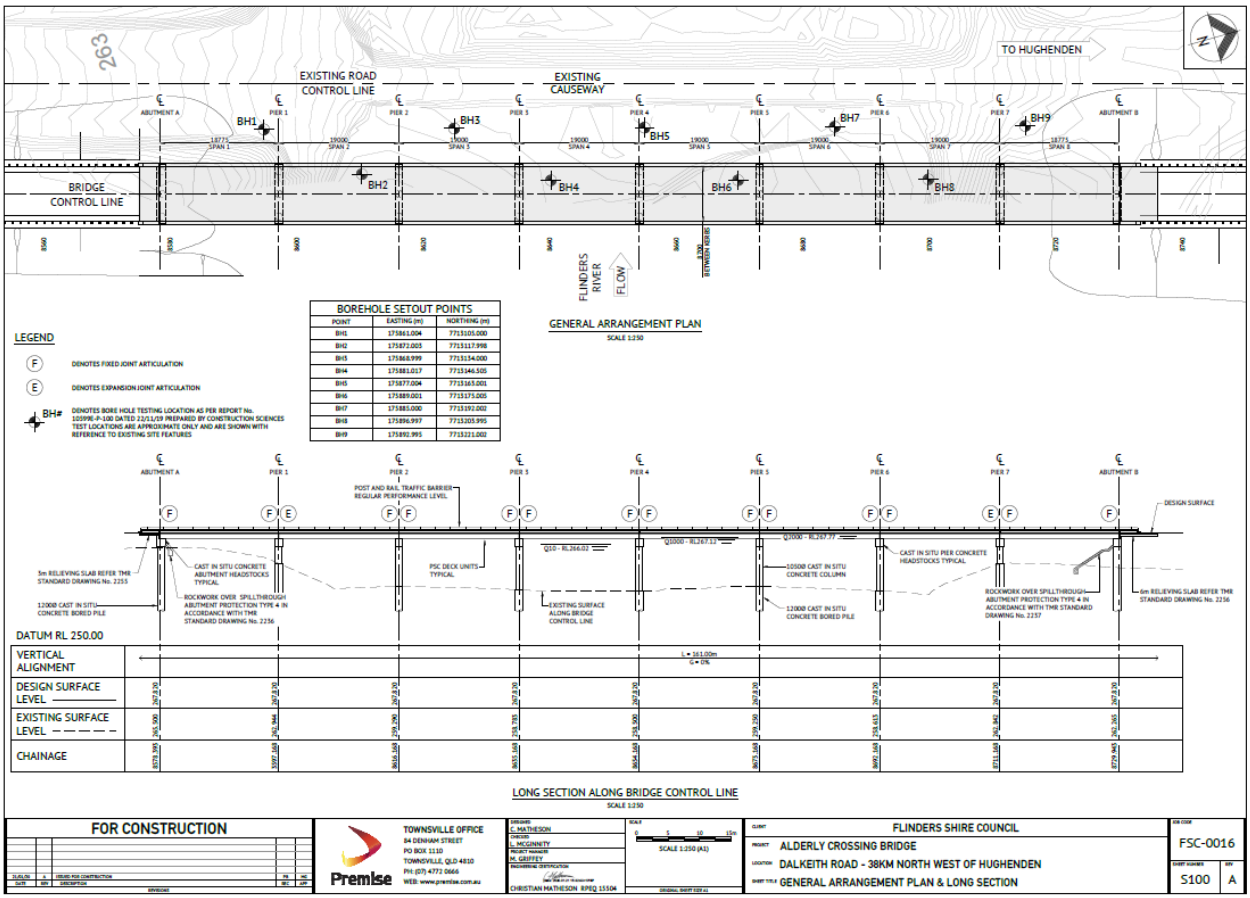


Table 3-2 Summary of subsurface profile from boreholes (BH01 – BH09)

Soil Descriptions	BH01	BH02	BH03	BH04	BH05	BH06	BH07	BH08	BH09
	Depth (mbgl)								
FILL: Silty Sand	-	-	0.0 – 0.5	-	-	-	-	-	-
Slty/Clayey Sand/ Sand	0.0 – 2.0	1.7 – 2.1; 2.3 – 3.0	-	4.8 – 5.1	1.9 – 3.0	-	1.8 – 2.5	-	1.2 – 2.1
Sandy Clay	2.0 – 3.2	-	-	-	3.0 – 3.9	-	-	-	-
Gravelly Sand	3.2 – 4.8; 6.3 – 6.8	0.0 – 1.7; 2.1 – 2.3; 3.2 – 4.4	0.5 – 2.1; 3.3 – 5.0	0.0 – 2.1; 3.3 – 4.8	0.0 – 1.9	0.0 – 1.3	0.0 – 1.8; 2.5 – 3.1	0.0 – 1.2	-
Sandy Gravel	-	4.4 – 5.5	-	-	-	1.3 – 2.1	3.1 – 4.3	1.2 – 2.1	-
Clayey Gravel	-	5.8 – 6.2; 6.8 – 7.1	5.8 – 6.1	-	3.9 – 4.3; 5.8 – 6.1	-	-	-	-
Clay / Organic Clay / Silty Clay	4.8 – 6.3; 6.8 – 8.0	3.0 – 3.2; 5.2 – 5.8	2.1 – 3.3	2.1 – 3.3	-	2.1 – 4.1	-	2.1 – 3.1	0.0 – 1.2; 2.1 – 3.0
Mudstone	8.0 – 15.35 (TD)	6.2 – 6.8; 7.1 – 15.92 (TD)	5.0 – 5.8; 6.1 – 15.0 (TD)	5.1 – 15.6 (TD)	4.3 – 5.8; 6.1 – 17.35 (TD)	4.1 – 14.3 (TD)	4.3 – 14.4 (TD)	3.1 – 14.02 (TD)	3.0 – 14.4 (TD)

Table 3-3 Summary of Alderley Bridge Crossing subsurface profile from test pits (TP01 – TP08)

Soil Descriptions	TP01	TP02	TP03	TP04	TP05	TP06	TP07	TP08
	Depth (mbgl)							
TOPSOIL: Silty / Clayey Sand	0.0 – 0.2	0.0 – 0.1	0.0 – 0.2	0.0 – 0.15	-	-	-	-
TOPSOIL: Gravelly Sand	-	-	-	-	-	0.0 – 0.15	0.0 – 0.2	0.0 – 0.1
Clayey Sand	0.2 – 0.6	0.1 – 0.78	0.2 – 1.5 (TD)	-	-	-	0.4 – 1.5 (TD)	-
Sandy Clay / Clay; with Sand / Clay, with gravel / Sandy Silt	0.6 – 1.5 (TD)	0.78 -1.5 (TD)	-	0.15 – 1.5 (TD)	0.9 – 1.5 (TD)	-	-	0.7 – 1.5 (TD)
Gravelly Sand	-	-	-	-	0.0 – 0.2	0.15 – 0.7	-	0.1 – 0.7
Sandy Gravel	-	-	-	-	0.2 – 0.75	0.7 – 1.1	0.2 – 0.4	-
Clayey Gravel	-	-	-	-	0.75 – 0.9	-	-	-
Clay	-	-	-	-	-	1.1 – 1.5 (TD)	-	-

Note: TD = Termination Depth

Limited laboratory testing was undertaken on recovered samples from this investigation campaign. The testing comprised particle size distribution, Atterberg Limits with linear shrinkage, moisture content, Emerson Class Number, compactions and CBR's, soil aggressivity and point load tests on rock samples. Laboratory test results are summarised in Table 3-4 to Table 3-7 inclusive.

Table 3-4 Summary of soil classification test results from Alderley Bridge Crossing investigation

Particle Size Distribution, Atterberg Limits, Linear Shrinkage, Moisture Content, Emerson Class									
Test Ref	**Depth (mbgl)**	**% Gravel**	**% Sand**	**% Clay and Silt**	**Liquid Limit (%)**	**Plasticity Index (%)**	**Linear Shrinkage (%)**	**MC (%)**	**Emerson Class**
TP01	0.7 – 1.5	-	48	52	26	5	6.5	7.7	-
TP04	0.15 – 1.5	-	28	72	32	9	8.0	11.7	-
TP05	1.0 – 1.5	8	28	64	62	37	15.0	14.2	-
TP07	0.4 – 1.5	28	55	17	42	18	12.5	6.1	-
TP08	0.7 – 1.5	9	29	62	41	18	12.5	4.7	-
BH01	2.0 – 2.45	-	49	51	-	-	-	-	-
	4.0 – 4.45	-	-	-	32	16	6.5	-	-
	7.0 – 7.45	-	-	-	49	32	15.0	-	-
BH02	1.0 – 1.45	-	-	-	-	-	-	-	5
BH05	3.0 – 3.45	12	52	36	-	-	-	-	-
BH07	2.0 – 2.45	27	68	5	-	-	-	-	-
BH08	0.0 – 1.0	-	-	-	-	-	-	-	5
BH09	3.0 – 3.45	-	-	-	51	30	13.0	-	-

Table 3-5 Summary of compaction and CBR test results, Alderley Bridge Crossing investigation

TP No.	Depth (mbgl)	Layer	MDD (t/m3)	OMC (%)	Placement Dry density Ratio (%)	CBR (%)
TP01	0.7 – 1.5	Sandy Clay / Sandy Silt	1.79	13.0	97.0	4.5#
TP04	0.15 – 1.5	Clay with Sand	1.72	15.0	96.5	2.5
TP05	1.0 – 1.5	Clay with Sand	1.74	16.5	97	1.0
TP07	0.4 – 1.5	Clayey Sand with gravel	1.98	10.5	97	12.0#
TP08	0.7 – 1.5	Clay with Sand	1.73	17.0	98	4.0#

Note : #CBR value at 2.5 mm penetration; 4 day soaking

Table 3-6 Soil aggressivity test results, Alderley Bridge Crossing investigation

BH No.	Depth (mbgl)	Layer	pH	Chlorides (mg/kg)	Sulphates SO_4 (mg/kg)
BH02	2.00 – 2.45	Gravelly/ Clayey Sand	9.2	80	70
BH05	6.00 – 6.45	Mudstone	8.4	<10	100
BH09	10.00 – 10.45	Mudstone	8.6	90	60

Table 3-7 Summary of rock Point Load test results, Alderley Bridge Crossing investigation

BH No.	Depth (mbgl)	Is_{50} (MPa)	Orientation	BH No.	Depth (mbgl)	Is_{50} (MPa)	Orientation
BH02	12.50	0.24	D	BH06	13.40	0.037	D
	12.59	1.30	A		13.90	0.45	A
BH03	12.90	0.12	D	BH07	9.50	0.14	D
	13.10	0.61	A		9.60	1.10	A
BH04	11.78	0.06	D	BH08	9.74	0.20	D
	11.94	0.47	A		9.91	1.40	A
	12.88	0.15	D	BH09	13.60	0.27	D
	13.26	0.77	A		13.75	1.40	A
BH05	16.90	0.12	D	-	-	-	-
	17.25	0.49	A	-	-	-	-

Note : A = Axial, D = Diametral

The main conclusions from the Alderley Bridge Crossing investigations were :-

- Approximately 4-7m of alluvial sediments comprising interbedded clayey medium dense sand and/or gravel deposit overlying a thinner stiff to very stiff high plasticity clay which in places could be considered a Residual Soil; noting that in certain boreholes within the Flinders River channel a gravel layer marks the base of the alluvium; in general the alluvium is thickest on the south side of the river, reaches 4m thick in the middle of the river and maintains this thickness typically to the north bank;

- A proven depth below Residual Soils/Alluvial sediments of at least 8-11m of Wallumbia Formation massive mudstones ranging from extremely weathered to fresh of reported strengths very low to high strength;

- No groundwater level conclusions could be made due to the drilling techniques employed;

- The laboratory testing undertaken on recovered soil samples (interpreted to be mainly of an alluvial origin) suggested the majority of deposits were either low plasticity clays or silts, with some outliers of high plasticity clay, having a high linear shrinkage, non-dispersive character, OMC's greater than natural moisture contents, and non-aggressive to buried concrete or reinforcement; and

- Point load tests indicated not an unsurprising anisotropic behaviour with a significant less diametral strength; axial test results indicating generally low to medium strength, and occasionally high strength at depth.

3.2 Hughenden Recreational Lake

The Hughenden Recreational Lake project was a Flinders Shire Council project, with the design/construction supervised by Premise Engineering. Douglas Partners were commissioned by Premise to undertake a geotechnical investigation of the proposed development, which consisted of the following main components :-

1. A 20-ha area lake;

2. 438 m long clay core with a minimum 3m deep cut-off trench at its base;
3. A 6.55m deep pumping station; and
4. A 580m long water main.

Douglas's investigation consisted of 31 excavated test pits and one shallow borehole (8.88m) drilled for the proposed pumping station. The published geology of the site indicated that for the most part it was underlain by mudstones of the Wallumbilla Formation Ranmoor Member (the same formation as for the HIPCO project site) with potentially minor sandstone interbeds. The investigations revealed that the proposed development footprint consisted of upper silty or sandy CLAY (probably Residual Soils or Extremely Weathered rock) overlying weathered mudstones. From recovered soil and weathered mudstone samples a suite of soil index, laboratory permeability, soil dispersion, chemical and compaction laboratory testing was undertaken. This investigation, along with engineering recommendations for the lake development, were submitted under Douglas Partner Report No. 93897.00_R.001.Rev0, dated 1 August 2018.

This initial report found that all samples tested were generally high plasticity clays, with very high linear shrinkage, being highly sodic and prone to dispersion. Recompacted clays were very impermeable (8×10^{-9} to 9×10^{-10} m/s) with recompacted mudstone samples of low to very low permeability (3×10^{-6} to 2×10^{-9} m/s). It was recommended that the mudstones should not be used as embankment fill.

Summaries of the Douglas Partners sampling and associated laboratory testing are provided in Table 3-108 & Table 3-9, while Atterberg Limits and PSD plots presented in Figure 3-4 & Figure 3-5 respectively.

A subsequent report by Douglas (Report No. 93897.00_R.002.Rev0, dated 9 November 2018) undertook a second suite of laboratory testing with gypsum additives, presumably to try and improve the inherent natural soils and weathered mudstones for use as embankment fill or for lake liner materials. A series of soil index, dispersion and chemical tests was performed on clays and mudstones samples from within the proposed lake footprint with 0, 2, 4 & 6% gypsum added. The gypsum addition testing revealed no significant improvement of the high sodic nature of the samples tested, with the following major summaries :-

- The majority of test revealed PD1 or PD2 pin hole results – indicating potential dispersive materials
- Electrical conductivity test results with one exception to be between 2200-5800 μS/cm – indicating dispersive/sodic characteristics
- Exchangeable Sodium Potential % results between 1 and 43.1, but with the vast majority >8 and again indicating generally extreme sodic potential.

Douglas's general summary of the findings of this second report was that even with up to 6% gypsum additives the clays and weathered mudstones were interpreted to be still susceptible to dispersive erosion under fresh water flow. With this in mind they suggested that the application of polymer based sprayed seals (Geo-R-Bond) to act as a waterproofing membrane should be considered.

Discussions with the actual earthworks contractors (Cheshire Contractors Pty Ltd, of Cairns) who undertook the construction of the lake, indicated the following salient points :-

- The liner from the lake was sourced from reworked shale/mudstone from the excavation;
- Seams within the shale rockmass were found to be very sodic/dispersive when immersed in water;
- Construction of the liner required constant moisture conditions for compaction purposes;
- Other areas were investigated for clay liner sources, but nothing else was found to be suitable; and
- The ground was seemingly very acidic in nature.

Table 3-8: Laboratory soil index test results Hughenden Recreational Lake, Douglas Partners, 2018

Location	Depth (mbgl)	Material Type	Particle Size Distribution mm (% Passing)															LL (%)	PL (%)	PI (%)	LS (%)	Emerson	MMD (t/m^3)	OMC (%)	MC (%)	K (m/s)
			53	37.5	26.5	19	13.2	9.5	6.7	4.75	2.36	1.18	0.6	0.425	0.3	0.15	0.075									
During Investigation																										
Town Burrow	Surface	Sandy gravelly CLAY	100	89	72	55	55	55	54	51	48	44	41	40	39	35	32	43	19	24	12		1.73	16	5.8	
Pit 1	1.2	Silty CLAY																65	25	40	20	4				
Pit 1	2.5					100	98	90	77	65	45	36	29	27	24	21	19	62	27	35	14.5	4	1.62	22		2 x E-09
Pit 2	1.2	Silty CLAY																58	24	34	12	4				
Pit 3	1.5					100	100	100	100	99	98	93	87	79	75	64	53	51	23	28	14	4	1.65	19.5		2 x E-09
Pit 3	3.2	SAND				100	99	99	98	98	95	81	59	51	48	45	44									
Pit 4	1.0	Silty CLAY																51	26	25	13	4				
Pit 4	2.1	Sandy CLAY				100	96	93	91	89	82	69	59	55	51	44	49	53	19	34	13.5	4				
Pit 5	1.4					100	100	100	100	100	98	95	92	90	87	80	73	65	24	41	16.5	4	1.53	23.5		1 x E-09
Pit 7	0.8	Silty CLAY																52	27	25	12.5	4				
Pit 8	1.3	Silty CLAY				100	95	93	89	83	59	30	10	5	4	4	4									
Pit 10	0.4					100	100	100	100	99	98	94	88	83	78	66	59	49	20	29	13.5	4	1.58	21		4 x E-09
Pit 11	2.8					100	82	70	58	48	33	25	21	19	18	16	14	57	29	28	13	4	1.63	22.5		2 x E-09
Pit 12	2.6					100	99	99	98	98	94	88	82	79	75	69	63	61	26	25	13	4	1.57	18		8 x E-09
Pit 13	0.5	Silty CLAY																51	20	31	13.5	4				
Pit 15	1.1	Silty CLAY																62	27	35	15	4				
Pit 17	0.7	Silty CLAY																60	30	30	16	4				
Pit 18	1.2	Silty CLAY																68	31	37	16	4				

Location	Depth (mbgl)	Material Type	Particle Size Distribution mm (% Passing)															LL (%)	PL (%)	PI (%)	LS (%)	Emerson	MMD (t/m^3)	OMC (%)	MC (%)	K (m/s)
			53	37.5	26.5	19	13.2	9.5	6.7	4.75	2.36	1.18	0.6	0.425	0.3	0.15	0.075									
Pit 19	1.5	Silty CLAY																59	31	28	15	4				
Pit 19	3					100	72	60	46	34	22	15	11	9	7	4	3	54	26	28	12	4	1.67	21		1 x E-08
Pit 20	1.2	Silty CLAY																62	27	35	15	4				
Pit 22	1.3	Silty CLAY				100	100	100	100	100	96	91	86	82	77	66	60	68	30	38	20.5	4	1.51	25		9 x E-09
Pit 23	1.6	Silty CLAY																59	26	33	13.5	4				
Pit 25	1.6					100	81	71	60	52	39	31	27	25	22	19	18	59	26	33	16	4	1.62	21.5		3 x E-06
Pit 26	0.9	Silty CLAY																68	33	35	17.5	4				
Pit 27	0.5	Silty CLAY				100	100	100	100	100	99	97	93	90	87	80	75	65	26	39	13.5	4	1.42	24.5		1 x E-09
Pit 30	0.9	Silty CLAY																67	31	36	17.5	4				
WB1	1	Silty CLAY																							18	
WB2	1.5	Silty CLAY																							13	
WB3	2	Silty CLAY										Denotes troublesome for a potential fill or dam clay core													11	
NWB1	1	Sandy CLAY																							19	
Mud	0.5	VLS Mudstone																							7	

Table 3-9: Laboratory soil chemical test results Hughenden Recreational Lake, Douglas Partners, 2018

Location	Depth (mbgl)	Material Type	Pinhole	pH	EC (µS/cm)	Sulphate (mg/kg)	ESP (%)	CEC (%)	SAR (%)	Chloride (mg/kg)	Exchangeable Ca	Exchangeable K	Ca:Mg Ratio	Exchangeable Mg
During Investigation														
Town Burrow	Surface	Sandy gravelly CLAY		8	2790		19.7							
Pit 1	1.2	Silty CLAY		7.9	1900	1900	13.2							

Location	Depth (mbgl)	Material Type	Pinhole	pH	EC (µS/cm)	Sulphate (mg/kg)	ESP (%)	CEC (%)	SAR (%)	Chloride (mg/kg)	Exchangeable Ca	Exchangeable K	Ca:Mg Ratio	Exchangeable Mg
Pit 1	2.5			8.1	1500	2700	12.1							
Pit 2	1.2	Silty CLAY		8.6	610	580	8.5							
Pit 3	1.5			8.8	750	380	11.4							
Pit 3	3.2	SAND												
Pit 4	1.0	Silty CLAY		8.7	210	64	4.6							
Pit 4	2.1	Sandy CLAY		8.7	790	400	19.1							
Pit 5	1.4			7.9	390	11,000	13.7							
Pit 7	0.8	Silty CLAY		8.6	590	810	8.1							
Pit 8	1.3	Silty CLAY												
Pit 10	0.4			8.7	150	30	1.6							
Pit 11	2.8			8.6	1100	1000	20.9				Denotes troublesome for a potential fill or dam clay core – sodic behaviour			
Pit 12	2.6			8.2	650	1200	8.6							
Pit 13	0.5	Silty CLAY		8.8	400	82	7.3							
Pit 15	1.1	Silty CLAY		8	2500	6000	9.9							
Pit 17	0.7	Silty CLAY		8	2800	5900	15.3							
Pit 18	1.2	Silty CLAY		7.8	4200	1200	10.6							
Pit 19	1.5	Silty CLAY		8.3	1100	1600	8.2							
Pit 19	3			7.8	2700	7900	10.7							
Pit 20	1.2	Silty CLAY		8.1	2600	6600	12.7							
Pit 22	1.3	Silty CLAY		8	3600	7500	16.9							

Location	Depth (mbgl)	Material Type	Pinhole	pH	EC (µS/cm)	Sulphate (mg/kg)	ESP (%)	CEC (%)	SAR (%)	Chloride (mg/kg)	Exchangeable Ca	Exchangeable K	Ca:Mg Ratio	Exchangeable Mg
Pit 23	1.6	Silty CLAY		8	3200	11,000	5.6							
Pit 25	1.6			7.7	2500	8000	3.4							
Pit 26	0.9	Silty CLAY		7.9	3500	10,000	11							
Pit 27	0.5	Silty CLAY		8.8	220	93	3.4							
Pit 30	0.9	Silty CLAY		8.6	190	730	25.4							
WB1	1	Silty CLAY	ND1	7.9	1030	1380	8.6	32.4	9.25	670	25.8	0.7	8.4	3.1
WB2	1.5	Silty CLAY	PD2	8.2	894	1100	8.7	22.6	9.38	520	17.9	0.4	8.1	2.2
WB3	2	Silty CLAY	PD2	8.1	1960	4200	20.4	22.3	12.6	680	14.8	0.8	7	2.1
NWB1	1	Sandy CLAY	PD2	7.9	4200	37,900	24.4	30.5	12.5	2650	20.6	0.6	11.6	1.8
Mud	0.5	VLS Mudstone	PD1	7.9	4240	20,000	43.1	29.1	13.2	470	19.4	0.6	3.2	3.8
During Construction														
101		Sandy CLAY		7.6	4300		13.3	80	1.8		64	1.4		
102		Sandy CLAY		7.8	4900		17.5	84	2.8		65	1.3		3.6
103		VLS Mudstone		7.5	4200		31.4	64	4.4		37	1.2		2.7
104		Sandy CLAY		7.6	4000		12.8	77	1.7		60	5.2		6.6
Pit 7		Silty CLAY		8.6	320		5.8	47	0.6		40	1.1		5.2
Pit 7		Silty CLAY	ND1	8.6	590		8.1							3.3
Pit 4		Sandy CLAY	PD2	8.6	450		6.3	27	0.5		22	0.83		2.3
Pit 22		Sandy CLAY	ND1	8	4000		3.1	70	0.4		65	1.3		2.1
Pit 27		VLS Mudstone	ND2	7.8	2500		1	54	0.1		51	1.2		0.93

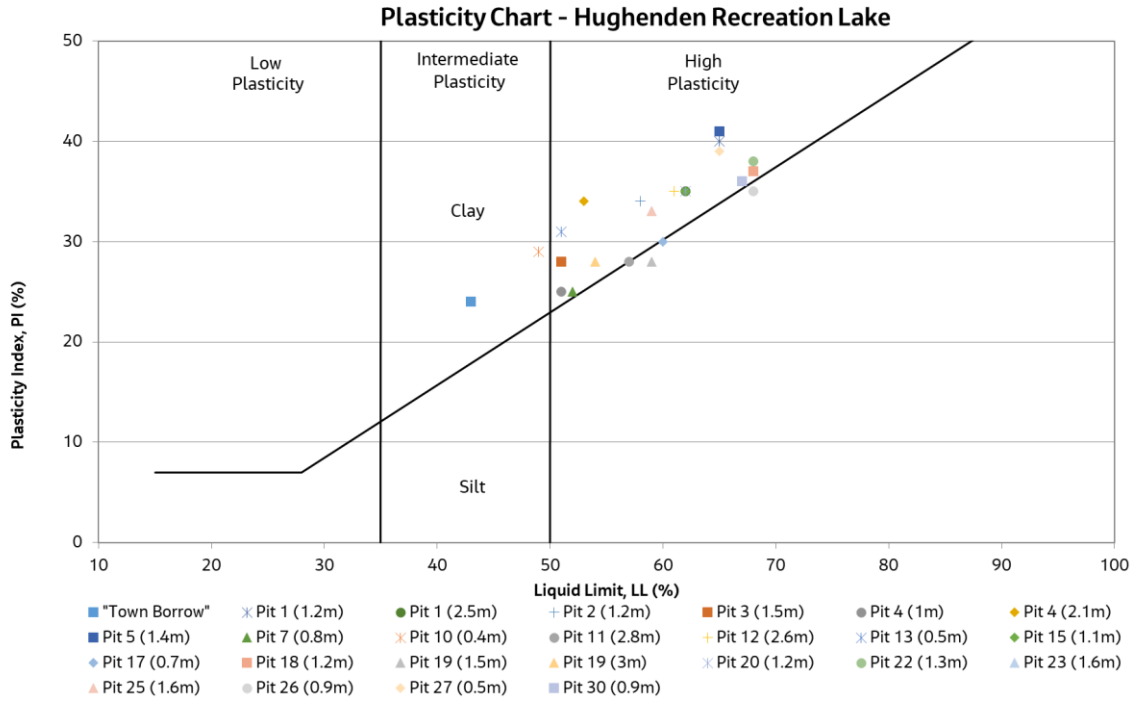


Figure 3-2: Hughenden Recreational Lake Atterberg Limits plot

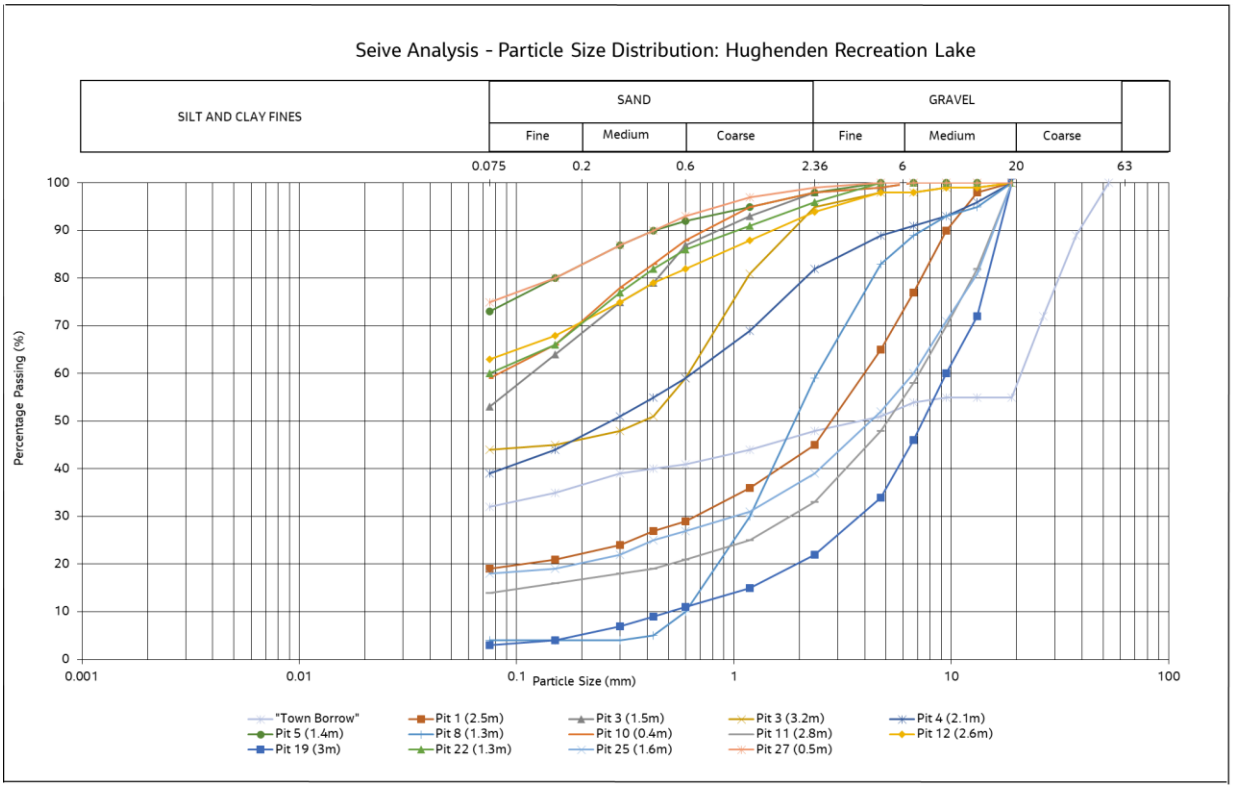


Figure 3-3: Hughenden Recreational Lake Particle Size Distributions

3.3 HIPCo Preliminary Business Case

During the Preliminary Business Case for the Hughenden Irrigation Project, Engeny commissioned Construction Sciences to undertake a Geotechnical Investigation for the reference project site, Saego Dam. These are reported in "Geotechnical Investigation – Factual Reporting" reference 10599E/P/106 (15 January 2020). The investigation included 24 boreholes drilled to maximum depth of 15.87mbgl and 11 test pits to maximum depth of 4.7mbgl. The investigation locations are shown in Appendix A. Details of the locations and drilling methods are provided in Table 3-10.

Table 3-10: Preliminary Business Case geotechnical investigation locations (Construction Sciences, 2020)

Investigation ID	Location		Investigation Method Depths (mbgl)				
	Easting (m)	Northing (m)	Solid Auger	Washbore	Air Hammer	NMLC Coring	Backhoe
BH1	798530	7715072	0.0-3.0	3.0-8.44			
BH2	798157	7715258	0.0-3.0	3.0-11.07			
BH3	797688	7715427	0.0-3.0	3.0-11.64			
BH4	797113	7715594	0.0-2.0		2.0-15.87		
BH5	794477	7715232			0.0-9.06		
BH6	792073	7714606			0.0-11.76		
BH7	796753	7715775			0.0-5.56		
BH8	796284	7715956			0.0-5.69		
BH9	794942	7715407			0.0-9.00		
BH10	793534	7714891			0.0-15.69		
BH11	793047	7714779			0.0-15.76		
BH12	793080	7714612			0.0-9.50		
BH13	792560	7714667			0.0-9.71		
BH14	791568	7714622			0.0-15.86		
BH15	791168	7714637		0.0-11.5	11.5-18.45		
BH16	790769	7714653			0.0-12.86		
BH17	794006	7715052			0.0.-7.20		
BH18	794504	7715003			0.0-9.60		
BH19	795341	7715656			0.0-3.60		
BH20	795828	7715863			0.0-6.50		
BH21	794410	7715454			0.0-6.45		
BH22	795477	7715638	0.0-1.1			1.1-12.90	
BH23	796189	7715985	0.0-3.0	3.0-6.45		6.45-12.00	
BH24	794779	7715641			0.0-6.60		
TP101	795180	7715871					0.0-4.7
TP102	790769	7714653					0.0-3.7
TP103	791568	7714622					0.0-3.0
TP104	792073	7714606					0.0-4.2

Investigation ID	Location		Investigation Method Depths (mbgl)				
	Easting (m)	Northing (m)	Solid Auger	Washbore	Air Hammer	NMLC Coring	Backhoe
TP105	793047	7714779					0.0-3.7
TP106	794006	7715052					0.0-3.5
TP107	794942	7715407					0.0-3.7
TP108	795828	7715863					0.0-3.2
TP109	796753	7715775					0.0-3.8
TP110	797688	7715427					0.0-3.5
TP111	798335	7715059					0.0-3.8

Due to the drilling methods utilised in this investigation, minimal in-situ testing was carried out. The material descriptions and depths encountered are likely to only be indicative, due to the air hammer method of drilling employed in the majority of the boreholes.

Samples were recovered from the test pits for laboratory testing, including:

- Particle size distribution (refer Figure 3-4)
- Atterberg limits (refer Figure 3-5)
- Linear shrinkage (refer Table 3-11)
- Emerson class (refer Table 3-11)
- Maximum Dry Density (refer Table 3-11)
- Optimum Moisture Content (refer Table 3-11)

The indicative interpreted geological long-section profile of the Saego Dam embankment is reproduced in Figure 3-6 (Engeny, 2020). This shows a varying thickness of alluvial and residual material (clays, sands and gravels) down to highly weathered rock. The inferred rock level is shallowest near the proposed spillway, at a depth of 3m below existing ground level. At the deepest point (BH15), the inferred rock level is 19m below existing surface level. These rock levels have not been confirmed, and levels are indicative only, due to the air hammer method of drilling. The rock has been logged as extremely weathered, very low strength Mudstone. The mudstone generally increases in strength and becomes less weathered with depth, with moderately to slightly weathered, low strength mudstone encountered at four (4) locations.

Table 3-11: PBC geotechnical investigations laboratory test results summary

Sample Location	Origin	Linear Shrinkage (%)	Emerson Class	Maximum Dry Density (t/m3)	Optimum Moisture Content (%)
TP101 (0.2 - 1 m)	Alluvium	16.	0 4		
TP101 (3.6 - 4.7 m)	XW	6.5	2 - dispersive		
TP102 (0.2 - 1 m)	Alluvium	17.5	4		
TP102 (1.6 - 2 m)	Alluvium	17.5	4	1.59	19.5
TP103 (0.1 - 0.5 m)	Alluvium	17.0	4	1.51	25.5
TP103 (0.6 - 1 m)	Alluvium	19.5	4		
TP103 (2 - 2.5 m)	Alluvium	20.0	4		

Sample Location	Origin	Linear Shrinkage (%)	Emerson Class	Maximum Dry Density (t/m3)	Optimum Moisture Content (%)
TP104 (0.8 - 1.2 m)	Alluvium	12.5	2 - dispersive		
TP105 (1 - 1.5 m)	Alluvium	18.5	4		
TP105 (3.2 - 3.6 m)	Residual	21.5	2 - dispersive		
TP106 (0.2 - 1 m)	Alluvium	14.0	4		
TP106 (1.1 - 1.5 m)	Alluvium	13.5	-		
TP107 (1.6 - 1.9 m)	XW	16.0	2 - dispersive	1.72	17.5
TP108 (1 - 1.3 m)	Alluvium	16.0	2 - dispersive		
TP108 (1.6 - 2 m)	XW	15.0	2 - dispersive	1.77	16.5
TP109 (0.2 - 0.6 m)	Alluvium	17.5		4	1.38
TP109 (1.7 - 2.4 m)	Alluvium	19.5	4	1.63	21.5
TP109 (2.9 - 3.2 m)	Residual	16.0	2 - dispersive		
TP110 (0.9 - 1.1 m)	Alluvium	18.0	2 - dispersive	1.48	23.5
TP111 (0.1 - 0.4 m)	Alluvium	13.5	3 - dispersive		

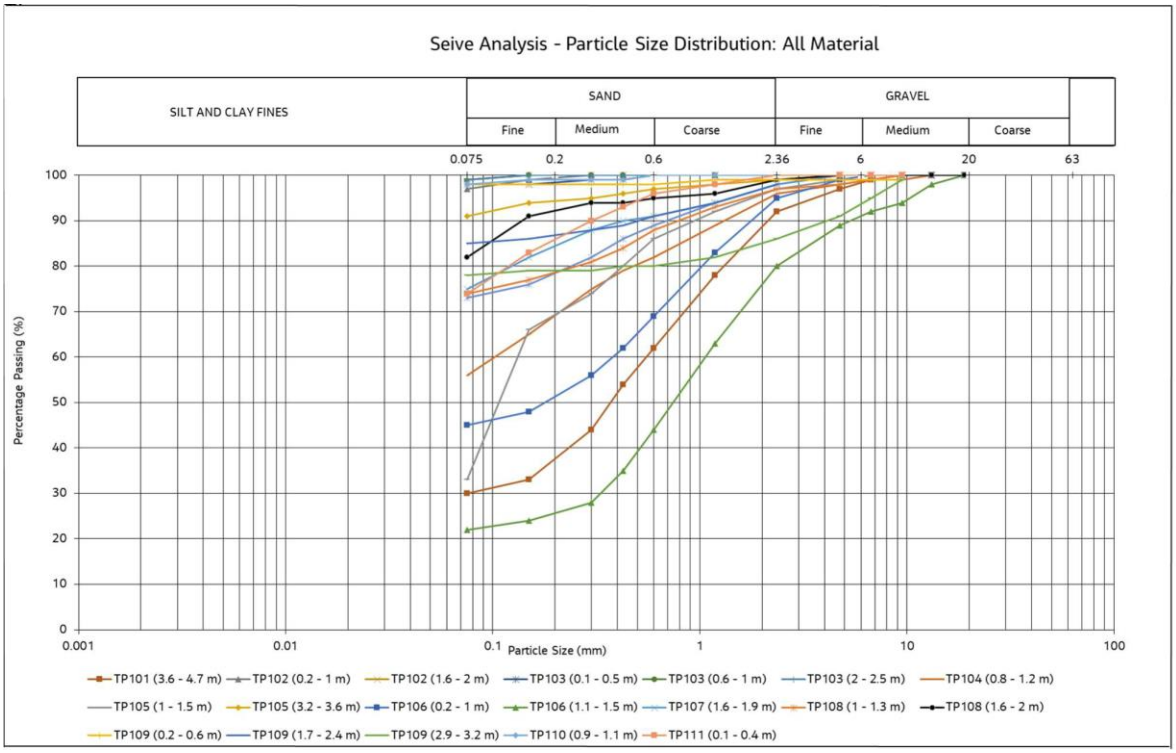


Figure 3-4: PBC geotechnical investigations Particle Size Distribution

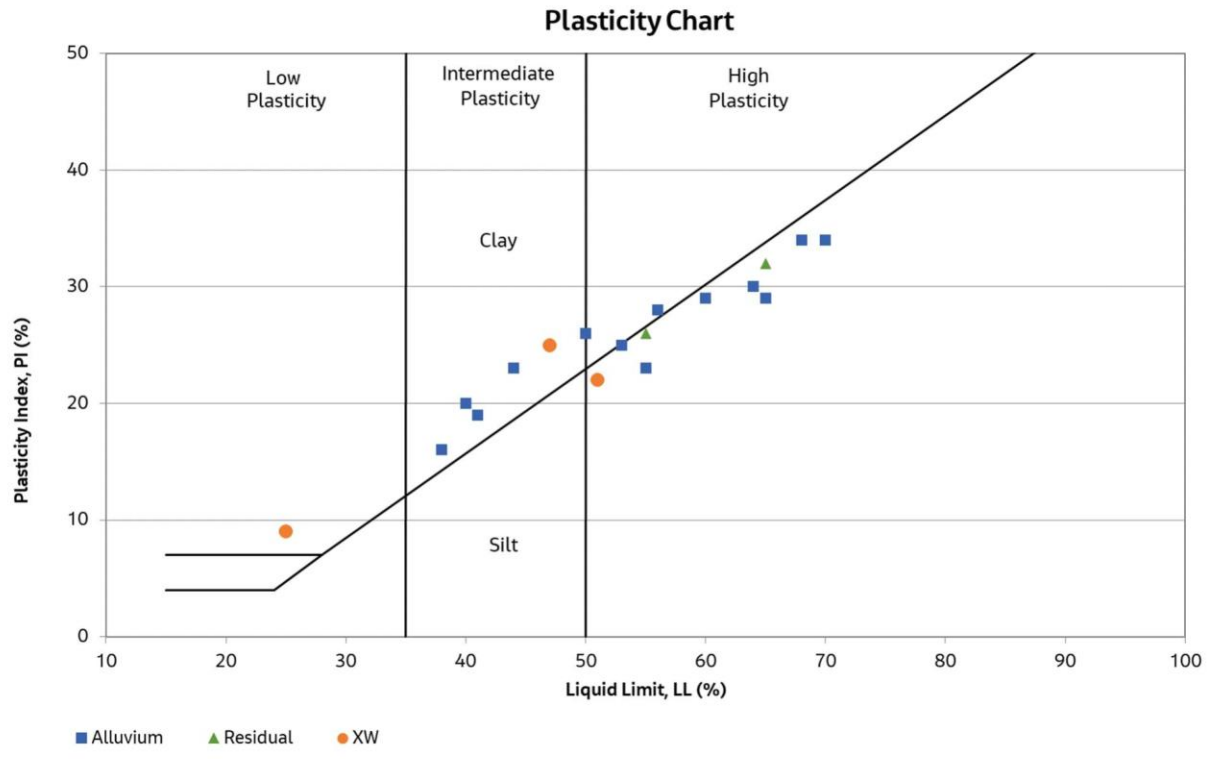
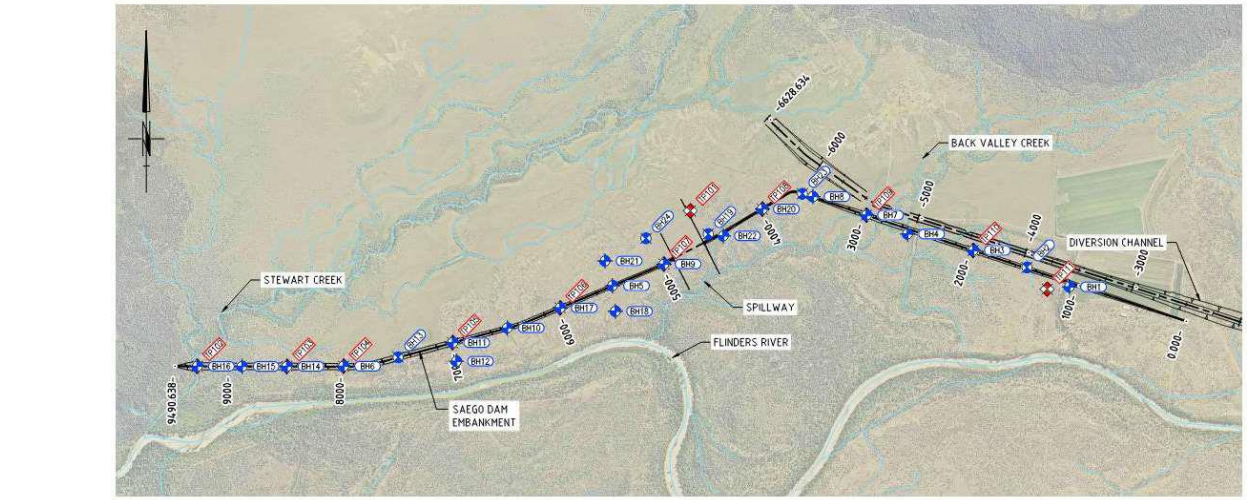


Figure 3-5: PBC geotechnical investigations Atterberg Limits plot



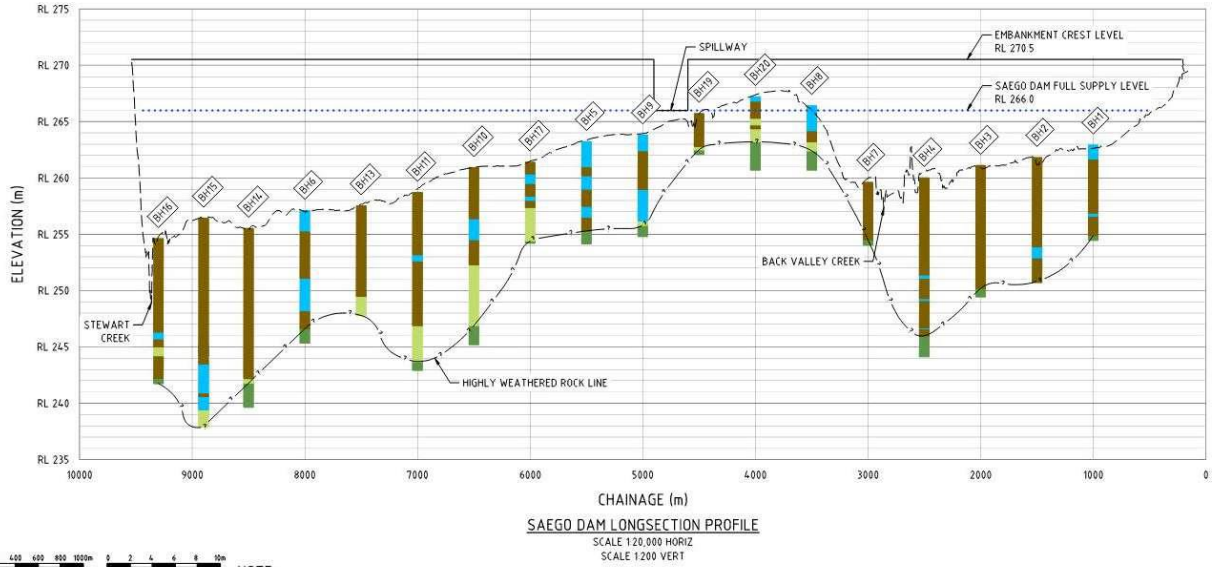
SAEGO DAM PLAN
SCALE 1:20,000

TEST PIT SETOUT POINTS

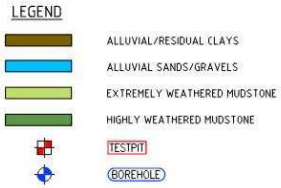
POINT No.	COORDINATES	
	EASTING	NORTHING
TP101	169891.298	7715225.827
TP102	165513.930	7713845.190
TP103	166308.996	7713843.677
TP104	166811.509	7713846.314
TP105	167780.712	7714055.259
TP106	168734.990	7714363.650
TP107	169656.884	7714753.074
TP108	170529.532	7715241.659
TP109	171454.267	7715187.837
TP110	172402.439	7714874.539
TP111	173058.553	7714530.553

BOREHOLE SETOUT POINTS

POINT No.	COORDINATES	
	EASTING	NORTHING
BH1	173256.300	7714550.790
BH2	172878.045	7714722.946
BH3	172402.439	7714874.539
BH4	171819.338	7715020.198
BH5	169198.892	7714560.972
BH6	166811.509	7713846.314
BH7	171454.267	7715187.837
BH8	170978.660	7715351.430
BH9	169656.884	7714753.074
BH10	168265.314	7714185.231
BH11	167780.712	7714055.259
BH12	167813.550	7713889.476
BH13	167296.111	7713925.286
BH14	166308.996	7713843.677
BH15	165910.965	7713843.915
BH16	165513.930	7713845.190
BH17	168734.990	7714363.650
BH18	169225.485	7714332.965
BH19	170049.871	7715019.748
BH20	170529.532	7715241.659
BH21	169132.902	7714780.508
BH22	170183.822	7715003.750
BH23	170885.092	7715376.936
BH24	169496.341	7714981.079



SAEGO DAM LONGSECTION PROFILE
SCALE 1:20,000 HORIZ
SCALE 1:200 VERT



NOTE:

Figure 3-6: PBC Saego Dam interpreted geotechnical long section profile (Engeny 2020)

The PBC geotechnical investigation provided useful information on the potential depth to rockhead, the complexity of the alluvial sequence and the weathered rock profile lying above it has been provided. In addition, some idea of the characteristics of the subsoil sequence was also provided by the associated laboratory testing undertaken. A review of this investigation has indicated the following conclusions, centred not only on considerations for the PBC scheme, but also for possible required modifications to it :-

- The investigations were generally limited to the main Saego Dam embankment, a potential central Spillway location, and part of the proposed Diversion Channel only, and did not cover the other scheme elements or identified construction material source locations;

- Although a number of drillholes were advanced through the potential alluvial sequence, the use of a down-the-hole-hammer drilling techniques limited the information of the stratigraphy, composition and consistency of these deposits; in addition, few drillholes were advanced by coring into the Wallumbia Formation weathered rock sequences below to ascertain, with an acceptable degree of confidence, the level of competent rockhead (defined as moderately weathered medium strength rock or better) and rockmass characteristics, specifically discontinuity patterns and their infilling and morphology;

- No piezometers were installed in any boreholes, with the groundwater details on the presented borehole logs being inconclusive in determining perched water tables or steady state piezometric levels within the alluvial sequences & at depth;

- No permeability or packer testing was undertaken;

- Limited associated geotechnical laboratory testing was undertaken as an aid for design parameter definition, as well as for potential ground treatments, durability and construction material suitability;

- From the laboratory testing undertaken it could be interpreted that from the areas investigated that the alluvial soils in themselves may not be appropriate sources of clay core materials without blending or some form of additives; and

- The investigation identified a potential significant thickness of alluvium below the main embankment dam and proposed weir structure, which could contain low density/strength and compressible soil sequences.

4. Walkover Survey

An initial site observation assessment was undertaken between the 20th and the 21st of January 2021 through a rapid helicopter flyover of the Saego Dam site and a drive over of the Gap, Alstonvale and Canterbury Creek options. The findings of this assessment are reported in the HIPCO – Geotechnical Desk Study and Initial Site Findings Report (IS362200-GT-RPT-0001 | 01).

A second walkover survey of the proposed dam site and adjacent areas, along with an additional helicopter survey (in part to collect bulk samples for possible construction material sourcing) was undertaken between February 16 and 19 2021.

Additional site specific details relative to the proposed investigation locations was obtained during a walkover between 27 to 31 April 2021, which included cultural heritage clearance of these positions.

A fourth site assessment / walkover was undertaken prior the commencement of the drilling and test pitting operations, between the 11 and 14 May 2021. The aim of this site assessment was to peg updated investigation locations, as well as investigate the site access requirements for each of these locations. Additionally, a further appreciation of the geology and general geotechnical conditions for the HIP scheme and its main components, such as the Saego Dam, Diversion Channel, Flinders River Weir, and the irrigation area was performed.

The main findings of these pre-investigation walk and flyover surveys were as follows :-

- An appreciation of the inherent potential ground conditions that could be expected to be encountered, as well as their possible behavioural characteristics;
- Sample recovery followed by initial suite of laboratory testing to identify possible construction material source locations – to be used in part to define practical dam type options and their possible concept design form and dimensions;
- To explore and validate other potential scheme design options/layouts;
- To identify any significant project critical/red flag geotechnical risk issues that could significantly effect the location/configuration of scheme options being considered; and
- To finalise proposed geotechnical investigation locations, access to them including any required earthworks.

Of particular note during these different walkover surveys were the following :-

1. Bank instability and regressive erosion of the Flinders River banks was observed near both planned boreholes for the proposed siphon crossing (HDBH01 & HDBH02), as shown in Photograph 4-1 & 4-2.
2. Piping erosion and highly dispersive soils was observed along the Stewart Creek banks (near HCPT3), south of HSBH14 near the Flinders River abandoned channel (see Photograph 4-3 and Photograph 4-4).

Photograph 4-1: Flinders River bank instability near HDBH01

Photograph 4-2 : South bank of Flinders River

Photograph 4-3: Piping erosion on top of the Stewart Creek bank, near HCPT3

Photograph 4-4: Tunnel erosion south of HSBH14

5. Project Specific Geotechnical Investigations

5.1 Overview and Staging

Taking into consideration the findings of the previous PBC Construction Science 2020 investigations, along with the present design development of the project, specific geotechnical investigations were scheduled and undertaken covering the following main areas of interest:

- Investigation of the dam wall footprint in terms of its competency for bearing, stability and seepage character, liquefaction potential, as well as generally durability;
- Depth and nature of the underlying bedrock in terms of its weathering profile, discontinuities and potential of a major geological structural feature (e.g., fault) below the dam alignment,, as well as for Spillway & Weir stability assessments;
- Multiple investigations for potential construction materials of clay core, filter, rockfill and rip-rap for the proposed dam (but not including specifically for concrete aggregate);
- Excavatability and nature of soils and rock along the proposed weir and channel alignment; and
- Investigation of the proposed Distribution Network (for an appreciation of pipeline excavatability, pipe trench stability, and backfill suitability, along with potential pumping station foundation conditions) including the major siphon crossing of the Flinders River.

The geotechnical site investigations comprised a combination of intrusive (borehole drilling, test pits and CPT's) and non-intrusive techniques (geophysical surveys of MASW, ERI and Seismic Refraction profiling), paired with in-situ testing data (SPT's, DCP's, in-situ permeabilities) and associated laboratory testing.

The site investigations targeted the following areas:

- Saego Dam & potential dam wall alternative alignments
- Diversion Channel
- Flinders River Weir
- Alderley Road realignment
- Distribution Network

Prior to fieldwork commencing, Jacobs were required to obtain necessary access approvals from all property owners where investigation locations were sited. In addition, all intrusive site investigations and access tracks to them were also cleared by local cultural heritage group representatives, who were also present for all test pit and certain borehole investigations.

The geotechnical investigation was completed in general compliance with Australian Standard AS1726-2017 Geotechnical Site Investigations and Jacobs' standard work procedures.

Following the investigation, soil, rock and groundwater samples, collected as part of the investigation, were submitted for laboratory testing.

Laboratory testing was undertaken in a two-stage approach as further explained in Section 5.9.

Investigation location plans are included in Appendix A (see Figure IS362200_G013).

The design, location and type of proposed geotechnical investigations were developed and amended to reflect the desk study and walkover findings, as well as changes in the ultimate layout and components of the HIP scheme.

The geotechnical site investigation was undertaken in stages between late May and July 2021. An initial geophysical survey was carried out to inform the potential ground conditions that could be encountered and for final siting of the planned intrusive boreholes, test pits and CPT's. The intrusive investigations were undertaken under fulltime site supervision by geotechnical engineers/engineering geologists from Jacobs, who managed the field investigations, conducted in-situ testing, nominated sampling and testing intervals, undertook field logging and prepared borehole and test pit logs.

Initially geotechnical investigations were scheduled to be undertaken along the Distribution Network, however due to land access paired with COVID-19 lockdown delays, this part of the program had to be postponed to future stages of the project. A good appreciation of the potential ground conditions that could be encountered during the construction phase was gained from walkover surveys, as well as shallow test pitting within the proposed Irrigation Area undertaken as part of the agricultural suitability assessment (see Section 6.4).

5.2 Topographical Survey

An initial precise survey of the positions and ground/collar levels for the proposed geotechnical investigation scope was undertaken by RPS (Townsville) in late April 2021. Latterly the investigation scope/locations were amended to suit the developing scheme configuration.

During the investigation coordinates for each of the intrusive geotechnical site investigation locations were surveyed using a Samsung Tablet with Collector App GPS to a horizontal and vertical accuracy of up to ± 5 m.

The horizontal coordinates were recorded to the new GDA2020 MGA Zone 54 datum, with the vertical coordinates recorded to Australian Height Datum (AHD). The vertical coordinates were then compared and adjusted with the current Digital Terrain Model (DTM) directly derived from the recent project specific LiDAR (2021) survey.

CPT locations were surveyed using the IGS onboard high accuracy GNSS GPS on the CPT rig, while again vertical elevations were obtained from the project DTM for each test location.

The surveyed positions of the field investigation locations are presented on the site investigation location plans in Appendix A, on Table 5-3 & Table 5-9, and annotated on the respective engineering log sheets contained in Appendix E.

5.3 Geophysical Survey

Geophysical field surveys were carried out by Golder Associates (Golder) between the 10 - 25 of May 2021. These surveys comprised lines of Electrical Resistivity Imagery (ERI), Multichannel Analysis of Surface Waves (MASW) and Seismic Refraction (SR), along the following areas:

- Area 1 - Western Saego Dam section; approximately 4.10 km in length
- Area 2 – Original Saego Dam eastern alignment & the Diversion Channel; approximately 3.35 km in length
- Area 3 – The Flinders River Weir crossing; approximately 0.09 km in length (only comprised seismic profiling)

Details of the geophysical survey undertaken by Golder are included in Appendix C.

It should be note that initial processing was based on the historic geotechnical investigations but was subsequently amended when project specific intrusive investigations were completed and made available for correlation purposes.

5.3.1 Electrical Resistivity

The Electrical Resistivity Imaging (ERI) is a non-intrusive geophysical method used for imaging surfaces features using electrical measurements collected at the surface through electrodes emplaced in the ground. The method maps the variations in electrical properties of the subsurface materials, which can be as a consequence of the variations in the sediment or rock type, pore-water content, pore-water chemistry, and presence of buried debris. However, most of the current flow takes place through the material's pore-water. Clay minerals and some metallic minerals are also likely to be more conductive due to the availability of free ions.

The ERI present the ground resistivity data along the alignments, which when correlated to the borehole and CPT data, provides an assessment of the possible internal layering within the alluvium (as well as the determination of cohesive or granular dominated deposits), potential soil moisture/groundwater levels, and structures within the bedrock.

5.3.2 MASW and Seismic Refraction

The Multi-channel Analysis of Surface Waves (MASW) is a non-intrusive seismic method that uses the dispersive properties of surface waves for the purpose of shear wave velocity (Vs) profiling. The MASW measures variations in surface wave velocity with increasing distance and wavelength. It can be used to determine the rock /soil interface and soil conditions. The relationship between surface wave velocity and wavelength is used to determine the Vs profile of the material as the depth increases. The Vs is directly proportional to the small strain shear modulus, which is a direct indicator of the stiffness of the ground. Vs30 profiles obtained from these geophysical surveys were used for seismic hazard risk assessments.

Seismic Refraction (SR) is a further non-intrusive geophysical method that measures the velocity of compression waves (Vp) and provides a continuous 2D subsurface section. The SR is used very often in geotechnical investigations to determine the depth of the bedrock/ thickness of overburden, as well as it can be used to determine the rippability of the rock materials. Because the method provides a continuous 2D profile, it is used often in conjunction with Boreholes and Test Pits to correlate subsurface profile of materials.

The MASW and SR present the P-wave and S-wave velocity data along the survey alignments, which when correlated to the boreholes and CPT data, can be used to assess the variability in the depth of the alluvium layers, as well as the underlying bedrock.

5.4 Cone Penetration Testing

Cone Penetration Testing (CPTs) were carried out at selected locations by In-situ Geotech Services (IGS). The CPTs comprised a combination of Conventional CPTu and Seismic CPTu profiling. The CPTs were undertaken to augment both geophysical surveys and other intrusive geotechnical investigations. The conventional CPTs provide a continuous record of soil properties & profiles, which is extremely useful in a potential dynamic alluvial floodplain environment with changing facies over the Quaternary period. The seismic CPT's were undertaken to provide specific shear wave velocity profiling for again as an input into the seismic hazard risk assessment.

Testing was carried out using a 15t Truck Mounted CPT rig (Beryl) on a single mobilization between 16/06/2021 and the 19/06/2021 (see Photograph 5-1). A total of 19 CPTu and 6 Seismic CPTu were carried out along the Saego Dam main alignment and the north-west alternative alignment, Diversion Channel, either side the Flinders River Weir and the south-east trailing embankment (which has now been removed from the final scheme layout).

Photograph 5-1: CPTu rig at HCPT1

All CPTu and Seismic CPTu testing were carried out to refusal. Refusal was encountered at a shallow depth at HCPT1, and as a result the rig was moved forward approximately 2 metres and a second test was carried out. This second test is notated with an alphanumerical suffix added to the investigation notation i.e. “HCPT1a”.

Dissipation testing were not carried out at any locations within this site investigation campaign, as no significant zones of compressible cohesively dominated soils were encountered.

CPTu and Seismic CPTu test results are provided in Appendix D.

5.4.1 Conventional CPTu

Conventional continuous electrical CPTu testing was carried out at the locations presented in Appendix A and in Table 5-1.

Table 5-1: CPTu location summary table

CPTu ID	Location	Easting (m)	Northing (m)	Elevation (mAHD)	Overall Pushed Depth (mbgl)
HCCPT01	Diversion Channel	798829	7714956	264	7.09
HCCPT02	Diversion Channel	798236	7715361	261.3	11.06
HCCPT03	Diversion Channel	797444	7715495	260.91	9.88
HCCPT04	Diversion Channel	797079	7715760	258	10.41
HCPT1	Saego Plains (NW Dam Wall Option)	792842	7715123	256.8	7.28
HCPT1a	Saego Plains (NW Dam Wall Option)	792838	7715120	256.8	6.22
HCPT2	Saego Plains (NW Dam Wall Option)	792394	7715181	255	13.37
HCPT3	Saego Plains (NW Dam Wall Option)	791882	7715268	256.5	12.48
HCPT4	Saego Plains (NW Dam Wall Option)	791397	7715513	255.2	14.89

CPTu ID	Location	Easting (m)	Northing (m)	Elevation (mAHD)	Overall Pushed Depth (mbgl)
HCPT5	Saego Plains (NW Dam Wall Option)	790904	7715684	254.3	16.34
HSCPT01	Saego Dam	790800	7714646	254.3	22.31
HSCPT02	Saego Dam	791103	7714622	256	19.44
HSCPT03	Saego Dam	792267	7714648	256.8	5.95
HSCPT04	Saego Dam	793119	7714826	259	5.04
HSCPT05	Saego Dam	793484	7714913	260.5	11.6
HSCPT06	Saego Dam	794196	7715145	262.1	8.33
HWCPT01	Flinders River Left Bank	801875	7712414	266.4	16.94
HWCPT02	Flinders River Left Bank	801898	7711800	266.4	14.54
HWCPT03	Flinders River Weir	800844	7713612	265	15.92
HWCPT04	Flinders River Weir	800888	7713724	263.5	8.68

5.4.2 Seismic CPTu

Sesimic CPTu were carried out at 6 selected locations as shown in Appendix A and Table 5-2 below to assist in the ground model of the dam foundations, as well as the liquefaction potential assessment. The seismic cones were pushed directly adjacent to the associated conventional CPT's carried out at the same location.

Table 5-2: Seismic CPTu Location Summary Table

Seismic CPTu ID	Location	Easting (m)	Northing (m)	Elevation (mAHD)	Overall Pushed Depth (mbgl)
HCCPT02s	Diversion Channel	798236	7715361	261.3	10.5
HCCPT04s	Diversion Channel	797079	7715760	258	10.5
HCPT2s	Saego Plains (NW Dam Wall Option)	792394	7715181	255	14
HSCPT01s	Saego Dam	790800	7714646	254.3	18.5
HSCPT03s	Saego Dam	792267	7714648	256.8	6
HWCPT03s	Flinders River Weir	800844	7713612	265	15

5.4.3 Cone Penetration Testing Data Interpretation

CPTu raw data as provided by IGS was exported and processed by using a commercially available software package CPeT-iT (v.3.3.2.17) from GeoLogismiki.

CPeT-IT takes Cone Penetration Test raw data and performs basic interpretation in terms of Soil Behaviour Type (SBT) and various other geotechnical parameters using the current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson.

The results of the CPTu interpretations are presented in Appendix H.

5.5 Test Pits

Test Pitting was carried out along the Dam, channel and road alignments to gain an understanding of the shallow ground conditions and to determine if suitable construction borrow material can be sourced.

The test pits were excavated using a CAT 308E2 CR Excavator provided by SJ and SC Reddie Earthmoving, as shown in Photograph 5-2. The same excavator, along with a local farmer's grader, was used to provide suitable access to various geotechnical investigation locations.

Photograph 5-2: 8t excavator used for test pitting

The test pits were generally excavated at a width of 0.7 to 1.4m and a length of 2.6 to 4.0 m to the target depth of 3 m or refusal of the machine. Pocket Penetrometer (PP) tests were undertaken in the fine-grained soils encountered, with the results presented on the machine test pit logs. In general, large bulk disturbed samples, with associated small bag samples were taken in every test pit in generally different stratigraphic units encountered. A summary of the test pit investigations is provided in Table 5-3.

Table 5-3: Summary of test pits investigation

Test Pit ID	Location	Easting (m)	Northing (m)	Elevation (mAHD)	Excavated Depth (mbgl)
HCTP01	Diversion Channel	801281	7713788	267	3
HCTP02	Diversion Channel	800765	7714455	271	3.15
HCTP03	Diversion Channel	799975	7714712	265	3.1
HCTP04	Diversion Channel	796867	7715866	210	3.35
HCTP05	Diversion Channel	796582	7716055	213	3
HCTP06	Diversion Channel	796316	7716265	234	3.05
HCTP07	Diversion Channel	796021	7716549	241	3.1
HDTP01	Distribution Network	790807	7714378	264	3.1
HRTP01	New Road Alignment	801051	7713905	259	3.2
HRTP02	New Road Alignment	800977	7714084	250	3
HRTP03	New Road Alignment	801014	7714152	241	3.5
HRTP04	New Road Alignment	801344	7714348	248	3

Test Pit ID	Location	Easting (m)	Northing (m)	Elevation (mAHD)	Excavated Depth (mbgl)
HRTP05	New Road Alignment	801616	7714435	253	3.4
HSTP01	Saego Dam Alignment	794993	7715459	305	3
HSTP02	Saego Dam Alignment	795240	7715559	300	3.45
HSTP03	Saego Dam Alignment	795586	7715695	285	2.85
HSTP04	Saego Dam Alignment	796446	7715851	210	3
HSTP05	Saego Dam Alignment	796092	7716057	220	3.3
HSTP06	Saego Dam Alignment	796588	7716376	241	3.15
HSTP07	Saego Dam Alignment	796902	7716609	253	2.9
HSTP08	Saego Dam Alignment	797163	7716862	253	3.4
HSTP09	Saego Dam Alignment	797063	7717232	244	3.2
HWTP01	Weir	801662	7712912	253	3.5
HWTP02	Weir	802195	7711126	245	3.1

5.6 Boreholes and In-situ Testing

Boreholes were located along the proposed Saego Dam alignment, Flinders River Weir, Diversion Channel, and both sides of the proposed Siphon Flinders River Crossing. Table 5-4 below provides a listing of the borehole locations and drilled depths, as well as the termination criteria for each borehole.

Table 5-4: Summary of borehole investigation locations

Borehole ID	Location	Easting (m)	Northing (m)	Elevation (mAHD)	Depth (mbgl)	Termination Criteria
HCBH01	Diversion Channel	800988	7714354	272.1	15.00	15m Target Depth
HCBH02	Diversion Channel	800546	7714515	269.5	12.10	15m Target Depth
HCBH03	Diversion Channel	800262	7714627	273.6	16.05	15m Target Depth
HCBH04	Diversion Channel	799763	7714808	271.0	12.00	15m Target Depth
HDBH01	Distribution Network – Siphon Crossing	790910	7714179	256.0	20.30	Prove Base of Alluvium
HDBH02	Distribution Network – Siphon Crossing	791010	7714006	257.1	25.29	Prove Base of Alluvium
HRBH01	Road	801122	7714205	269.1	11.88	Prove Base of Alluvium
HSBH02	Saego Dam Alignment	791445	7714660	255.6	20.10	5m Competent Rock
HSBH03	Saego Dam Alignment	791944	7714643	256.9	18.50	5m Competent Rock
HSBH04	Saego Dam Alignment	792738	7714735	257.8	10.70	5m Competent Rock
HSBH05	Saego Dam Alignment	793794	7714993	261.1	15.90	5m Competent Rock

Borehole ID	Location	Easting (m)	Northing (m)	Elevation (mAHD)	Depth (mbgl)	Termination Criteria
HSBH06	Saego Dam Alignment	794448	7715239	263.2	13.90	5m Competent Rock
HSBH07	Saego Dam Alignment/Spillway	795179	7715757	265.7	20.70	7m Competent Rock
HSBH08	Saego Dam Alignment/Spillway	795265	7715365	259.7	12.00	5m Competent Rock
HSBH09	Saego Dam Alignment	795880	7715833	267.0	15.00	5m Competent Rock
HSBH10	Saego Dam Alignment/Spillway	797125	7715645	260.5	15.24	Prove Base of Alluvium
HSBH11	Saego Dam Alignment	797726	7715375	260.9	14.55	Prove Base of Alluvium
HSBH12	Saego Dam Alignment	798454	7715126	262.4	7.41	Prove Base of Alluvium
HSBH13	Saego Dam Alignment	799268	7714829	266.4	4.29	Prove Base of Alluvium
HSBH14	Spillway	795644	7715639	265.6	14.70	Prove Base of Alluvium
HWBH03	Weir	800891	7713730	263.5	14.00	5m Competent Rock

All boreholes were drilled by use of a D1200 GEN truck mounted rig supplied by GEO Investigate and associated support vehicles. A typical rig set up is shown in Photograph 5-3.

All boreholes were initially augered to 2.5 m using a TC auger bit. PQ casing was then driven to 2.5 m with then the annulus of the sides of the hole sealed by use of foam to provide a water tight seal for flush returns. Wash borehole drilling techniques were then employed through the soil layers using a blade bit. Where in-situ permeability testing was scheduled HWT casing was advanced with the wash bore drilling to minimise disturbance in the side walls. Standard Penetration Tests (SPT) were undertaken during the wash bore drilling. The SPTs commenced at 0.5 m bgl with 1 m spacing to 2.5 m, from 2.5 m the SPTs were carried out at 1.5 m spacing to 10.0 m, after which the SPTs were carried out every 2 m. U50 undisturbed pushed tubes were taken following an SPT where the SPT indicated soft to firm cohesive soils were present (not typically encountered). Washbore drilling was employed until weathered rock was encountered and/or two successive SPT refusals occurred, or to the target depth of the hole.

Once weathered rock was identified HWT casing was seated into the rock and wireline triple tube coring techniques were employed using HQ3 and NLMC size core barrels. A variety of diamond, tungsten and composite drill bits were utilised throughout the investigation to improve core recovery and drilling efficiency through the very low strength rock encountered. Drilling was generally aided by the use of fresh water (sourced from local farm surface water turkey nests/irrigation channels) to flush up the cuttings, however where there were poor cutting returns or potential hole collapse, drilling polymer additives were employed to aid core recovery. All boreholes were drilled vertically except HSBH07 at the proposed spillway which was drilled at 60° from the horizontal.

Rock coring runs were generally drilled between 1.0 to 1.50 m length depending on the competency of the rockmass and the discontinuities encountered. However, where poor core recovery was encountered drilling runs were generally reduced to 0.5m. Zones of coreloss were encountered, but these were generally associated with

more weathered strata between competent rock blocks. Recovered cores were extruded and retrieved from core barrels by hydraulic means, and placed in core boxes, where they were wrapped in clingfilm to aid moisture retention.

Photograph 5-3: Borehole rig setup HDBH02 south side of Flinders River

Borehole logs and associated core photographs are provided in Appendix E.

5.6.1 Soil Permeability Testing

Variable head, falling head, testing were carried out to determine the permeability characteristics in the alluvial soils encountered. Where falling head testing was carried out HWT casing was advanced with washbore drilling to the required base depth of the testing zone. Where upon pea gravel as washed down the casing at act as a filter zone for the test. The casing was then lifted to the top of the filter zone and the permeability test was then undertaken. A summary of the soil permeability testing is provided in Table 5-5, while the In-situ permeability testing results are presented in Appendix F.

Table 5-5: Summary of insitu soil permeability testing

Borehole ID	Permeability Test Type	Soil within filter test zone	Testing Zone				Permeability (m/s)
			Top (mbgl)	Bottom (mbgl)	Top RL (mAHD)	Bottom RL (mAHD)	
HSBH02	Falling Head	Silty CLAY with trace sand	7.2	10.0	248.4	245.6	6.54×10^{-7}
HSBH03	Falling Head	Silty CLAY to Gravelly CLAY	5.5	8.88	251.4	248.02	7.76×10^{-8}
HSBH05	Falling Head	Gravelly SAND	5.5	7.0	255.6	254.1	6.85×10^{-6}
HSBH14	Falling Head	SAND with trace silt	2.5	5.5	263.1	260.1	8.13×10^{-8}

Borehole ID	Permeability Test Type	Soil within filter test zone	Testing Zone				Permeability (m/s)
			Top (mbgl)	Bottom (mbgl)	Top RL (mAHD)	Bottom RL (mAHD)	
HWBH03	Falling Head	SAND with trace silt	4.0	7.0	259.5	256.5	1.04×10^{-6}

5.6.2 Rock Packer Testing

Packer testing in competent rock sequences was undertaken by the use of both single and double packer strings provided by GEO Investigate. Testing sections were determined on completion of the drilling so that the rockmass conditions could be assessed and to determine a suitably competent zone of rock to 'seat' the packer. The initial packer testing was carried out at HSBH02, HSBH03, HSBH05 and HSBH14, however it was determined that the pressure gauge supplied for the testing was unsuitable for the pressure ranges required for the project. The testing was then delayed until a suitable pressure gauge was supplied. During this period the boreholes were left open with thick gauge PVC pipe supporting the hole, and packer testing was carried out again with the new pressure gauge.

During packer testing significant issues were encountered while attempting to get a competent seal between the packer and the walls of the borehole. This was particularly noticeable in the highly to moderately weathered rockmass. Packer testing was not able to be carried out at HSBH05 and HSBH14 due to collapsing of the borehole wall.

A summary of the packer testing is presented in Table 5-6, while the packer testing results are provided in Appendix G.

Table 5-6: Summary of rock packer testing

Borehole ID	Permeability Test Type	Rock Weathering Grade	Testing Zone				Permeability (m/s) (L=Lugeon)
			Top (mbgl)	Bottom (mbgl)	Top RL (mAHD)	Bottom RL (mAHD)	
HSBH02	Single Packer	Slightly Weathered	18.0	20.2	237.6	235.4	1.0×10^{-8} (0.1L)
HSBH03	Single Packer	Slightly to Moderately Weathered	16.0	18.5	240.9	238.4	3.0×10^{-9} (0.03L)
HSBH08	Double Packer	Slightly to Moderately Weathered	7.0	9.0	252.7	250.7	2.8×10^{-9} (0.03L)
HSBH08	Single Packer	Slightly Weathered	10.0	12.0	249.7	247.7	9.6×10^{-10} (0.01L)
HWBH03	Single Packer	Slightly to Moderately Weathered	10.0	12.0	253.5	251.5	2.0×10^{-9} (0.02L)

5.6.3 Televiewer Surveys

Borehole televiewer surveys over most of the competent bedrock encountered were undertaken in boreholes as shown in Table 5-7 below.

Table 5-7 : Summary of borehole televiewer surveys

BH	Easting (m)	Northing (m)	Borehole RL (mAHD)	Length of Survey (mbgl)	Borehole Orientation	
					Inclination (^{0})	Azimuth (^{0})
HSBH02	791445.0	7714660.0	255.6	14.88-19.73	90	N/A
HSBH03	791944.0	7714643.0	256.9	12.44-17.42	90	N/A
HSBH07	795179.0	7715757.0	265.7	11.57-14.70* & 15.36-19.26*	59.9	136.5
HSBH08	795265.0	7715365.0	259.7	5.33-11.54	90	N/A
HSBH09	795880.0	7715833.0	267.0	10.34-14.83	90	N/A
HSBH14	795644.0	7715639.0	265.6	5.86-7.83 & 9.14-12.47	90	N/A

Note : * Length on incline

These surveys were undertaken to define the rockmass discontinuity characteristics, as well as for inclined borehole survey measurement and were undertaken by Golder Associates Pty Ltd. Temporary PVC casing was installed in the upper soil / alluvial layers above the rock materials to ensure no fall-in or necking. Both ATV and OTV surveys were undertaken with all dip/dip directions of identified discontinuities reported as True North.

Before undertaking the televiewer, a calliper tool was run down the borehole to determine if the hole had remained open. Significant issues with collapse of the sidewalls were encountered (which in some cases could have been caused by possible fracturing of the ground due to the packer testing inflation pressures) and boreholes often had to be redrilled to clear blockages. Where the hole was still blocked after redrilling, drilling polymer was added in an attempt to support the side walls.

Televiewer survey results are presented in Appendix C Geophysical Survey Report .

5.6.4 Piezometer Installations

On completion of drilling standpipe piezometers were installed in selected boreholes across the dam alignment, as shown in Table 5-8. The piezometers were installed using 50mm PVC with a sand filter and a bentonite cap.

Table 5-8: Summary of piezometer installations

Borehole ID	Filter zone			
	Top (mbgl)	Bottom (mbgl)	Top RL (mAHD)	Bottom RL (mAHD)
HCBH03	3.0	16.0	270.565	257.565
HDBH01	5.0	11.5	251	244.5
HSBH02	3.0	20.1	252.609	235.509
HSBH03	3.0	18.5	253.885	238.385
HSBH05	8.0	15.9	253.1	245.2
HSBH08	5.0	12	254.703	247.703
HSBH11	5.0	13.0	255.913	247.913
HSBH14	7.0	14.7	258.572	250.872

Where a piezometer was not installed the boreholes were grouted to surface with a 5:1 cement:bentonite grout mix.

Response tests (to ensure good working order), comprising of pseudo variable head tests (either filling or bailing out) were undertaken on all installed piezometers.

Headworks of each piezometer consisted of a "concreted-in" steel monolith cover.

Groundwater monitoring of these installations is provided in Section 5.8.

An overall summary of the project specific borehole investigations is provided in Table 5-9.

5.7 Logging and Photography

Logging was undertaken in general accordance with AS1726:2017. All logging was carried out on site by suitably qualified Jacobs Ground Engineers. Test pit and borehole logs have been generated by the use of gINT computer software and are provided in Appendix E, which also provides Jacobs standard notes on logging and symbols used on the gINT logs. Core and typical test pit face photographs are also provided after the associated logs in Appendix E.

On receipt of soil index and strength related laboratory test results the logs were amended appropriately to reflect these results.

5.8 Groundwater Field Testing and Monitoring

Field and laboratory groundwater testing was carried out at selected boreholes, as well as river water and farm storage pond water were also obtained. Onsite groundwater testing was carried out using a YSI Pro Plus Multi-parameter Meter with pH, DO, ORP, EC and Temp readings recorded. Results from the field testing are reported in Table 5-11. Sampling and field testing was undertaken on the last day of fieldwork after the development of the standpipes. Development of standpipes was undertaking by manual bailing by taking out at least 3 times the theoretical volume of water and the allowing recharge overnight.

Groundwater samples from the recharge piezometers, along with Flinders River and dam storage samples, were obtained by use of a Teflon bailer. After field testing was completed samples were stored in purpose procured plastic bottles and then stored at 4°C in cold boxes. The samples were then transported by road to the NATA accredited ALS Townsville Laboratory and tested within 24 hours of field recovery. Table 5-12 presents the results of the groundwater laboratory testing with the associated test certificates provided in Appendix I.

Groundwater levels were recorded and presented in Table 5-10.

Table 5-9: Summary of Borehole Investigations

Borehole ID	Easting (m)	Northing (m)	Elevation (mAHD)	Orientation	Diameter	Coring Start – Finish (mbgl)	Termination (mbgl)	Permeability Testing (Depth and Permeability)	Packer Testing (Type – Depth and Permeability)	Televiewer Survey	Borehole Installations
HCBH01	800988	7714354	272.0801711	Vertical	H Size	7.41	15	No	No	No	Grouted
HCBH02	800546	7714515	269.4978306	Vertical	H Size	2.8	12.1	No	No	No	Grouted
HCBH03	800262	7714627	273.5651356	Vertical	H Size	1 – 16.05	16.05	No	No	No	Standpipe piezometer at base
HCBH04	799763	7714808	271.0497797	Vertical	H Size	2.95 – 12.0	12	No	No	No	Grouted
HDBH01	790910	7714179	254.0503764	Vertical	H Size	No Coring	20.3	No	No	No	Standpipe Piezometer Screen from 5.0 to 11.0m
HDBH02	791010	7714006	257.1352859	Vertical	H Size	No Coring	25.29	No	No	No	Grouted
HRBH01	801122	7714205	269.054387	Vertical	H Size	No Coring	11.88	No	No	No	Grouted
HSBH02	791445	7714660	255.6092542	Vertical	H Size	15.14 – 20.1	20.1	7.2 to 10.0m / 6.19 x10^{-7} m/s	Single – 18.0 to 20.2m / 1.0 x10^{-8}		Standpipe piezometer at base
HSBH03	791944	7714643	256.8850395	Vertical	H Size	8.88 – 18.5	18.5	5.5 to 8.88m / 3.3 x10^{-8} m/s	Single – 16.0 to 18.5m / 3.0 x10^{-9} m/s		Standpipe piezometer at base
HSBH04	792738	7714735	257.8001529	Vertical	H Size	4.41 – 10.7	10.7	No	No	No	Grouted
HSBH05	793794	7714993	261.0521939	Vertical	H Size	8.86 – 15.9	15.9	5.5 to 7.0m / 3.15 x10^{-6} m/s	No	No	Standpipe piezometer at base
HSBH06	794448	7715239	263.1506199	Vertical	H Size	8.88 – 13.9	13.9	No	No	No	Grouted
HSBH07	795179	7715757	265.6514539	Vertical	H Size	11.4 – 20.7	20.7	No	No		Grouted
HSBH08	795265	7715365	259.7025129	Vertical	H Size	5.82 – 12.0	12	No	Double –		Standpipe piezometer at base

Borehole ID	Easting (m)	Northing (m)	Elevation (mAHD)	Orientation	Diameter	Coring Start – Finish (mbgl)	Termination (mbgl)	Permeability Testing (Depth and Permeability)	Packer Testing (Type – Depth and Permeability)	Televiewer Survey	Borehole Installations
									7.0 to 9.0 m / 2.8 x10^{-9} m/s Single – 10.0 to 12.0m / 9.6 x10^{-10} m/s		
HSBH09	795880	7715833	266.9791882	Vertical	H Size	5.92 – 15.0	15	No	No		Grouted
HSBH10	797125	7715645	260.5473969	Vertical	H Size	No Coring	15.24	No	No	No	Grouted
HSBH11	797726	7715375	260.913459	Vertical	H Size	No Coring	14.55	No	No	No	Standpipe piezometer at base
HSBH12	798454	7715126	262.3788333	Vertical	H Size	No Coring	7.41	No	No	No	Grouted
HSBH13	799268	7714829	266.4387899	Vertical	H Size	No Coring	4.29	No	No	No	Grouted
HSBH14	795644	7715639	265.5722824	Vertical	H Size	7.29 – 14.7	14.7	2.5 to 5.5m / 3.18 x10^{-8} m/s	No		Standpipe piezometer at base
HWBH03	800891	7713730	260.6980099	Vertical	H Size	8.64 – 14.0	14	4.0 to 7.0m / 1.04 x10^{-6} m/s	Single – 10.0 to 12.0m / 5.3 x10^{-9} m/s	No	Grouted

Table 5-10: Summary of recorded groundwater levels

Borehole ID	Date	Water level (mbgl)	Water level RL (mAHD)	Remarks
HCBH03	30/06/2021	Dry to 14.3	Dry to 259.4	Dipped in standpipe.
	16/06/2021	Dry to 14.35	Dry to 259.25	Dipped in standpipe.
HDBH01	30/06/2021	7.32	248.68	Dipped in standpipe.
	08/07/2021	7.38	248.62	Dipped in standpipe.
	19/07/2021	7.47	248.53	Dipped in standpipe.
HSBH02	06/07/2021	17.4	238.2	Dipped down open hole.
	08/07/2021	12.8	242.8	Dipped down open hole, collapse at 15.9.
	09/07/2021	14.0	241.6	Dipped down open hole, collapse at 15.9.
	14/07/2021	Dry to 13.9	Dry to 241.7	Dipped down open hole, collapse at 13.9.
	20/07/2021	17.0	238.6	Dipped in standpipe.
HSBH03	06/07/2021	Dry to 8.3	Dry to 248.6	Dipped down open hole, collapse at 15.1.
	09/07/2021	14.1	242.8	Dipped down open hole, collapse at 15.1.
	14/07/2021	7.5	249.4	Dipped down open hole, collapse at 13.9.
	19/07/2021	12.0	244.9	Dipped in standpipe.
	20/07/2021	14.1	242.8	Dipped in standpipe.
HSBH05	08/07/2021	Dry to 7.5	Dry to 253.6	Dipped in standpipe.
	19/07/2021	Dry to 7.7	Dry to 253.4	Dipped in standpipe.
	20/07/2021	Dry to 7.8	Dry to 253.3	Dipped in standpipe.
HSBH08	09/07/2021	5.3	254.4	Dipped down open hole, collapse at 9.2.
	20/07/2021	11.1	248.6	Dipped in standpipe.
HSBH11	30/06/2021	Dry to 12.5	Dry to 248.4	Dipped in standpipe.
	08/07/2021	12.3	258.9	Dipped in standpipe, water added to flush piezometer.
	16/07/2021	Dry to 12.6	Dry to 248.3	Dipped in standpipe.
HSBH14	09/07/2021	Dry to 9.15	Dry to 256.45	Dipped down open hole, collapse at 9.5.
	20/07/2021	13.0	252.6	Dipped in standpipe.

Table 5-11: Field groundwater testing summary

Borehole ID	Date	Time	Water level (mbgl)	Water level RL (mAHD)	Temp (°C)	DO (%)	DO (mg/L)	SPC_u (S/cm)	C_u (S/cm)	TDS (mg/L)	SAL (ppt)	pH	ORP (mV)
HSBH02	20/07/2021	8:08:45 AM	17.0	238.6	20.06	39.28	3.51	4798.60	4346.40	3118.60	2.58	7.04	-193.12
HSBH03	20/07/2021	8:32:37 AM	14.1	242.8	24.70	39.73	3.24	5576.00	5544.00	3625.38	3.01	7.38	-250.95
HDBH01	20/07/2021	8:49:01 AM	7.47	248.53	24.25	13.15	1.10	318.28	313.80	206.88	0.15	7.25	-218.70
RIVER	20/07/2021	8:59:33 AM	0.7	251	20.20	3.33	0.30	280.00	254.25	181.98	0.13	7.62	-228.48
HSBH08	20/07/2021	9:28:07 AM	11.1	248.6	26.77	38.33	3.03	3808.33	3936.67	2474.33	2.00	7.59	-269.90
HSBH14	20/07/2021	9:42:24 AM	13.0	252.6	26.70	39.63	3.12	5866.33	6057.67	3813.33	3.17	7.40	-307.03
FARM DAM	20/07/2021	9:55:22 AM	0.0	270	20.53	77.43	7.40	641.25	586.50	416.00	0.31	8.05	-249.03

Table 5-12: Laboratory groundwater testing summary

Borehole ID	Date	Water level (mbgl)	Water level RL (mAHD)	pH	Conductivity (µS/cm)	Total Dissolved Solids (mg/L)	Total Hardness as $CaCO_3$ (mg/L)	Sulphate as SO_4 (mg/L)	Chloride (mg/L)	Dissolved Calcium (mg/L)	Dissolved Magnesium (mg/L)	Dissolved Sodium (mg/L)	Dissolved Potassium (mg/L)
HSBH02	20/07/2021	17.0	238.6	7.3	5,240	3,410	1,370	1,730	462	356	118	736	9
HSBH03	20/07/2021	14.1	242.8	7.6	5,530	3,590	1,430	1,710	806	360	124	849	11
HDBH01	20/07/2021	7.47	248.53	7.3	310	202	116	28	9	35	7	16	4
RIVER	20/07/2021	0.7	251	7.8	260	169	120	20	10	35	8	18	6
HSBH08	20/07/2021	11.1	248.6	8.0	3,360	2,180	461	770	647	117	41	729	8
HSBH14	20/07/2021	13.0	252.6	7.9	5,300	3,440	802	1,520	875	196	76	1,120	11
FARM DAM	20/07/2021	0.0	270	8.2	603	392	216	4	40	32	33	44	5

5.9 Soil Laboratory Testing

Laboratory testing was scheduled on recovered soil samples from test pits and boreholes. Samples were freighted to Brisbane and testing was carried out by TriLabs at their NATA accredited laboratory.

Laboratory testing was scheduled in two rounds. The first round of testing was aimed to assist with the material classification; assess the dispersivity and erosion potential (for potential use as a dam clay core source); and assess the mechanical and chemical properties of the natural soils. The first round of laboratory testing comprised Particle Size Distribution with Hydrometers, Atterberg Limits with Linear Shrinkage, Pinhole dispersion and Lime Demand tests. Materials with different origins (Holocene and Pleistocene alluvium and residual to highly weathered rock) , depths and locations within the proposed HIP were selected for testing.

A summary of geotechnical testing methods undertaken for the first round of testing is outlined in Table 5-13.

Table 5-13 : Summary of first round testing methods undertaken of the geotechnical investigation

Test	Standard	No. of tests
Moisture Content	AS 1289.2.1.1	13
Atterberg Limits with Linear Shrinkage	AS 1289.3.1.2, 3.2.1, 3.3.1, 3.4.1	12
Particle Size Distribution	AS 1289.3.6.1	13
Pinhole Dispersion	AS 1289 3.8.3	13
Lime Demand	DTMR Q133	12
Soil Chemical (pH, sulphate & chloride)	Various Standards including USEPA, APHA, AS and NEPM.	6

Selected samples and their respective laboratory test results are summarised in Table 5-15 with test result certificates provided in Appendix I.

Once the test results from the first round were received, the data was analysed, and the second round of testing was scheduled. The second round of testing was focussed on the assessment of the suitability of the in-situ soil materials for incorporation into a potential earthen embankment dam. Due to the high plasticity, high linear shrinkage and dispersive nature of the in-situ materials across the site (as indicated by the construction material testing and the first round of the geotechnical investigation samples), it was determined that conditioning and amelioration of the alluvial and residual clay materials may be likely to be required for probable incorporation into the proposed dam embankment.

Naturally occurring gypsum was encountered across the site during walkover surveys, in project specific intrusive geotechnical investigations, and was found throughout the ground profile horizons. With unsatisfactorily improvement of gypsum additives trialled for the Hughenden Lake development (see Section 3.2), as well as literature research, it was determined that Hydrated Lime was a credible alternative additive to attempt to condition the clay materials and improve their index and dispersive nature.

This initiative of lime additives resulted in the second round of laboratory testing, which comprised a series of index and earthworks tests with 1% and 2% of hydrated lime added to the natural alluvial and residual soils. The determination of the potentially optimum lime addition percentages was defined by a review of the first round of Lime Demand tests on the same materials. These revealed that either after 1 or 2% lime addition no significant increase in pH was experienced by the soils tested, and therefore it was interpreted that an associated little significant betterment in soil properties would be forth coming with an increase in lime percentages.

Tested materials for the second round of tests were from a similar origin and location as those tested in the first round. Testing comprised Standard Compactions on natural materials, and then on samples with 1% and 2% of hydrated lime added (and allowed to cure in laboratory atmospheric conditions for 2 days). After the same lime addition percentages and curing period Atterberg Limits, Pinhole Dispersions, Laboratory Variable Head Permeability, and Remoulded Triaxial tests (at 95% MDD @ OMC) were undertaken. Table 5-14 presents a summary of the test methods undertaken during the second round of laboratory testing. Table 5-16 provides summaries of the associated soil index test results, Table 5-17 presents results of the chemical related testing, while Table 5-18 provides results of triaxial recompacted effective stress & laboratory permeability tests.

Table 5-14: Summary of the test methods for the second round of geotechnical laboratory testing undertaken

Test	Standard	No. of tests
Moisture Content	AS 1289.2.1.1	10
Atterberg Limits with Linear Shrinkage	AS 1289.3.1.2, 3.2.1, 3.3.1, 3.4.1	8
Pinhole Dispersion	AS 1289 3.8.3	8
Standard Compaction (Moisture/Density Relationship)	AS 1289 5.1.1 & AS 1289.2.1.1	9
Soil & Groundwater Chemical (pH, EC, Resistivity, Lime Dissolving Carbonic Acid, Sulphate, Chloride, Ammonia, Dissolved Major Cations)	Various Standards including USEPA, APHA, AS and NEPM.	8
Laboratory Falling Head Permeability	AS1289.6.7.2, 5.1.1	2
Consolidated Undrained Triaxial Test	AS1289.6.4.2	9

Table 5-15 : Summary of first round of laboratory test results

Sample ID	Depth (mbgl)	Material Type	Origin	Pinhole Dispersion	Moisture Content (%)	Atterberg Limits				Particle Size Distribution – Retained (%)				pH	Lime Demand (%)
						LL %	PL %	PI %	LS %	2.36 (mm)	0.425 (mm)	0.075 (mm)	0.0027 (mm)		
HRTP03	1.60-2.00	Sandstone: engineering properties of SAND	Alluvium	Highly Dispersive	2.3	NP	NP	NP	NP	10	74	16	0	8.7	3 (1)
HCTP01	1.50-1.90	Siltstone: engineering properties of gravelly CLAY	Alluvium	Highly Dispersive	20.3	68	24	44	19	22	13	35	43	9.8	4 (2)
HCTP02	1.60-1.90	Silty CLAY	Alluvium	Highly Dispersive	18.8	69	26	43	21.5	9	4	31	60		4 (2)
HCTP03	2.80-3.10	Siltstone: engineering properties of gravelly CLAY	Residual	Highly Dispersive	18.4	71	28	43	17.5	7	5	32	61		4 (2)
HCTP04	0.50-0.90	CLAY	Alluvium	Highly Dispersive	18.5	71	29	42	22.5	3	2	35	62	8.4	4 (2)
HCTP05	1.10-3.00	Siltstone: engineering properties of gravelly CLAY / clayey GRAVEL	Residual	Dispersive	18.0	74	28	46	18.5	5	3	39	56	8.3	4 (2)
HCTP06	1.30-1.70	Siltstone: engineering properties of clayey GRAVEL	HW Rock	Highly Dispersive	17.1	69	29	40	19	2	1	42	56		3 (1)
HSTP01	2.50-2.90	Sandy CLAY	Residual	Highly Dispersive	10.5	48	20	28	16	26	11	39	33		3 (1)
HSTP02	1.05-1.50	Silty SAND	Alluvium	Dispersive	9.2	43	18	25	15.5	41	16	24	30	9.2	3 (1)
HSTP04	1.50-3.00	Siltstone: engineering properties of silty CLAY	HW Rock	Highly Dispersive	14.1	59	25	34	17.5	3	2	51	46		3 (1)
HSTP05	1.00-3.00	Siltstone: engineering properties of gravelly CLAY / clayey GRAVEL	HW Rock	Highly Dispersive	16.8	68	27	41	18	6	3	51	43		4 (1)
HSTP06	1.00-1.50	Siltstone: engineering properties of clayey GRAVEL	HW Rock	Highly Dispersive	13.1	56	23	33	16.5	5	4	59	36		3 (1)

Sample ID	Depth (mbgl)	Material Type	Origin	Pinhole Dispersion	Moisture Content (%)	Atterberg Limits				Particle Size Distribution – Retained (%)				pH	Lime Demand (%)
						LL %	PL %	PI %	LS %	2.36 (mm)	0.425 (mm)	0.075 (mm)	0.0027 (mm)		
HSTP07	1.50-2.00	Gravelly CLAY / clayey GRAVEL	Alluvium	Highly Dispersive	12.8	60	22	38	19	18	7	39	41	8.8	4 (1)

Note : NP = Non-Plastic; Lime Demand Tests – values shown in (brackets) denote % lime added where no further change in pH occurred subsequently

Table 5-16 : Summary of second round of laboratory soil index test results

Investigation ID	Depth	Material Type	Origin	Hydrated Lime	Pinhole Dispersion	Atterberg Limits				Moisture / Density		
	(mbgl)			(%)		LL %	PL %	PI %	LS %	MDD (Mg/m^3)	OMC (%)	MC (%)
HCTP01 & HRTP01	1.5-3.0	Gravelly sandy CLAY / Clayey GRAVEL	Pleistocene Alluvium	0	-	-	-	-	-	1.7	20.1	14.1
				1	Completely Erosion Resistant	58	26	32	18	1.7	21.1	14.1
				2	Completely Erosion Resistant	57	37	20	12	1.6	21.6	14.1
HCTP05 & HCTP06	1.1-3.0	Siltstone: engineering properties of clayey GRAVEL / remoulds to Gravelly CLAY	Wallumbilla Formation	0	-	-	-	-	-	1.6	23.5	15.7
				1	Completely Erosion Resistant	75	38	37	18	1.6	22.0	15.7
				2	Completely Erosion Resistant	78	45	33	17	1.6	22.4	15.7
HSTP01 & HSTP02	1.05-3.0	Clayey SAND / Sandy CLAY	Holocene / Pleistocene	0	-	-	-	-	-	1.8	15.3	9.1
				1	Completely Erosion Resistant	41	26	15	11.5			

Investigation ID	Depth	Material Type	Origin	Hydrated Lime	Pinhole Dispersion	Atterberg Limits				Moisture / Density		
	(mbgl)			(%)		LL %	PL %	PI %	LS %	MDD (Mg/m^3)	OMC (%)	MC (%)
				2	Completely Erosion Resistant	39	29	10	8			
HSTP05 & HSTP06	1.0-3.0	Siltstone: engineering properties of clayey GRAVEL / Gravelly CLAY	Wallumbilla Formation	0	-	-	-	-	-	1.6	22.3	13.7
				1	Completely Erosion Resistant	65	39	26	15.5	1.7	18.9	13.7
				2	Completely Erosion Resistant	66	46	20	14	1.7	19.6	13.7

Table 5-17: Summary of second round of laboratory tests soil lime additive & chemical analysis

Investigation ID ^	Depth	Material type	Origin	Hydrated Lime	pH	Chloride	Sulphate	EC	Resistivity	TriLab Corrosion Classification after AS 2159 (Piling Code)			
	(mbgl)			(%)		(mg/kg)	(mg/kg)	(µS/cm)	(ohm/cm)	Concrete Piles - Soil Condition A	Concrete Piles - Soil Condition B	Steel Piles - Soil Condition A	Steel Piles - Soil Condition B
HCTP01 & HRTP01	1.5-3.0	Gravelly sandy CLAY / Clayey GRAVEL	Pleistocene Alluvium	0	-	-	-	-	-	-	-	-	-
				1	11.1	240	220	668	1500	Mild	Non Aggressive	Moderate	Mild
				2	12.2	180	100	4540	220	Mild	Non Aggressive	Severe	Moderate
HCTP05 & HCTP06	1.1-3.0	Siltstone: engineering properties of	Wallumbilla Formation	0	-	-	-	-	-	-	-	-	-
				1	11.7	1260	1590	2330	429	Mild	Non Aggressive	Severe	Moderate

Investigation ID ^	Depth	Material type	Origin	Hydrated Lime	pH	Chloride	Sulphate	EC	Resistivity	TriLab Corrosion Classification after AS 2159 (Piling Code)			
	(mbgl)			(%)		(mg/kg)	(mg/kg)	(µS/cm)	(ohm/cm)	Concrete Piles - Soil Condition A	Concrete Piles - Soil Condition B	Steel Piles - Soil Condition A	Steel Piles - Soil Condition B
		clayey GRAVEL / remoulds to Gravelly CLAY		2	12.1	1240	2770	4540	220	Mild	Non Aggressive	Severe	Moderate
HSTP01 & HSTP02	1.05-3.0	Clayey SAND / Sandy CLAY	Holocene / Pleistocene	0	-	-	-	-	-	-	-	-	-
				1	11.9	80	200	1620	617	Mild	Non Aggressive	Severe	Moderate
				2	12.4	60	110	5770	173	Mild	Non Aggressive	Severe	Moderate
HSTP05 & HSTP06	1.0-3.0	Siltstone: engineering properties of clayey GRAVEL / Gravelly CLAY	Wallumbilla Formation	0	-	-	-	-	-	-	-	-	-
				1	11.7	760	2560	2370	422	Mild	Non Aggressive	Severe	Moderate
				2	12.2	840	3980	6500	154	Mild	Non Aggressive	Severe	Moderate

Note : ^ - Combined bulk samples of similar materials

Table 5-18: Summary of second round of laboratory tests soil permeability & effective stress

Investigation ID	Depth	Material type	Origin	Hydrated Lime	Permeability	Bulk Density	q (σ'_1-σ'_3)	p (σ'_1+ 2σ'_3)/2
	(mbgl)			(%)	(m/s)	(t/m^3)	(kPa)	(kPa)
HCTP01 & HRTP01^	1.5-3.0	Recompacted - Gravelly sandy CLAY / Clayey GRAVEL	Pleistocene Alluvium	0	-	-	-	-
				1	-	1.90	83	59
					-		126	96
					-		228	173

Investigation ID	Depth	Material type	Origin	Hydrated Lime	Permeability	Bulk Density	q (σ'_1-σ'_3)	p (σ'_1+ 2σ'_3)/2
	(mbgl)			(%)	(m/s)	(t/m^3)	(kPa)	(kPa)
				2	-	1.86	216	86
					-		300	135
					-		396	204
HCTP05 & HCTP06^	1.1-3.0	Recompacted - Siltstone: engineering properties of clayey GRAVEL / remoulds to Gravelly CLAY	Wallumbilla Formation	0	-	-	-	-
				1	9.2 x 10^{-9}	1.89	151	94
							227	168
							324	252
				2	-	1.89	194	89
					-		265	139
					-		344	202
HSTP01 & HSTP02^	1.05-3.0	Recompacted - Clayey SAND / Sandy CLAY	Holocene / Pleistocene	0	-	-	-	-
				1	-	1.96	183	87
					-		249	137
					-		308	188
				2	2.3 x 10^{-8}	1.97	579	247
							826	361
							948	490
HSTP05 & HSTP06^	1.0-3.0	Recompacted - Siltstone: engineering properties of clayey GRAVEL / Gravelly CLAY	Wallumbilla Formation	0	-	-	-	-
				1	-	1.91	92	41
					-		150	80
					-		223	134
				2	-	1.90	152	72
					-		238	127
					-		338	196

Investigation ID	Depth	Material type	Origin	Hydrated Lime	Permeability	Bulk Density	q (σ'_1-σ'_3)	p (σ'_1+ 2σ'_3)/2
	(mbgl)			(%)	(m/s)	(t/m^3)	(kPa)	(kPa)
HSBH02*	10.50-11.00	Silty CLAY	Pleistocene Alluvium	0	-	2.14	146	114
					-		254	215
					-		453	460

Note : ^ - Combined bulk samples of similar materials ; * CU tests undertaken on natural undisturbed sample, with all other tests on recompacted samples at 95% MDD @ OMC%

6. Site Conditions

From the published data and project specific geotechnical investigations, as set out in Section 5 of this report, the geological conditions encountered were generally consistent across the site.

The site bedrock geology generally comprises interbedded Siltstone and Sandstone of the Wallumbilla Formation overlain by a variable thickness of Holocene and Pleistocene Alluvium. Subsurface conditions have been significantly influenced by palaeochannels which eroded into the underlying Wallumbilla Formation. Changes in depositional environments were believed to have occurred as a result of a period of sea level regression, which was subsequently infilled with Holocene and Pleistocene deposits as the relative sea level rose. The alluvium is deepest/thickest close to the present perennial surface water courses of the Flinders River, Stewart Creek and Back Valley Creek.

Reactive soils were also noted to be present across the site being typically less than 1.5m thick. These soils are believed to have been derived from the original basaltic residual soils that either were transported from colluvial slopes of the previously mentioned mesas in the area, &/or from former lava flows that have now been eroded by the River and creeks meandering across the inherent alluvial floodplains.

The slopes of the basalt mesas are draped in basalt talus and scree generally consisting of gravel to boulder size clasts.

Holocene Alluvium has been divided into cohesive dominated (Qh-1) and granular dominated soil horizons (Qh-2). Qh-1 typically consists of very stiff to hard dark brown silty CLAY to sandy CLAY with high plasticity. Slickensided fissures were also observed indicating shrink swell properties. Qh-2 was typically encountered near the Flinders River and Stewart Creek and consists of medium dense to dense silty SAND.

The Pleistocene Alluvium is distinguished by the weathering of clasts and surface staining and has also been subdivided into cohesive Qp-1) and granular dominated soils (Qp-2). Qp-1 is dominated by light brown very stiff to hard clayey SILT to silty CLAY with high plasticity. Qp-2 consists of dense to very dense clayey SAND to clayey GRAVEL. The distribution of Qp-2 varies across the site, however it is often encountered at low points in the underlying weathered rock profile. A gravel layer was often encountered at the base of the alluvium and in contact with the underlying weathered bedrock. This deposit is likely to be due to the erosional conditions present during the formation of these palaeochannels.

During the investigation traces of gypsum were encountered throughout the soil horizons often occurring in concentrated masses. Gypsum rich layers were identified in Flinders Shire Council borrow pits within road corridors adjacent to the proposed Irrigation Area. Significant and laterally extensive layers of gypsum crystals were identified and are thought to be associated with the Ranmoor Member of the Wallumbia Formation (see Photograph 6-1), although as previously intimated are likely within the entire stratigraphic sequences of the project site. The gypsum may have formed as evaporite beds during significant climate change through the Quaternary during periods of higher rainfall (where gypsiferous laiden water infiltrated through the soil profile – eluviation) followed by subsequent drought.

The Wallumbilla Formation encountered on site was typically described as subhorizontally interbedded Siltstone and fine grained Sandstone. The rockmass generally exhibited a normal weathering profile with the degree of weathering decreasing with depth from highly weathered to slightly weathered (see Figure 6-1). However, zones of increased weathering depths were encountered throughout the rockmass, typically around defects where the

increased flow of water caused preferential weathering. Residual soil was rarely encountered during the investigation, likely due to the erosional environment prior to the deposition of the Pleistocene Alluvium.

Photograph 6-1: Gypsum bands in residual soil layers within the Wallumbilla Formation, Council Borrow Pit

It should be noted that more mudstone dominated rock stratigraphy was encountered during the investigations for the new Alderley Bridge Crossing, while interbedded sandstones and shales were found outcropping during walkover surveys on the south side of the Flinders River (see Photograph 2-2).

Highly to moderately weathered rock generally displayed very low to low strength, and typically as weathering decreases the strength increases. Where encountered, the slightly weathered to fresh rock generally had medium to high strength. During the investigation it was noted that if the rockmass was allowed to dry out, it would quickly deteriorate becoming brittle, experience cracking and commences disintegration.

The rockmass encountered also had inclusions of Ironstone concretions and Gypsum. Iron stone concretions were encountered across the sites and are the result of localised chemical alteration where iron has precipitated around some form of nucleus causing the parent sandstone and siltstone to harden. Gypsum typically occurred as coating or infill along defects or in decomposed seams where it formed as a precipitate.

Table 6-1 below presents a summary of the geological units encountered during the investigation.

Figure 6-1: Typical Weathering Profile of the Wallumbilla Formation, HSBH04

Table 6-1: Typical descriptions of HIPCO site geological units

Geological Unit	Typical Description
Reactive Soil	Silty CLAY to CLAY, very stiff to hard, high plasticity, dark brown, black, with trace sand, fine gravel and gypsum.
Holocene Alluvium Cohesive – Qh-1	Sandy CLAY / silty CLAY, very stiff to hard, high plasticity, brown to dark brown, with trace sand, fine gravel and gypsum, trace polished slicken-sided fissures. Trace lenses of sand and gravel.
Holocene Alluvium Granular – Qh-2	Silty SAND, dense to medium dense, subrounded to angular, well graded, light brown orange, trace clay and gravel.
Pleistocene Alluvium Cohesive – Qp-1	Silty CLAY, high plasticity, very stiff to hard, light brown to brown with black staining, trace sand, highly to slightly weathered gravel, and gypsum. Lenses of sand, silt and gravel.
Pleistocene Alluvium Granular – Qp-2	Clayey SAND / Clayey GRAVEL, dense to very dense, fine to coarse grained, sub rounded to subangular, slightly to extremely weathered, pale brown with white grey mottles and black weathering, trace silt and gypsum
Residual Soil	Silty fine SAND, poorly graded, very dense.
Talus	Basalt boulders, up to 2m diameter, subangular, poorly graded, trace silt and sand.
HW Wallumbilla Formation	Interbedded Siltstone/Sandstone, extremely low to very low strength, brown, with orange black staining, sub-horizontal thinly laminated to thinly bedded, highly fractured to fracture, with trace clay coating in defects (weathered gypsum), trace highly weathered ironstone concretions.

Geological Unit	Typical Description
MW Wallumbilla Formation	Interbedded Siltstone/Sandstone, very low to low strength, grey brown, with orange black staining, sub-horizontal thinly laminated to thinly bedded, slightly fractured with trace gypsum coating / infilling in defects, trace Ironstone concretions.
SW Wallumbilla Formation	Interbedded Siltstone/Sandstone, low strength, grey with brown weathering, with orange black staining, sub-horizontal thinly laminated to thinly bedded, slightly fractured with trace gypsum coating / infilling in defects, trace Ironstone concretions.
Fresh Wallumbilla Formation	Interbedded SILTSTONE & SANDSTONE; medium to high strength, fresh, grey, poorly defined thinly laminated to laminated bedding with trace of shells and cast fossil

The following sections provide an interpretation of the of the site-specific geological model at each of the key scheme elements.

6.1 Interpreted Saego Dam Geological Model

The north east section of the Saego Dam alignment is located on a shallow ridge between the Flinders River, Stewart Creek and Back Valley Creek. The ground level steadily falls from east to west from above 270 m AHD at the north eastern end of the alignment, then to 255 m AHD along the Steward Creek / Flinders River floodplain, dropping to 249 m AHD at the Stewart Creek channel. West of the channel, the ground rises quickly and steeply up the basalt mesa to over 320 m AHD.

To facilitate the description of the site and ground models, the Saego Dam has been separated into three geological domains, the Western, Central and Eastern portions.

The Eastern portion of the Saego Dam from HSTP05 to HSTP09 is the highest elevation of the Saego Dam alignment with surface elevation between 264 to 270 m AHD. There is typically only 1 to 3m of soil overlying highly weathered Wallumbilla Formation through this area. The soil is predominantly cohesive Holocene Alluvium (Qh-1) with isolated pockets of granular Pleistocene Alluvium (Qp-2), typically filling depressions in the bedrock profile. The soil profile was underlain by highly weathered siltstone bedrock of the Wallumbilla Formation. No boreholes were drilled along this portion of the alignment therefore the depth of moderately and slightly weathered rock is unknown. However, based on the weathering profile elsewhere on the site it is likely that there would be no more than 10 m of highly to moderately weathered rock overlying the slightly weathered rockmass.

The Central portion of the Saego Dam, from HSBH09 to HSBH04, surface elevation drops from 267 m AHD to 258 m AHD. Along this section of the dam, the bedrock level becomes deeper dropping to 252 m AHD at HSBH05, while the thickness of alluvial deposits increase to up 9m. Only a thin layer of Holocene Alluvium was encountered, typically 1 to 2m, overlying up to 7m of Pleistocene Alluvium. The Holocene Alluvium was dominated by the Qh-1 unit described in Table 6-1, consisting of sandy CLAY to silty CLAY. The Pleistocene Alluvium typically was comprised a layer of sandy CLAY (Qp-1) overlying clayey SAND with lenses of sand and gravel (Qp-2). At HSBH05, a significant layer of sand and gravelly sand (approximately 5m thick) was

encountered at the base of the soil profile. This layer may occur as a depression in the bedrock profile and therefore could be material deposited in a palaeochannel.

A slight rise in the rock profile from HSBH05 to HSBH04 is observed, where only 2.5m of alluvial deposits overly the bedrock. The bedrock through this area is generally consistent with that encountered elsewhere on the site, consisting of interbedded siltstone and sandstone. The weathering profile varies in thickness from greater than 10 m of highly to moderately weathered material at HSBH09 to less than 5 m at HSBH05. This is consistent with the coarser soil encountered at HSBH05, with the higher energy environment (which deposited the sands and gravels) possibly having washed away the more weathered rock leaving a thinner weathering profile.

The Western portion of the Saego Dam extends from HSBH03 to the western abutment at the foothills (piedmont slopes) of the basalt mesa. Surface elevation is between 257 m AHD and 255 m AHD along the Stewart Creek / Flinders River floodplain, until the proposed dam wall reaches the channel of Stewart Creek where it drops to 249 m AHD then slopes up the basalt mesa. The thickness of the alluvial deposits increases up to 14m as the bedrock profile drops from 255 m AHD at HSBH04 to 242 m AHD at HSBH02. The soil comprises approximately 7m of silty CLAY of Holocene Alluvium (Qh-1) overlying up to 7m of silty to gravelly CLAY of Pleistocene Alluvium (Qp-1). A gravelly layer was encountered at the contact between the Pleistocene Alluvium and the underlying Wallumbilla Formation. The weathering profile of the Wallumbilla Formation was generally consistent with the typical site geology with highly to moderately weathered rock between 5 to 10m thick. The proposed western abutment of the dam is interpreted to be composed of basaltic derived talus, overlying finer colluvial deposits, residual clays and finally weathered rock sequences of the Wallumbilla Formation.

Table 6-2 below provides a summary of the ground model across the Saego Dam alignment.

Table 6-2: Summary of Saego Dam ground model

Geology Unit	Description	Location					
		Western Portion		Central Portion		Eastern Portion	
		Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)	Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)	Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)
Holocene Alluvium – Qh-1	Silty CLAY, high plasticity, very stiff to hard	0.0 / 7.0 – 7.3	256 / 249 - 250	0.0 / 1.0 – 4.0	258 - 267 / 257 - 268	0.0/ 0.5 – 1.35	265 - 269/ 265.5 - 268
Holocene Alluvium – Qh-2	N/A	N/A	Occurs as lenses	Occurs as lenses	Occurs as lenses	N/A	N/A
Pleistocene Alluvium – Qp-1	Silty CLAY high plasticity with lenses sandy silt stiff to hard	7.0 – 7.3 / 8.5 – 13.8	249 – 250 / 248 - 242	1.0 – 4.0/ 4.0 – 6.0	257 – 265/ 253 - 264	Occurs as lenses	Occurs as lenses
Pleistocene Alluvium – Qp-2	Clayey SAND / Clayey GRAVEL, dense to very dense,	Occurs as lenses	Occurs as lenses	4.0 – 6.0/ 5 – 8.6	257 – 263/ 252 - 264	0.5– 1.35/ 1.0 – 3.0+	265.5 – 268/ 264 – 265*

Geology Unit	Description	Location					
		Western Portion		Central Portion		Eastern Portion	
		Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)	Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)	Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)
HW Wallumbilla Formation	Interbedded Siltstone/Sandstone, very low strength	8.5 – 13.8/ 13.6 - 16.0	248 – 242 / 243 - 240	3.15 - 8.6/ 10.0 - 10.9	252 - 264/ 250 - 257	0.85 – 1.5+	264 – 268+
Note: * Only encountered in the North Eastern portion; + proven extent of layer from geotechnical investigation							

Piezometers installed along the main dam alignment recorded groundwater levels decreasing from 249.5m AHD at HSBH08 to 239m AHD at HSBH02. However, the connectivity of the aquifers between piezometers is unknown. The Flinders River is between the 250 to 252 m AHD through this area with water less than 1m below the surface. In the Western Portion of the Main Dam groundwater levels recorded at HSBH02 and 03 were approximately 10m below the water levels observed in Stewart Creek and the Flinders River which indicate the latter may be perched aquifers. An interpreted geological long section of the proposed dam alignment is presented in Appendix A on Drawing Nos. IS362200-0000-CG-DG-0011 to 0018, with a second set of drawings providing seismic refraction overlays for the western part of the proposed dam wall on Drawing Nos. IS362200-0000-CG-DG-0051 to 0055.

6.2 Interpreted Geological Model for Spillway

The Spillway is proposed to be located in the Central Portion of the Dam, where the Saego Dam transitions from shallow rock of the north eastern portion to deeper rock in the Western Portion. The alluvium is typically 5 to 7m thick at this location, becoming shallower to the east, with Reactive Soil overlying a thin layer of Holocene Alluvium and Pleistocene Alluvium. The Reactive Soil and Holocene Alluvium generally consists of sandy silty CLAY (Qh-1) with some layers of clayey GRAVEL. The Pleistocene Alluvium was typically comprised of a layer of sandy CLAY (Qp-1) overlying clayey SAND with lenses of sand and gravel (Qp-2). On the downstream side of the Spillway a creek feeds into an abandoned channel of the Flinders River. A site walkover of the area indicated that the presence of coarser SAND soils closer to the Flinders channel.

The bedrock located at the Spillway was consistent with that encountered across the site comprising sub-horizontally interbedded Siltstone and Sandstone of the Wallumbilla Formation. However, the weathering profile at the Spillway was deeper than that encountered elsewhere, with approximately 5m of highly weathered rock and 6-8m of moderately weathered rock overlying slightly weathered material. Due to the Saego Dam alignment being situated along a rising bedrock ridge, the weathered rock levels are expected to be deeper to the north and south of the Spillway.

Table 6-3 below provides a summary of the geological model for the Spillway.

Table 6-3: Summary of geological model for the Spillway

Geology Unit	Description	Spillway	
		Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)
Reactive Soil	Silty CLAY, high plasticity, very stiff to hard	0.0 / 0.5 – 1.5	265.5 - 267/ 265 – 255.5
Holocene Alluvium – Qh-1	Silty CLAY, high plasticity, very stiff to hard	0.5 - 1.5 / 1.0 – 1.85	265 - 255.5/ 264.5 - 255
Holocene Alluvium – Qh-2	Clayey GRAVEL, medium dense	1.5 / 1.85	265.5 / 265.15
Pleistocene Alluvium – Qp-1	Silty CLAY high plasticity very stiff to hard	1.0 - 1.85 / 2.5 – 3.3	255 – 264.5 / 260 – 263
Pleistocene Alluvium – Qp-2	Clayey SAND / Clayey GRAVEL, dense to very dense	2.5 – 2.8 / 3.3 – 5.62	263.0 – 263.0 / 260.5 - 263.7
HW Wallumbilla Formation	Interbedded Siltstone/Sandstone, very low strength	3.3 - 5.62 / 9.0 - 9.7	260 – 264 / 257 - 258

Notes : - + Proven depth based on geotechnical investigations; NE – Not Encountered but believed to be present below investigated depth

Groundwater was recorded to be at 248.6m AHD at HSBH08 approximately 10m below present ground level and occurring within the competent bedrock sequence.

6.3 Interpreted Geological Model for Diversion Channel

The subsurface conditions along the Diversion Channel are generally consistent with the site geology however, the thickness and nature of the alluvium varies based on the geomorphology along the alignment. Where the Diversion Channel intersects the Saego Dam, there is only a thin layer of clay dominant alluvium overlying the bedrock ridge composed of highly weathered Wallumbilla Formation.

Along the north western section of Diversion Channel where the alignment crosses a small creek, deep alluvial deposits were encountered in HSBH10 to HSBH12. The alluvial deposits consist of typically a thin layer of reactive soil, generally less than 1m, overlying 1 to 4m of cohesive Holocene Alluvium (Qh-1) and up to 12m of cohesive Pleistocene Alluvium (Qp-1). At the contact between the Pleistocene Alluvium and Wallumbilla Formation gravels were noted by 'grinding' of the washbore drill bit. No coring was carried out through this portion of the Diversion Channel and therefore the rock weathering profile is unknown.

On the south eastern section of the Diversion Channel shallow weathered rock was encountered along the flanks of basalt mesas. In this area there is generally 2m of Alluvium overlying the weathered rock profile with the exception at HCBH03 where a sandstone terrace was encountered outcropping at surface (see Figure 6-2). The Alluvium consists of less than1.5m of cohesive Holocene Alluvium (Qh-1) and less than 1m of Pleistocene Alluvium. The Pleistocene Alluvium was typically sandy to clayey SILT (Qp-1) with lenses of clayey GRAVEL (Qp-2). The bedrock is generally consistent with that encountered elsewhere on site with sub-horizontally interbedded siltstone and sandstone of low strength. The rock typically has a thin weathering profile with a less

than 1m of highly weathered rock overlying 3 to 7m of moderately weathered rock lying above slightly weathered material.

HCBH03 was drilled on a sandstone terrace which was raised above the surrounding terrain as shown in Figure 6-2. The sandstone terrace was encountered at multiple locations on the slopes of the basalt mesa. It is possible that this sandstone is a younger layer of the Wallumbilla Formation, which due to the increased strength of the coarser sandstone has resisted erosion. At HCBH03 there was approximately 3m of high to very high strength sandstone overlying the low strength interbedded siltstone and sandstone consistent with the general site geology. This stronger sandstone is typically described as:

SANDSTONE, very high to extremely high strength, slightly weathered, light brown with black mottles, fine to medium grained, with trace veins of calcite.

Figure 6-2: Sandstone Terrace near HCBH03

At the eastern end of the alignment where the Diversion Channel intersects the Flinders River, Pleistocene Alluvium was absent, with the Holocene Alluvium dominated by sand deposits. The weathering profile here includes 2m of highly weathered rock and 1m of moderately weathered rock overlying the slightly weathered to fresh material.

Two Piezometers were installed along the Diversion Channel to a depth of 12.5m (248.5m AHD) at HSBH11 and 16m (257m AHD) at HCBH03. Groundwater was not encountered in either piezometer. Groundwater was observed less than 1m below ground surface in the Flinders River near HWBH03. Based on the geomorphological character of the alignment it is likely that perched water tables (at or close to the existing ground surface) may exist, in wetter periods of the year, in the alluvial ephemeral channels that the Diversion Channel traverses. In areas outside these channels it is likely that the piezometric surface is below the base of the

proposed channel cut. The exception of a steady state high groundwater table could be the area of the Diversion Channel directly adjacent to the Flinders River.

Table 6-4 provides a summary of the geological model of the Diversion Channel.

Table 6-4: Summary of Diversion Channel geological model

Geology Unit	Description	Location			
		North Western Section		South Eastern Section	
		Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)	Depth Top / Bottom (mbgl)	Depth Top /Bottom (mbgl)
Reactive Soil	Silty CLAY, high plasticity very stiff to hard	0.0 / 0.5 – 1.5	260.5 – 262 / 259.5 - 261.5	NE	NE
Holocene Alluvium – Qh-1	Silty CLAY, high plasticity, very stiff to hard	0.5 – 1.5 / 2.5 - 4.0	259.5 -261.5 / 258 - 258.5	0.0 / 0.93 – 1.65	266.5 - 271 / 265.5 – 269.5
Holocene Alluvium – Qh-2 (5)	Clayey GRAVEL, medium dense to dense	NE	NE	0.0 / 6.5	263.5 / 257
Pleistocene Alluvium – Qp-1	Silty CLAY high plasticity very stiff to hard	1.5 – 4.0 / 5.5 – 14.2	258 - 259.5 / 246.5 - 257	0.93 – 1.65 / 1.6 – 1.85	265.5 - 269.5 / 265 – 269
Pleistocene Alluvium – Qp-2	GRAVEL, dense to very dense	11.3 / 13.5	249.5 / 247.5	Occurs as Lenses	Occurs as Lenses
Residual Soil (1)	Silty fine SAND, very dense	NE	NE	0.0 / 0.59	273.5 / 273
HW Wallumbilla Formation	Interbedded Siltstone/Sandstone, very low strength	5.5 – 14.2 +	245.5+	1.6 – 1.85 / 2.8 – 2.85	265 – 273 / 268 - 272.5
MW Wallumbilla Formation	Interbedded Siltstone/Sandstone, very low to low strength	NE	NE	2.8 – 2.85 / 7.7 – 10.5	268 - 272.5 / 260.5 – 263.5
SW Wallumbilla Formation	Interbedded Siltstone/Sandstone, low strength	NE	NE	7.7 – 10.5 +	260.5 – 263.5+

Note:

(1) Only encountered at HCBH03

(2) No core samples were taken in the north western section of the alignment, therefore the MW and SW material was not encountered

Geology Unit	Description	Location			
		North Western Section		South Eastern Section	
		Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)	Depth Top / Bottom (mbgl)	Depth Top /Bottom (mbgl)

(3) Holocene Alluvium was not encountered at HSBH11

(4) + proven depth based on geotechnical investigations

(5) Only encountered at HWBH03

An interpreted geological long section of the proposed Diversion Channel is presented in Appendix A on Drawing Nos. IS362200-0000-CG-DG-0021 to 0026, with a second set of drawings providing seismic refraction overlays for the majority of the alignment on Drawing Nos. IS362200-0000-CG-DG-0061 to 0065.

6.4 Interpreted Geological Model for Distribution Network

The Distribution Network covers a large area across different geomorphological environments. The main pipeline from the dam outlet is situated predominately on alluvial floodplain deposits of the Flinders River. Two boreholes drilled near the banks for the Flinders River on either side recorded approximately 18 to 23 m of alluvial deposits overlying highly to moderately weathered Wallumbilla Formation. The alluvial deposits consisted of 8.5 to 12 m of Holocene Alluvium overlying 6 to 14 m of Pleistocene Alluvium. The soils were generally coarser than those encountered along the Diversion Channel and Saego Dam possibly due to the higher energy deposition environment closer to the river channel. The Holocene Alluvium is dominated by SAND and silty SAND (Qh-2) with beds of sandy SILT (Qh-1). The Pleistocene Alluvium was more variable with layers of cohesive CLAY and clayey SILT (Qp-1) and SAND (Qp-2). At the contact of the Pleistocene Alluvium and the Wallumbilla Formation a layer of clayey GRAVEL was encountered.

South of the active river channel the floodplain extends for approximately 3.5 km along the pipeline alignment (see Section 7.12). No intrusive investigations were able to be carried out through this area. However, based on the likely depositional environment and site observations the soil profile is expected to consist of interfingered clays, silts and sands deposited by the meandering of the Flinders River and the ephemeral creeks throughout this area. The thickness of the alluvial deposits is expected to thin gradually to the south as the weathered rock profile rises away from the Flinders River.

Figure No. IS362200_G010 (Appendix A) indicates that the proposed Irrigation Area is composed of sodic soils. As previously indicated no intrusive geotechnical investigations were able to be carried out in the Irrigation Area. However, during the site walkover Council borrow pits excavated in the area indicated subsurface conditions consist of a thin layer of topsoil overlying residual weathered mudstone. The mudstone encountered had very low to low strength and had inclusions of gypsum veins up to 100mm thick (see Photograph 7-16). The thickness of soil lying above the weathered rock profile may vary across the site particularly where ephemeral creeks are present.

As part of the agricultural assessment and related soil management considerations of the proposed Irrigation Area, six shallow test pits (noting that Site 6 was not dug as weathered rock was identified at the surface) were excavated by use of a front loader on the Dunluce property (see Figure No. IS362200_G014 (Appendix A) & Peritus Ag Advisory Report, 18 June 2021). An example of the test pit records is provided as Figure 6-3.

At each of the shallow test pits excavation was advanced until weathered rock sequences were encountered, this being typically between 1.5 to 1.8 mbgl. The test pits revealed similar near surface ground conditions as revealed from the Council Borrow pits with the majority excavations encountering clay dominated residual soils with gypsum mottling/crystals below approximately 0.50mbgl. Moisture content of the soils encountered were initially dry, becoming moist to wet with depth. Where the underlying weathered rock sequences were encountered these were typically described as green grey gravelly clay composed of decomposed shale/mudstone.

From laboratory testing undertaken, as well as CSIRO datasets suggests that the Irrigation Area has generally a relatively low erosive potential (see Figure 6-4). This is somewhat supported by the walkover survey evidence of limited rill or gully development.

Groundwater tables are interpreted to be close to surface for the area of the alignment north of the Flinders and within the southern floodplain areas. In wetter periods perched water tables may rise to existing ground levels in these areas. For the majority of the Distribution Network outside the floodplain areas, to the south of the Flinders River, the rising topography, limited shallow test pits, and Council Borrow pits suggest that the groundwater table will be generally below the proposed pipeline invert levels.

Project: HIPCo - DBC Site: 2

Location:
WGS84. 20°42'05.9"S, 143°48'49.6"E
Described by: Lindsay Ward
Date: 9 March 2021

Site description

Geology: Alluvial
Landform Pattern: Colluvial plain
Element: Plain
Permeability: Moderately permeable
Microrelief: Zero or none
Drainage: Gradual slope
Slope: <0.25%
Rock outcrops: Nil
Surface coarse fragments: Nil
Surface condition: Self mulching, periodic cracking
Disturbance: Grazing

ASC classification: Ug6.3

Profile morphology

Horizon	Depth (m)	Description
A1	0.0 to 0.35	Reddish brown (2.5YR 5/4) dry; no mottles; medium – heavy clay; no coarse fragments; no segregations; medium to strong lenticular structure; ribbon 80mm, field pH 8.5; clear change to:
B21	0.35 to 0.85	Brown (7.5YR 4/3) hard to wet; gypsum mottles; medium clay; no coarse fragments; no segregations; lenticular structure; ribbon 55mm, field pH 8.5; clear change to:
B22	0.85 to 1.25	Dark reddish brown (5YR 3/3) hard to wet; gypsum mottles; medium clay; no coarse fragments; no segregations; lenticular structure; ribbon 55mm, field pH 9; clear change to:
C1	1.25 to 1.5	Greenish grey (Gley 1 5/10Y) dry; decomposing shale/mudstone; gravelly clay shale; gravelly coarse fragments; no segregations; field pH 10.

Figure 6-3: Example of Peritus Ag shallow test pit record for the proposed irrigation area

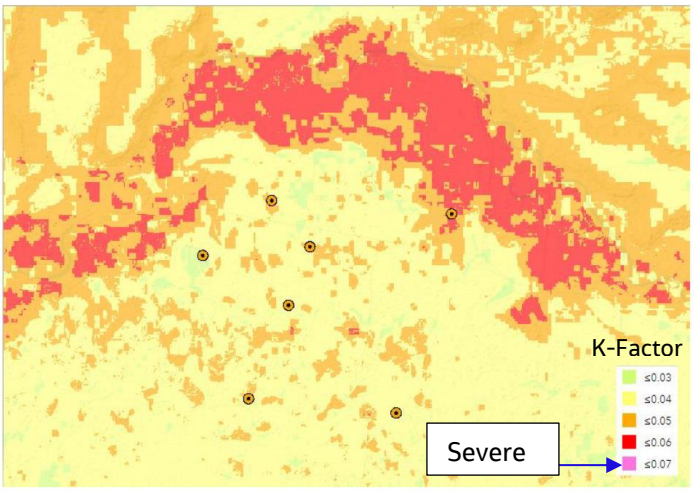


Figure 6-4: FGARA CSIRO Erosivity Prediction Map for the Irrigation Area

6.5 Interpreted Geological Model for Weir

The Flinders River where the proposed Weir is to be situated has steep banks approximately 7m high to the south and 5m high to the north side of the River. The banks of the river consist of 6.5 to 10m of Holocene Alluvium dominated by SAND to silty SAND (Qh-2) with lenses of sandy SILT (Qh-1). The alluvial deposits are underlain by highly weathered interbedded siltstone and sandstone of the Wallumbilla Formation. The CPT and geophysical investigations carried out at the Weir indicate that the depth to the interbedded siltstone and sandstone increases from 6.5m at HWBH03 to approximately 10.0m at HWCPT03 on the southern bank. The weathering profile is thinner than what was encountered elsewhere on the site with Fresh rock encountered 12.5m bgl (251m AHD).

From the investigations undertaken the general trend suggested is for deepening alluvium and a thicker weathering profile on the edges of the channel, both being greater on the southern bank indicating the potential for an old palaeochannel.

Table 6-5 below give a summary of the geological model at banks the Weir.

Table 6-5: Summary of Weir Geologic Model

Geology Unit	Description	Weir	
		Depth Top / Bottom (mbgl)	RL Top / Bottom (mAHD)
Reactive Soil	Silty CLAY, high plasticity, very stiff to hard	NE	NE
Holocene Alluvium – Qh-1	Silty CLAY, high plasticity, very stiff to hard	0.0 / 6.5 – 10.0	263.5 – 265 / 255.0 – 257.0
Holocene Alluvium – Qh-2	Clayey GRAVEL, medium dense	Occurs as lenses	Occurs as lenses
Pleistocene Alluvium – Qp-1	Silty CLAY high plasticity very stiff to hard	NE	NE
Pleistocene Alluvium – Qp-2	Clayey SAND / Clayey GRAVEL, dense to very dense	NE	NE
HW Wallumbilla Formation	Interbedded Siltstone/Sandstone, very low strength	6.5 – 10.0 / 9.1 – 13.0	255.0 – 257.0 / 252 – 254.5
MW Wallumbilla Formation	Interbedded Siltstone/Sandstone, low strength	9.1 – 13.0 / 10.1 – 16.0+	252 – 254.5 / 253 – 249+
SW Wallumbilla Formation	Interbedded Siltstone/Sandstone, low to medium strength	10.1 – 16.0 / 12.5 – 16.0+	253 – 249 / 251 – 249+
Fresh Wallumbilla Formation	Interbedded Siltstone/Sandstone, medium to high strength	12.5+	251+

From the geophysical survey correlated to the intrusive bank investigations, it is suggested that the riverbed may comprise of approximately 1m to 2m of alluvium overlying approximately 2m of XW to HW and then MW interbedded siltstone and sandstone of the Wallumbilla Formation, occurring generally at RL 254mAHD. The competent bedrock level, as mentioned previously may drop close to the existing channel banks to RL 251mAHD

on the south and 252.5mAHD on the north side. The alluvial thickness and bedrock stratigraphy and level differ quite markedly from the new Alderley Bridge Crossing located approximately 250m upstream of the proposed weir site (see Section 3.1). Here the alluvium was generally thicker & competent mudstone of the Wallumbilla Formation was encountered between approximately 246.6 and 249.7 mAHD, being deepest in the middle of the river. The differences in the geological stratigraphy between the two locations again shows the potential for not just a dynamic alluvial environment, but perhaps also indicates a change in facies between the two sites for the bedrock sequence.

The Weir is located at the site of a previous causeway used for crossing the river. The main section of the causeway is no longer present, however remnants of this were noticed at the riverbanks, as shown in Figure 6-5. The material remaining from the causeway consists of basaltic cobbles and boulders.

Figure 6-5: Remnants of Old Causeway, New Weir Location

Groundwater was encountered less than 1m below the riverbed and is likely to rise above the existing surface levels during wetter periods of the year

An interpreted geological long section of the proposed Weir is presented in Appendix A on Drawing No. IS362200-0000-CG-DG-0031, with Drawing No. IS362200-0000-CG-DG-0071 providing a seismic refraction overlay of the riverbed.

6.6 Rockmass Discontinuity Assessment

As set out in Section 2.0 of this report, no major geological structural feature lying within or in close proximity to the project site, has been identified from either published sources, API, walkover surveys and the project specific

geotechnical investigations. This is reflected in the general horizontal to subhorizontal lying rockmass of the Wallumbilla Formation, which in addition has not been affected by any contact or thermal metamorphism.

The current HIPCO scheme concept relies in the Saego Dam and the Diversion Channel Levees to be founded directly in the alluvial materials, with the Spillway and the Flinders River Weir as the only proposed elements to be founded on competent bedrock sequences.

A rockmass discontinuity assessment was carried out based on the limited data from the borehole televiewer surveys (by use of the computer software program DIPS (V8.007).

A global analysis was carried out for all boreholes where televiewer data was available within the project site (i.e. HSBH02, HSBH03, HSBH07, HSBH08, HSBH09 and HSBH14). A second analysis was undertaken for those boreholes located at or near the proposed Spillway (i.e. HSBH07, HSBH08, HSBH09 and HSBH14). No specific data was available for the Flinders River Weir.

A total of 123 poles were plotted for the project wide case, and 99 poles for the Spillway area. The result of the DIPS analysis is presented in Table 6-6 and Table 6-7 for the project wide and Spillway only analysis respectively.

Table 6-6 Project wide rockmass defects

Set	Dip (°)	Dip Direction (°)
Set 1 (Bedding Planes)	3	080
Set 2	57	020
Set 3	29	027
Set 4	37	257

Table 6-7 Spillway area rockmass defects

Set	Dip (°)	Dip Direction (°)
Set 1 (Bedding Planes)	3	050
Set 2	57	022
Set 3	32	028
Set 4	38	250

Figure 6-8 provides a graphical representation of the potential discontinuity sets along the Saego Dam alignment. It can be seen that many of the identified discontinuities have low angle dips, while Tables 6-6 & 6-7 indicate that dip directions are generally in a NE-SW direction.

Analysis of the borehole logged joints and televiewer surveys indicates the following typical planarity, defect roughness, coatings, apertures and infilling descriptions:

Bedding Planes

- Planar to irregular (in paces slickensided)
- Rough
- Iron or manganese stained
- Closed.

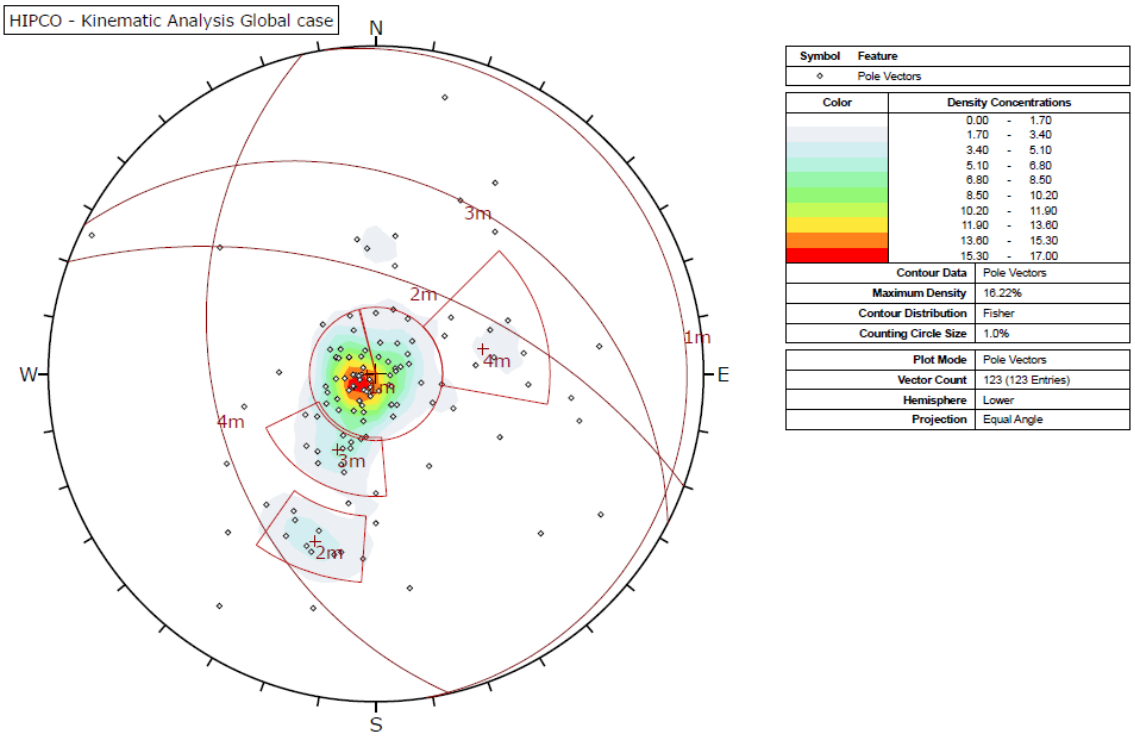


Figure 6-6: Stereoplot of Project Wide Discontinuities

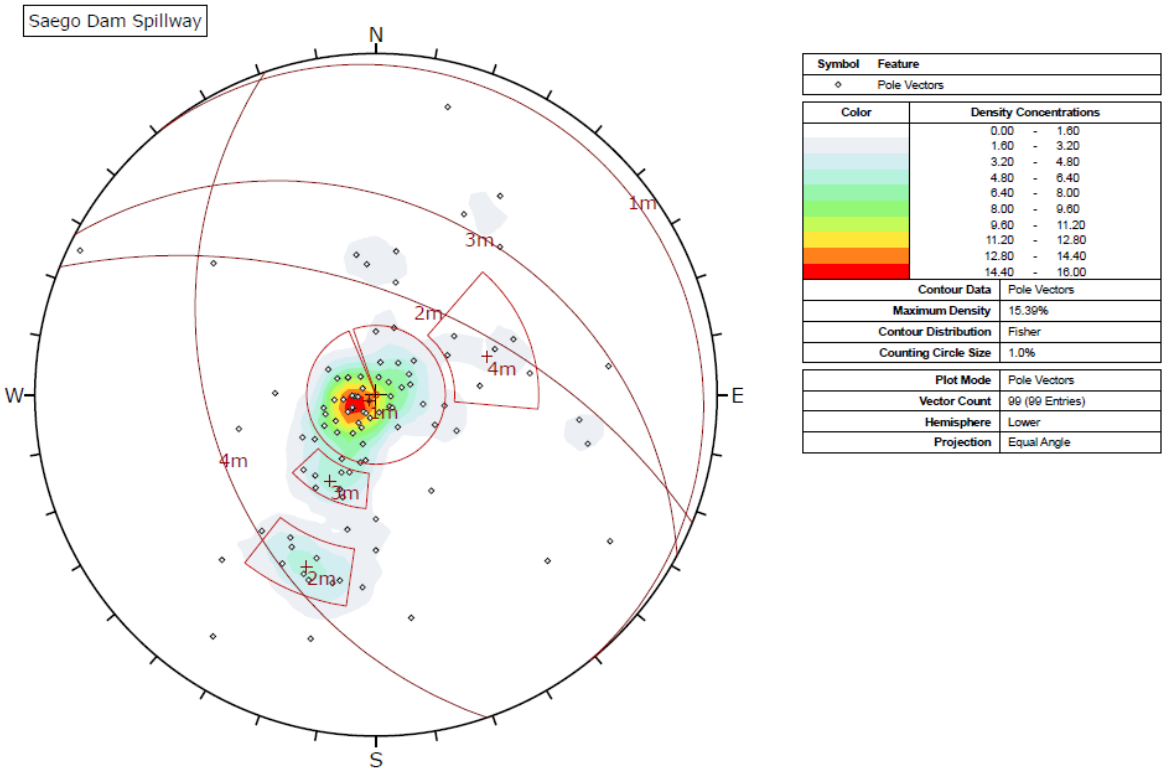
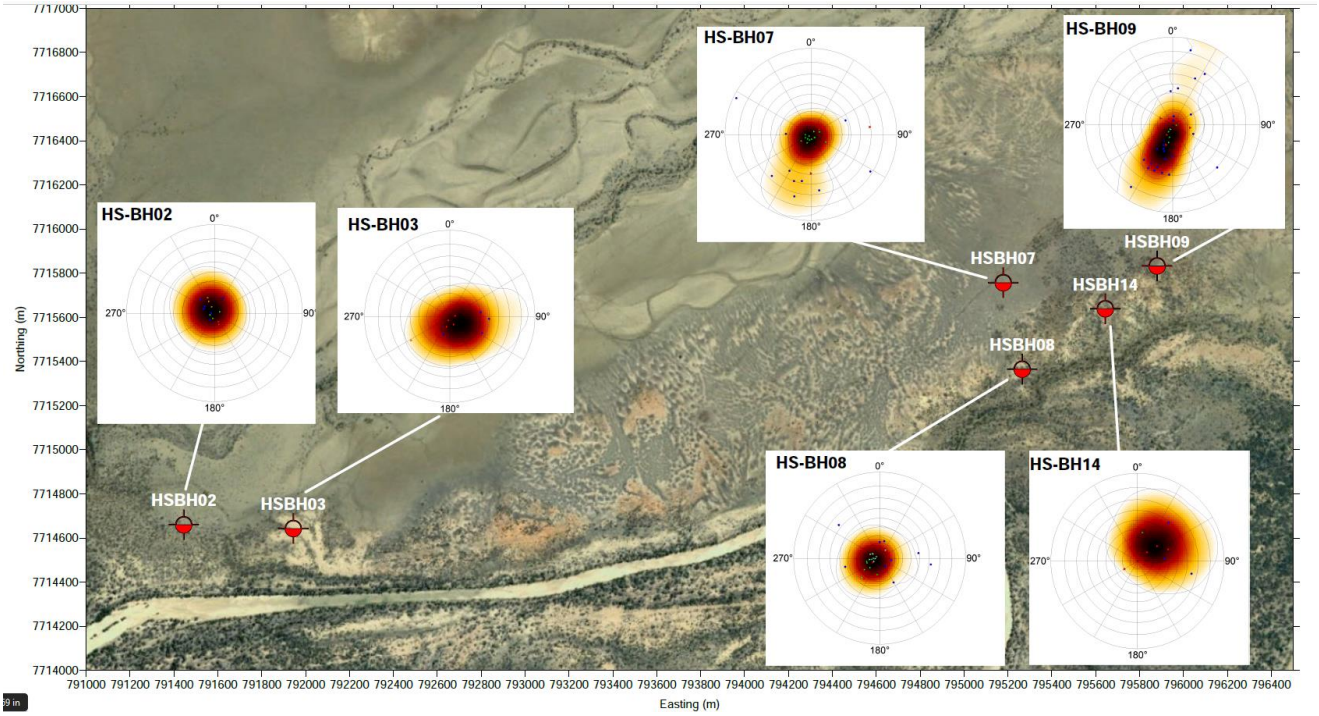


Figure 6-7: Stereoplot of Spillway Discontinuities

Figure 6-8: Televiewer Discontinuity Trends Main Saego Dam Alignment



Joints

- Planar (in paces curved)
- Rough
- Coated with Clay or Gypsum
- Closed to 10mm open.

Based on borehole televiewer surveys and logging the following typical joint spacing assessments have been made:

- Outside the Spillway area bedding planes and joints range from >2.0m (very thickly bedded or very widely spaced) to approximately 0.20m (medium bedded or moderately widely spaced)
- Within the vicinity of the Spillway discontinuity spacings reduce being typically between 0.02 to 0.06m (very thinly bedded or very closely jointed) to 0.50m (thinly bedded to closely jointed).

6.7 Site Specific Seismic Hazard Assessment

A site specific Seismicity Hazard Assessment was carried out by the Seismology Research Centre (August 2021) and has been included in Appendix J.

The report primarily considers the earthquake hazard arising from ground shaking. Secondary earthquake-related hazards, such as fault displacement, subsidence, liquefaction, landslides, seiche and tsunami, have not been considered.

The main points arising from this study, as well as the methods of application, were as follows :

- The assessment has considered Vs30 values of 760 & 448 m/s for the computation of the ground motion recurrence; no significant differences were found between applying the two velocity values;

- Calculations for this study were carried out using the weighted mean of two regional models (AUS7 210805 & NSHA13), two smoothed seismicity models (Risk Frontiers and Cuthbertson) and a background model (ARUP);

- Two ground motion models developed for Australia, four NGA-West2 models and one ENA model developed overseas were weighted and used for most calculations;

- Probabilistic hazard curves for MMI (intensity), PGA, PGV and response spectra (for a range of damping values) are also presented within the report;

- A minimum magnitude of 5.0 was used in the hazard calculations;

- Peak Ground Acceleration (PGA) levels for the proposed HIPCo Saego Dam site for a 475 year return period (10% probability of exceedance in 50 years) were calculated to be 0.0026 g and for a 10,000 year return period is 0.032 g, when considering earthquakes of magnitude 5.0 or greater and a Vs30 value of 760 m/s; little significant differences in these values were obtained for the lower Vs30 448 m/s value;

- Previous studies (using different input and analysis) produced PGA levels of less than 0.005 g up to 0.03 g for a 475 year (10% in 50 years) return period;

- Conditional Mean Spectra and de-aggregation plots were produced for the 5,000 year (SEE) return period; and

- The proposed HIPCo Saego Dam will be a High B Consequence Category Dam. ANCOLD guidelines recommend the 475 year return period probabilistic spectra for the Operating Basis Earthquake (OBE) and the 5,000 year return period probabilistic spectra for the Safety Evaluation Earthquake (SEE).

7. Geotechnical Design

7.1 Key Geotechnical Design Drivers for the Project

7.1.1 Saego Dam Geotechnical Design Drivers

The present key geotechnical drivers for the proposed Saego Dam Site are:

- Dam wall stability (upstream and downstream relating to Flinders River flood events, as well as rapid draw-down on either side of the dam wall) as a function of the intended dam construction type and dimensions, basal bearing capacity, liquefaction potential and/or the potential effects of piping failure or uplift by groundwater pressures;
- Excessive seepage of impounded water under the dam wall due to the existence of potential permeable stratigraphy possibly related to lateral extensive granular alluvium or a fractured rockmass (possibly associated with geological structural features);
- The loss of yield through the base of the impounded area by potentially a fractured rockmass or a significant thickness of granular dominated soils;
- Spillway excavatability, stability (including bearing capacity & potential kinematic feasible failure mechanisms), and erodibility; and
- The economically available source of construction materials for the earthen embankment including clay core, filter and rockfill.

7.1.2 Weir Geotechnical Design Drivers

The present key geotechnical drivers for the proposed Weir are:

- Weir stability as a function of the intended mass concrete construction type and dimensions, including basal bearing capacity, and the potential for kinematic feasible failure modes (sliding) within the rockmass foundation; and
- Excavatability and related temporary works required in the Flinders River to allow the construction of the intended mass concrete structure on the underlying competent rockmass.

7.1.3 Diversion Channel Geotechnical Design Drivers

The present key geotechnical drivers for the proposed Diversion Channel are:

- Excavatability and reuse of won material for the required channel excavation;
- Long term stability of the channel banks and any necessary support measures required; and
- Durability of the excavated channel slopes and bed and the need or otherwise of permanent lining.

7.1.4 Distribution Network Geotechnical Design Drivers

The present key geotechnical drivers for the proposed Distribution Network are:

- The excavatability and stability of the proposed pipeline trench;
- The depth and method of pipeline installation below creeks & the Flinders River floodplain;
- The optimum method of construction based on the ground conditions & potential temporary works;

- The erodibility of backfill materials for pipelines, the resistance of waterway banks, the potential scour within waterways, and buoyancy issues related to pipeline traverses through alluvial floodplains; and
- The ground conditions and the associated appropriate foundations for the proposed pumping stations.

7.2 Design Parameters Definition

7.2.1 Soil Index Parameters

From project specific laboratory testing undertaken on natural and treated both alluvial and residual soil deposits (tested for both design and possible construction material sourcing assessments), Figures 7-1 to 7-3 provide summaries of index properties for these materials.

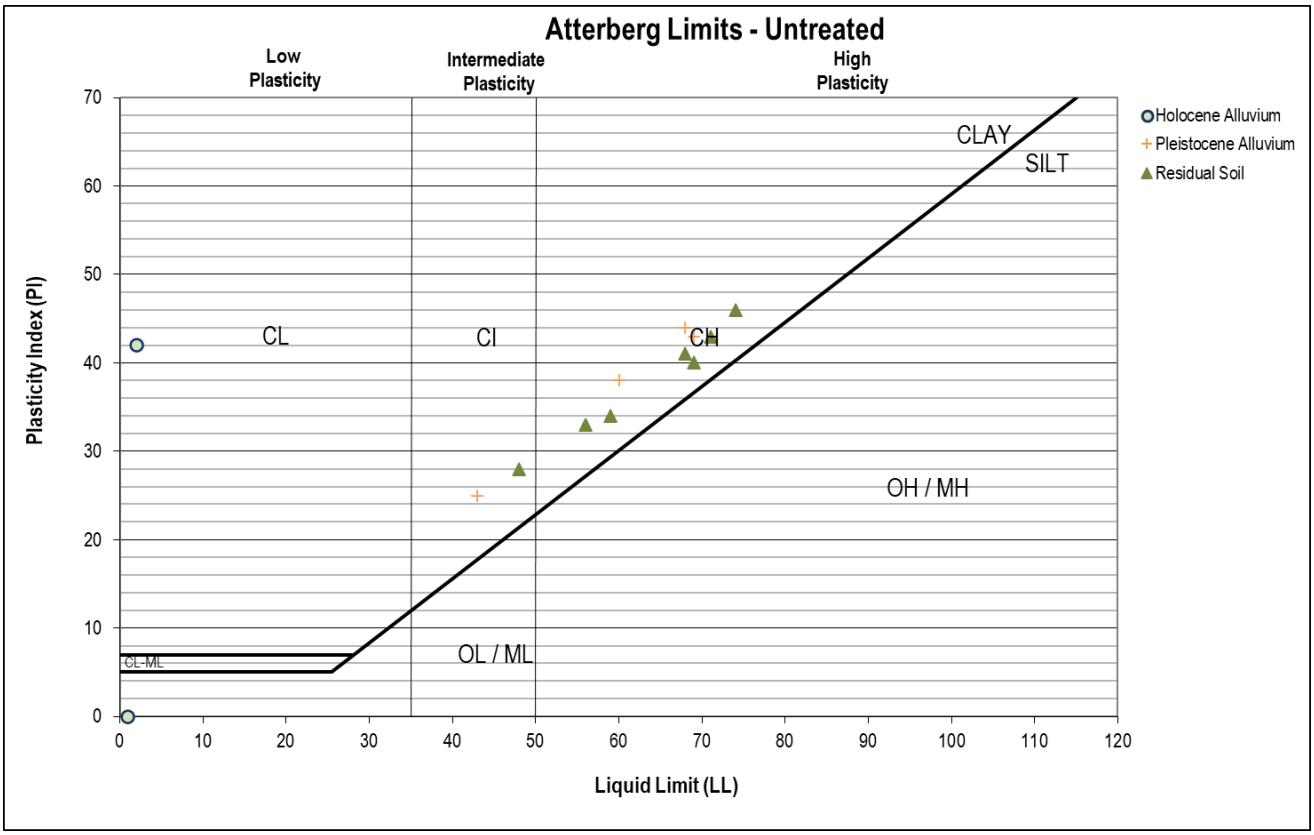


Figure 7-1: Atterberg Limits for Natural Soils

It can be seen that the natural soils generally have a high plasticity, and with lime additions this plasticity is not radically altered, but results in the modification of the natural materials from clays to silts.

As per AS1726:2017, the PSD results suggests that the vast majority of the alluvial and residual soils tested are and will behave as a cohesively dominated material. This mirrors both the factual investigation data and the interpreted geological models of the main scheme elements. The one PSD grading curve for Holocene Alluvium may represent the relatively isolated lenses of granular dominated deposits (perhaps representing palaeochannels) that occur sporadically through the alluvial stratigraphy. Here this grading curve suggests a fine to medium grained SAND.

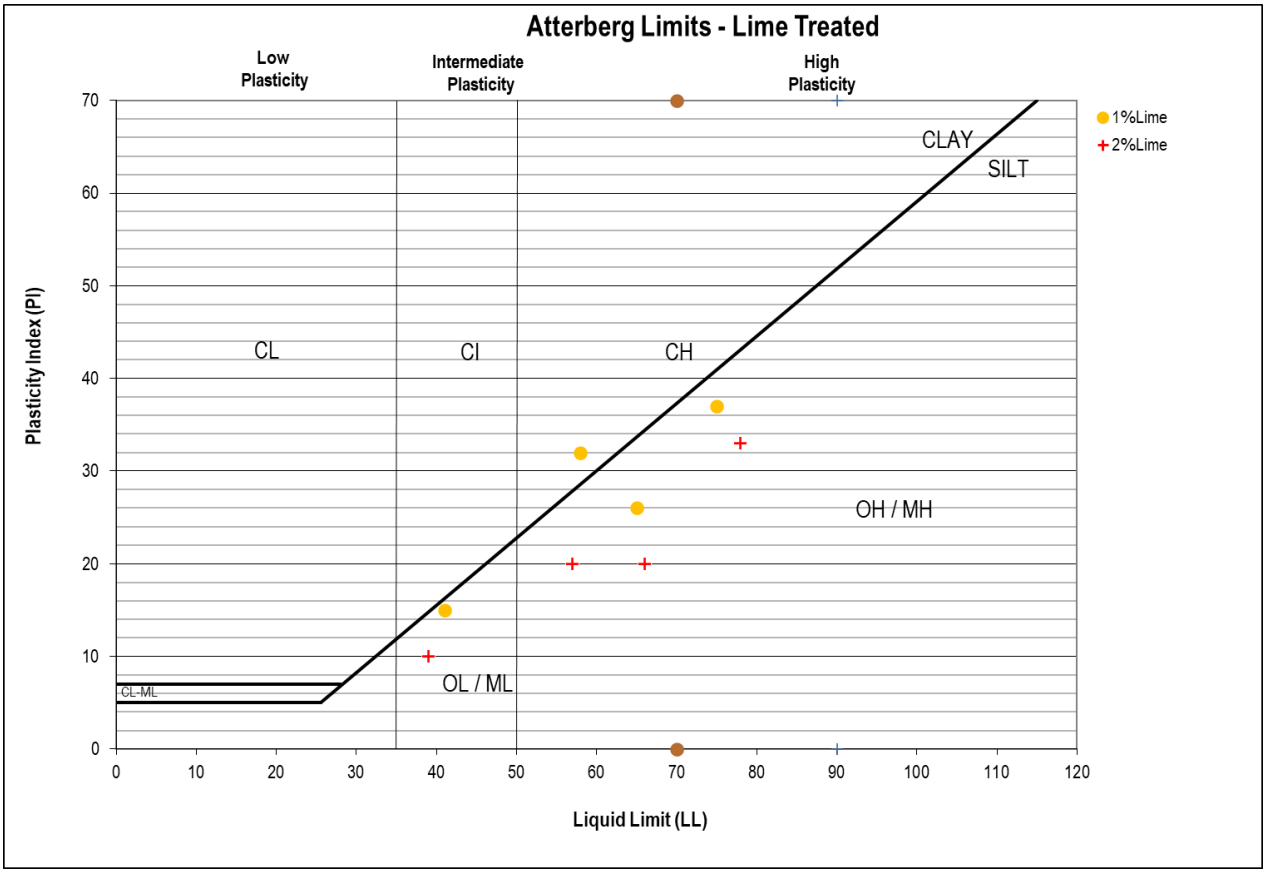


Figure 7-2:: Atterberg Limits for Alluvial & Residual Soils Treated with Lime Additives

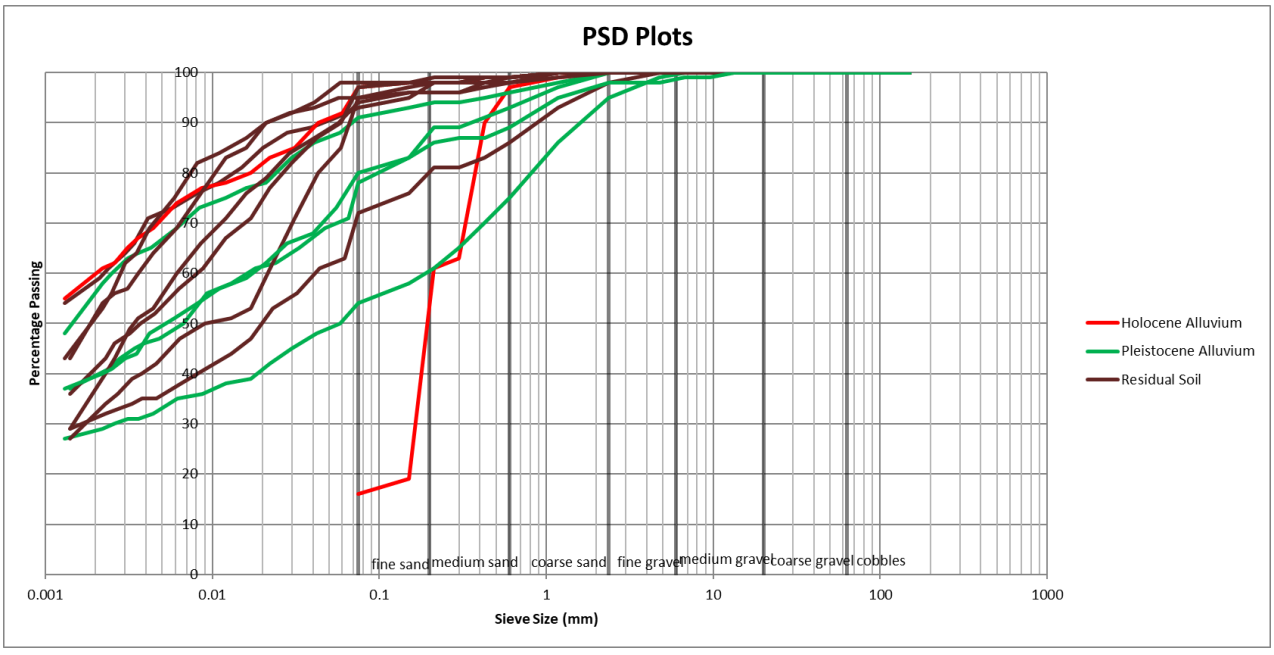


Figure 7-3: PSD for Natural Soils

7.2.2 Soil Strength & Permeability

Table 7-1 below provides specific insitu test results of encountered soil stratigraphy within the project. It can be seen that all superficial deposits, as well as residual soils, can be typically described as hard with associated undrained shear strengths >200 kPa. Permeability of these same deposits may have typically values between E -07 to E-08 m/s.

Table 7:1 Soil strength/stiffness & permeability

Geological Unit	SPT N Value No. of tests - Ave / Min / Max	Shear Wave Velocity (m/s) No. of tests - Ave / Min / Max	Permeability (m/s)
Reactive Soil	6 – 45 / 24 / 100	2 – 302 / 231 / 372	-
Qh-1	30 – 43 / 9 / 153	14 – 309 /233 / 438	7.8×10^{-8}
Qh-2	19 – 25 / 11 / 58	18 – 288 / 194 / 808	1.0×10^{-6} to 8.9×10^{-7}
Qp-1	39 – 36 / 13 /107	-	6.5×10^{-7} to 7.8×10^{-7}
Qp-2	13 – 68 / 32 / 167	-	5.6×10^{-6} to 8.1×10^{-8}
Residual Soil	1 – 173 / - / -	-	-

7.2.3 Rock Strength and Stiffness

No specific laboratory testing to determine or derive rock strength has been carried out during the project site investigations. The rock strength parameters have been derived from field descriptors.

The stiffness parameters for the project have been calculated from Serafim and Pereira (1983). The method is based on a relationship between the insitu modulus of deformation and Bieniawski's Rockmass Rating (RMR) classification. This relationship is based upon back analysis of dam foundation deformations.

Rock strength/stiffness will apply in this project for the proposed Spillway and weir where mass concrete structures have been designed to bear directly onto the bedrock.

An analysis of the boreholes within the vicinity of the Spillway (HSBH07, 08, 09 & 14) suggests that generally stiffness are not unsurprisingly relatively low because of the typical low strength bedded sedimentary rock sequence. Below approximately 9.00 to 10.00 mbgl stiffness of ≥ 5GPa could be expected within the vicinity of the Spillway Channel (see Appendix L).

An analysis of the boreholes close to the proposed Weir (including those from the new Alderley Bridge Crossing) suggests again that bedrock stiffness is typically low again due to the low strength sedimentary rocks of the Wallumbilla Formation. Below approximately 10.00 to 12.50 m below riverbed level stiffness of ≥ 5GPa could be expected within the vicinity of the Weir (see Appendix L).

7.2.4 Soil Parameters used in Stability and Seepage Analysis

As part of the project specific laboratory testing suite, triaxial Consolidated Undrained effective stress testing was undertaken on both natural undisturbed alluvial and lime treated recompacted near surface soils. Figures 7-4 & 7-5 provide collective p v q plots (Cambridge Stress Fields) used to derive potential C′ and Ø′ design parameters for primarily dam stability analysis. It should be noted that due to the consistency of the encountered alluvial and residual soils (i.e., typically hard) very few suitable undisturbed samples from these deposits were able to be recovered.

Table 7-2 provides potential effective stress parameters for the analysis undertaken based on the Mohr-Coulomb approach for collective triaxial testing on similar properties, ie., for those that could be used for Clay Core & Earthfill materials (alluvial and residual soils). It should be noted that the use of lime treated Core (Type 1A) or Earthfill materials (Type 1B) is seen as an Opportunity at the present time and the dam stability analysis has been undertaken on untreated locally won cohesively dominated soils.

The values in Table 7-2 when compared with previous projects are higher than typically used in dam stability analysis.

Table 7-2 : Analysed CU Test Results for Natural and Treated/Recompacted Alluvial & Residual Soils

Material Type/Design Line	Slope Angle (°)	Ŋ (°)	Ø′ (°)	C′ (kPa)
All Materials/Linear Regression	26.5	53	30	5
All Materials/Average	33	66	40	3
Natural Alluvium/Linear Regression	20.5	41	22	25
All Materials 1% Lime/Linear Regression	26.5	53	30	19
All Materials 2% Lime/Linear Regression	36.5	73	48	3
All Materials 2% Lime/Lower Bound Line	30.5	61	36	21

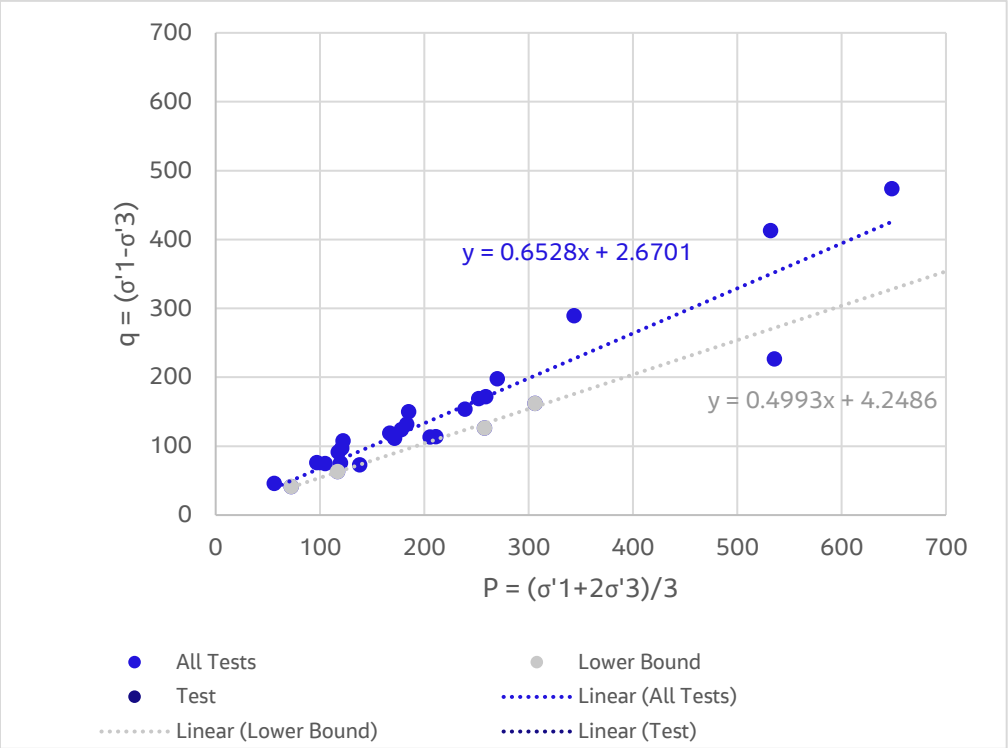


Figure 7-4: p vs q Plots for All Natural and Recompacted/Treated Alluvial & Residual Soils

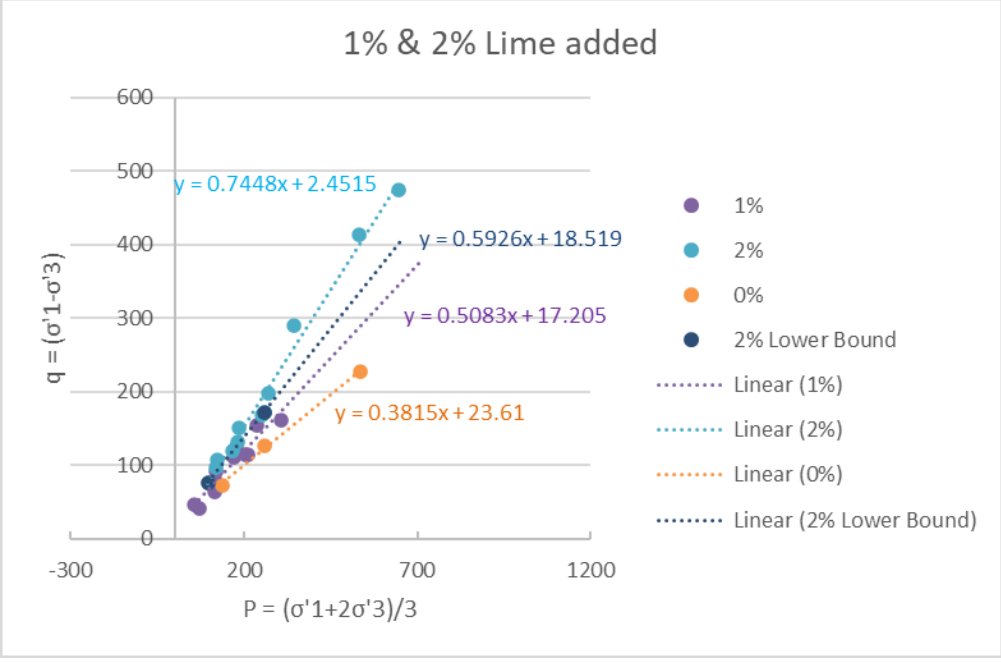


Figure 7-5: p vs q Plots for Recompacted/Treated Alluvial & Residual Soils

Table 7-3 sets out the soil design parameters that have been used in the dam stability analysis. These values are based on previous projects using similar dam material types, and also in the case of Core & Earthfill, have been

derived by the use of the Mesri approach, but modified by Stark and Idries (2021). The basis of the Mesri approach is softening of stiff clays which can lead to slope failures. The resulting shear strength relationship is curved and passes through the origin (i.e. C′ = 0 kPa). The original Mesri paper presented some relationships for the intact strength envelope related to Plasticity Index, however this has been further developed by others up to the present time with Stark and Idries being the latest approach. The Mesri paper concluded that "Fully softened shear strength is the lower bound for first time slope failures in homogenous soft to stiff clay slopes".

The modified Mesri approach is seen as particularly applicable for embankment dams with clay cores, as this is the case for this project.

Table 7-3: Summary of Soil Geotechnical Dam Construction Material Design Parameters used for Seepage & Stability Analysis

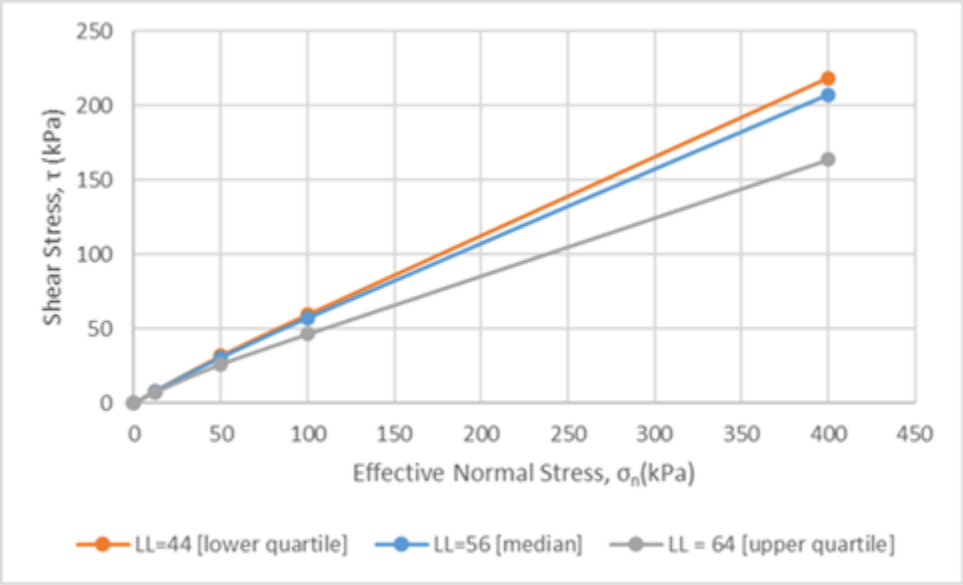
Dam Zones	Material Type	Bulk Density (kN/m^3)	c′ * (kPa)	ϕ′ * (°)	Model	Porosity (%)	k (m/s)	Residual Water Content
1A	Clay Core	18.5	0	Varies: fs	Shear/Normal: fs	0.50	k_V:10^{-9} k_H:10^{-8}	0.05
1B	Earthfill	18	0	Varies: fs	Shear/Normal: fs	0.50	k_V:10^{-8} k_H:10^{-7}	0.05
2A	Filter Fine Sand	19	0	33	Mohr-Coulomb	0.30	10^{-5}	0.03
2B	Downstream coarse filter	19.5	0	35	Mohr-Coulomb	0.35	10^{-4}	0.04
2C	Upstream coarse filter	19.5	0	35	Mohr-Coulomb	0.35	10^{-4}	0.04
3A	Rockfill	20	0	40	Mohr-Coulomb	0.40	10^{-2}	0.04
3B	Upstream Rip Rap	20	0	42	Mohr-Coulomb	0.40	10^{-1}	0.04
3C	Downstream Rip Rap	20	0	42	Mohr-Coulomb	0.40	10^{-1}	0.04

Note : - * - The fully softened shear strength values have been adopted for the locally sourced clay core and earthfill material, to characterise first time slope failures in the clay slopes. The fully softened shear strength is a function of the material's liquid limit and clay fraction (Stark and Idries, 2021), and the project specific laboratory testing has been used to derive the effective stress parameters. The upper quartile input parameters have been adopted, which then result in the lower bound strength parameters, i.e. the grey lines in Figure 7-6.

Table 7-4 provides the dam foundation insitu soil design parameters used for stability analysis.

Table 7-4: Summary of Dam Foundation Design Parameters used for Stability Analysis

Unit	Material Type	ϒ (kN/m^3)	Ø' (°)	C' (kPa)	Model	Porosity (%)	k (m/s)	Residual Water Content
Reactive Soil	-	20	Varies - fully softened	0	Mesri: fs	0.40	10^{-8}	0.05
Qh-1	Holocene Cohesive	20	Varies - fully softened	0	Mesri: fs	0.50	10^{-8}	0.05
Qh-2	Holocene Granular	20	30	0	-	0.30	10^{-5}	0.03
Qp-1	Pleistocene Cohesive	21	Varies - fully softened	0	Mesri: fs	0.50	10^{-8}	0.04
Qp-2	Pleistocene Granular	21	33	0	-	0.30	10^{-5}	0.04
Residual Clay	-	21	32	10	Mohr-Coulomb	0.50	10^{-8}	0.04
HW	Wallumbilla	-	-	-	Impenetrable	0.10	10^{-8}	0.04
MW-SW	Wallumbilla	-	-	-	Impenetrable	0.10	10^{-9}	0.04



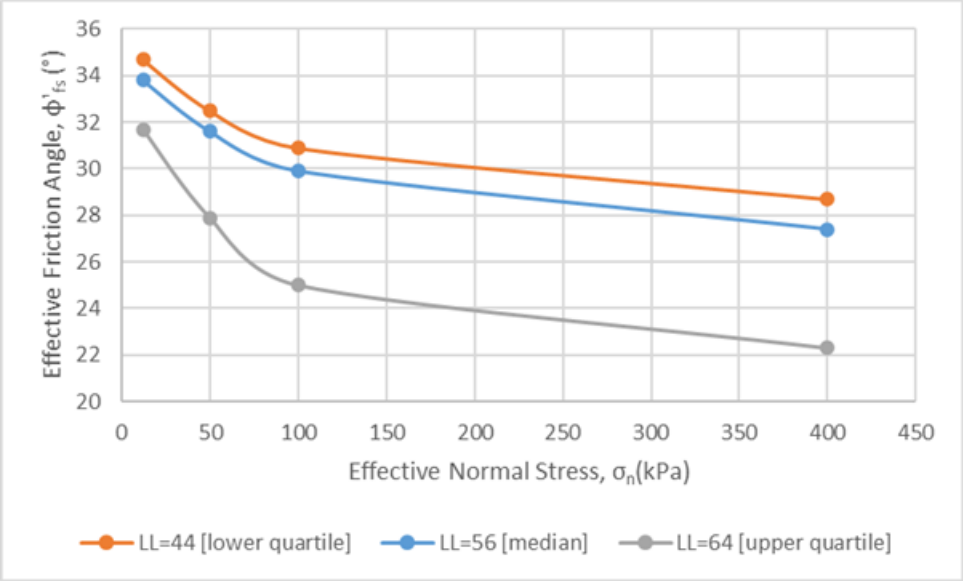


Figure 7-6: Derivation of Effective Stress Parameters for Dam Stability Assessments following Modified Mesri Approach

7.3 Dam Stability Analysis

7.3.1 Dam Wall & Foundation

Two typical sections for the main Saego Dam embankment have been designed & analysed :-

- Typical Section A – Chainage 0 to 2300m – Comprises a central clay core and earthfill dam with filters and upstream rip rap. The earthfill slopes are proposed at 1V:2.5H, with the embankment founding 2.5m below the existing surface, founded on residual soils. This section has been modelled at Ch2300, which has a maximum height of 9m (see Drawing No. IS362200-0000-CI-DG-0201, Appendix A); and

- Typical Section B – Chainage 2300 to 7825m (see Drawing No. IS362200-0000-CI-DG-0201, Appendix A) - Comprises a central clay core with rockfill shoulders and earthfill berms. The rockfill slopes are proposed at 1V:1.5H and the earthfill berms have been modelled at 1V:3H, with the embankment again founding 2.5m below the existing surface, founded on alluvium. The critical section modelled is at the crossing of Stewarts Creek (Ch7650), where the alluvium is at its thickest. This section is a maximum height of 24m, including the 2.5m stripping depth.

The dam stability assessment has been modelled within the GeoStudio 2021 suite, involving both SEEP and SLOPEW analyses.

The scenarios considered in the analysis are presented in Table 7-5, along with the minimum requirements for stability.

Table 7-5: Stability scenarios and minimum requirements

Scenario	Upstream water level (m AHD)	Minimum required Factor of Safety
1. End of construction	Nil	1.3
2. Steady state seepage (FSL)	265.1	1.5
3. Maximum flood level	269.5	1.3
4. Rapid draw down	Ground level	1.3
5. Eroded rockfill (scour)	265.1	1.3

Given the upstream water level boundary conditions, the phreatic line through the 1A core material has been determined through SEEP modelling. To achieve the minimum required factors of safety, it was necessary to model the effect of horizontal drains within the 1B earthfill material, to lower the porewater pressure within these zones. This is applicable for the upstream slope of Typical Section A and both the upstream and downstream slope of Typical Section B.

The results of the stability model are presented in Table 7-6 with the stability outputs provided in Appendix K. It should be noted that the upstream face critical stability situation is Rapid Drawdown. As this analysis has satisfied the minimum FoS (1.3) and is actually greater than 1.5, it is deemed that at FSL and Maximum Flood Level the upstream dam stability minimum FoS of 1.5 will also be satisfied.

Table 7-6: Slope stability analysis results

Typical Section	Chainage (m)	Scenario	Achieved Factor of Safety			
			Upstream		Downstream	
			(upper)	(lower)	(upper)	(lower)
A	2300	1. End of construction	-	-	1.58	-
		2. Steady state seepage (FSL)	-	-	1.58	-
		3. Maximum flood level	-	-	1.58	-
		4. Rapid draw down	1.63	-	1.58	-
		5. Eroded rockfill (scour)	Not applicable – no flood flows past Section A			
B	7680	1. End of construction	-	-	1.59	1.62
		2. Steady state seepage (FSL)	-	-	1.56	1.58
		3. Maximum flood level	-	-	1.56	1.61
		4. Rapid draw down	1.51	1.57	1.57	1.56
		5. Eroded rockfill (scour)	-	-	1.48	1.31

Dam Wall & Foundation

The potential excavations required, in the order of 2.50m, to found the embankment dam wall will be through typically very stiff to dense alluvial sediments and/or very low strength weathered rock horizons, and as such should be effectively excavated by the use of free-digging techniques by a suitable large excavator.

Liquefaction Assessment

In accordance with ANCOLD Guidelines for Design of Dams and Appurtenant Structures for Earthquake (C3.4.4.2) the "Simplified Method" for Assessing Liquefaction, has been undertaken for the main Saego Dam embankment design.

The Factor of Safety against liquefaction is calculated using the following equation:

$$FoS = \frac{CRR}{CSR}$$

The Cyclic Resistance Ratio (CRR), which is the capacity of the soil to resist liquefaction, was estimated from both the SPT results and CPT data along the Saego Dam alignment.

The Cyclic Stress Ratio (CSR), which is a measure of the cyclic load applied to the soil by the earthquake, was estimated from the outputs of the Seismic Hazard Assessment (SRC, 2021 – Appendix J). The key inputs to the liquefaction assessment were:

Minimum magnitude = 5.0

Peak Ground Acceleration (PGA) = 0.032g for a 10,000 year return period

The CSR and CRR calculations are based on a Magnitude of 7.5. A magnitude scaling factor was applied to relate the results to a magnitude 5.0.

The CSR is calculated using the following equation:

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d$$

Where:

a_{max} = peak horizontal ground acceleration at the ground surface generated by the earthquake

g = acceleration of gravity

σ_{vo} and σ'_{vo} = total and effective vertical overburden stresses, respectively

r_d = stress reduction coefficient

The CRR is calculated from insitu test data, namely SPT results from the boreholes or CPT data.

The procedures and equations are outlined in "Soil Liquefaction During Earthquakes" (Idriss and Boulanger, 2008), with the equations presented below. Note that this method is not considered to be reliable below about 15 or 20m. The maximum depth of investigation data for input into the analysis was < 20m.

For SPT data:

$$\mathrm{CRR}_{M=7.5,\sigma'_{vc}=1} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right)$$

For CPT data:

$$\mathrm{CRR}_{M=7.5,\sigma'_{vc}=1} = \exp\left(\frac{q_{c1Ncs}}{540} + \left(\frac{q_{c1Ncs}}{67}\right)^2 - \left(\frac{q_{c1Ncs}}{80}\right)^3 + \left(\frac{q_{c1Ncs}}{114}\right)^4 - 3\right)$$

The q_{c1NCS} was obtained from the estimated parameters output of CPT processing by commercial software CPeT-IT (Geologismiki, 2007).

The analysis of 8 x Boreholes and 6 x CPT data are presented in Appendix K.

Overall, the cyclic stress ratio at this site is low, owing to low peak ground accelerations. At the same time, the cyclic resistance ratio is high, due to the presence of stiff soils and limited sand lenses, which are not laterally extensive. These inputs result in high factors of safety (>2) against liquefaction at this site.

7.3.2 Dam Abutments

Access constraints did not allow the abutments for the proposed dam wall to be investigated in this phase of the project. Site observations suggest that the proposed abutments will lie within the lower piedmont (accumulation) slopes below basalt mesas. The likely stratigraphy, at both locations, of an upper talus deposit (composed basalt cobbles and boulders) overlying thin colluvium and highly weathered interbedded sedimentary rock sequences is likely to be required to be removed to key in the abutments into more competent rock.

The competent rock is likely to be near horizontally bedded and may require anchoring to provide adequate stability for these potential critical points of weakness.

7.4 Under Dam and Inundation Area Seepage Potential

7.4.1 Saego Dam Potential Piping

Section 6.0 provides a detailed assessment of the interpreted geological model of the proposed Saego Dam alignment. In addition, Tables 5.5 and 5.6 provide summaries of the insitu soil and rock permeability testing undertaken as part of the project specific geotechnical investigation. The salient points in terms of the possibility of piping failure under the proposed dam foundation, and the associated need for the installation of a grout curtain to negate this possible risk, are as follows :

- Granular dominated alluvial sequences do exist within the superficial deposits, although they are interpreted not to be laterally extensive and possibly associated with more erosional surfaces or palaeochannels; insitu permeability testing suggests Lugeon values of between 0.8 and up to 69L; and

- Across the dam alignment, as well as on a site wide basis, the bedrock sequence is a consistent interbedded siltstone and sandstone stratigraphy; the upper horizons of the more weathered rockmass may be more fractured than the competent moderately to slightly weathered rock units; packer testing in these less weathered units indicate all have permeabilities ≤1L.

With reference to Houlsby's (1976) approach, which recommends grouting required where permeabilities > 4L, as well as the potential existence of permeable layers within the alluvial sequence, it is suggested that a grout curtain or slurry trench should be installed through the alluvium and be seated into the less weathered rockmass sequence over the dam Ch 2,300-7825m (covering the Type B embankment). With this in mind, and taking into consideration the removal of approximately 2.5m of the current superficial materials to found the dam embankment upon, the possible grout curtain depths required could be in the range of 10 to 20m.

7.4.2 Inundation Area Potential Loss of Yield

From the PBC and project specific geotechnical investigations, and as mentioned previously, the majority of the alluvial sediments lying above the thin highly weathered rockmass sequence, consists of impermeable cohesively dominated soils. Only isolated and non-laterally extensive palaeochannel granular dominated lenses may occur

within the alluvial sequences. The rockmass itself is impermeable and no major geological structural features occur within or in close proximity to the proposed dam inundation area.

Taking the above into consideration it is interpreted that there is a low probability of any significant loss of yield of impounded water below the base of the proposed impounded area.

7.5 Main Spillway

7.5.1 Excavatability

Project specific boreholes HSBH07, 08 & 09 lie within close proximity of the proposed Spillway location, while HSBH14 is located within the structure's footprint just on the upstream side of the dam. PBC boreholes BH19, 20 & 22 again lie in close proximity to the proposed Spillway structure footprint. With the presumption that the Spillway structure will be located on competent rock (defined here as moderately weathered and of low to medium strength or better), required excavation could be in the order of a maximum of 10 mbgl, or RL 254 to 257 mAHD.

The potential excavations required will be through typically very stiff to dense alluvial sediments and very low to low strength weathered rock horizons, and as such should be effectively excavated by the use of free-digging techniques by a suitable large excavator.

7.5.2 Cut Slope Design

The proposed Spillway arrangement is presented in Drawing No. IS362200-00000-CI-DRG-0202 (Appendix A). The required cuttings for the Spillway structure will be temporary only and permanent retaining walls will be cast to retain the natural soil/fill, being designed to be robust enough to withstand active soil and groundwater pressures from behind the walls.

7.5.3 Potential Kinematic Feasible Failure Mechanisms

The Spillway and apron will be designed to withstand uplift pressures, being reservoir pressure at the upstream face varying linearly to tailwater pressure at the end of the apron. The critical feasible kinematic failure case is deemed to be sliding (rather than overturning) given the low strength of the siltstone.

Analysis of televiewer data, as well as derivation of appropriate rockmass design parameters, from boreholes HSBH07, 08, 09 & 14 indicates the probability of a potential kinematic feasible sliding failure mechanism below the Spillway is very low (as shown in Appendix M).

It would be preferable not to rely on anchors for the Spillway monoliths, due in part to the relatively weak rockmass, and it may be required that a shear key and or fattening the Spillway section could be needed to provide additional resisting force if it does not meet acceptance criteria. The Spillway apron will require anchors to withstand the uplift pressure.

7.5.4 Erodibility

As indicated above the wing wall and base excavations for the Spillway structure will be fill/rip-rap covered or have a concreted surface. The associated stilling basin/apron and Spillway return channel (draining to Back Valley Creek) will potentially be in erodible materials and overtime could significantly deepen with the possibility of undermining the associated downstream cuttings and the Spillway downstream apron, as well as causing detrimental environmental effects (increasing sediment load in Back Valley Creek). With this in mind the Spillway return channel cuttings will be laid back on 1v:3h and some form of protective covering may well be required for this downstream excavated channel.

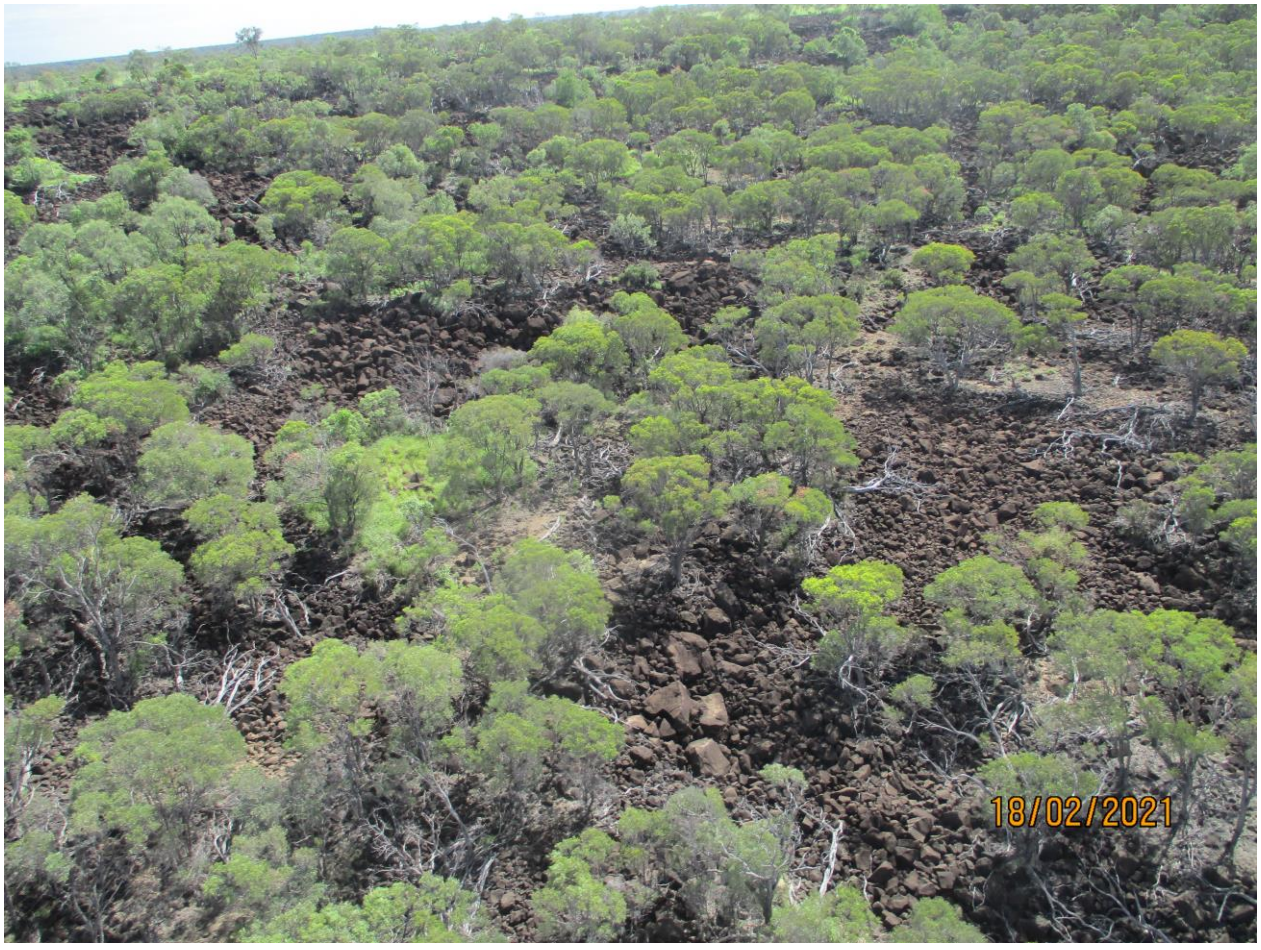
The optimum solution of channel lining will be a balance between CAPEX and OPEX, with perhaps the most effective approach could be the use of locally won available rock armour (from the proposed basalt quarry) sized to suit the critical flow case (perhaps in the order of D50=750mm). This would extend over the full extent of the Spillway return channel to Back Valley Creek.

7.6 Proposed Inundation Area Rim Stability Assessment

Walkover, helicopter flyover and remote sensing undertaken, have not identified any major historical evidence of slope instability within the proposed dam inundation rim. In addition, within the adjacent region little if any evidence of such instability has been identified from similar topographic and geological characteristics.

As previously indicated, it is evident that the basalt capped mesas weather overtime resulting in the development of colluvial piedmont slopes (see Photograph 7-1). These act as a somewhat armouring layer reducing erosion rates of the weathered Wallumbilla Formation sedimentary sequence below.

It is interpreted that there is a very low probability of a major slope instability event occurring on the slopes above the dam inundation area over the design life of the project.



Photograph 7-1: Lower mesa piedmont slopes covered in basalt cobbles and boulders

7.7 Weir

The proposed gated Weir (see Drawing IS362200-0000-CI-DRG-0203, Appendix A) will require the construction of a mass concrete base structure founded on appropriately stiff enough bedrock. Based on the interpreted ground model (see Section 6.5) and the analysis undertaken in Section 7.2.2, excavations in the riverbed may be required between approximately 9.00 and 12.5mbgl.

In order to install the weir foundation a significant temporary works excavation will be required in both cohesive and potentially highly permeable granular alluvium perhaps in the order of 4-10m thick, being well below the interpreted winter water table level (approximately 1m below existing riverbed level). An insitu permeability test undertaken in borehole HWBH03 within SAND alluvium provided a permeability of 1.04 $x10^{-6}$ m/s, however it is interpreted that higher permeabilities in the order of E-04 to -03 m/s could be expected in these deposits.

In order to reduce the risks of flooding of the works and minimize the need for dewatering/excavation support, construction should be undertaken during the dry season.

To provide a stable and dry excavation it is likely that the majority of the excavation would need to be dewatered probably by a mixer of well points (at the top) and deep wells. The excavation dimensions would also have to be relatively conservative in design and probably require a two-batter arrangement with a wide intermediate bench. Possible dimensions of the maximum excavation could be an upper 6m high batter laid back at 1v:1.5h, a 5m wide intermediate bench (on which deep dewatering wells could be installed) and a lower 6m high batter at again 1v:1.5h.

An alternative to an open excavation could be sheet piling installed from the riverbed level with deep water wells from this level. However, driving likely double clutch Type 4 thick sheet piles through the dense and densifying granular soils and gaining sufficient embedment may be highly problematic, with the probable need for anchors, and the problems associated of installing these support measures, making this option deemed to be impractical.

Ground freezing in this climate is not deemed practical and it is not a technique that is used in Australia, while jet grouting of the alluvium is not deemed to be practical to control sufficiently and may have detrimental effects to the alluvial aquifer.

Analysis of global site specific televiewer data, as well as derivation of appropriate rockmass design parameters, indicates the probability of a potential kinematic sliding failure mechanism below the Weir is very low (as shown in Appendix M), however anchors may still be required to resist water uplift pressures.

7.8 Diversion Channel Excavatability, Cut Slope Support & Lining

The potential ground conditions for the proposed Diversion Channel are presented in Appendix A on Drawing Nos. IS362200-0000-CG-DG-0021 to 0026, with Drawing Nos. IS362200-0000-CG-DG-0061 to 0065 providing a seismic refraction overlay of majority of the channel alignment

The maximum depth of cut required is approximately 12.6 mbgl, and a small section of fill will be required close to the interface with the main Sego Dam wall.

From the formulated ground model it is interpreted that the majority of the proposed cuttings can be effectively excavated by the use of standard plant (dozers, excavators and graders), with only the potential requirement for D11 with tyne rippers or excavators with hydraulic breakers between approximate Chainages 1,200 to 2,100m (noting chainages commence at the Flinders River) corresponding to the deepest section of the required cuts where slightly weathered sandstones may be encountered. In addition, between Chainages 550 & 1,200m the majority of the required cuttings could encounter highly to moderately weathered rock sequences, which could require a D10 with a tyne ripper.

Long term stability cut slope batter dimensions will be dependent on the encountered geology, but as a rule of thumb the preliminary minimum cut slope designs as set out in Table 7-7 could apply, which take into consideration the potential erodibility of the Wallumbilla Formation sequence when exposed.

Excavated materials could be used for both required areas of fill and for the proposed trailing embankment.

Due to the very high probability of degradation/erosion of exposed both soils and rock sequences, for stability, to maintain full diversion dimensions and to minimise operational phase maintenance requirements, all cut and fill slopes are more than likely be required to be lined. Linings for channels of these lengths and dimensions are of a considerable undertaking and cost-effective solutions will be difficult to achieve. Rock armouring from the winning of appropriately sized clasts from proposed project specific basalt quarries could be considered. As an alternative, composite liners including a geofabric underlay with the use of Firestone EPDM liner could be considered, or a grouted mattress arrangement, or a concrete impregnated canvas (Concrete Canvas https://www.geofabrics.co/products/concrete-canvas).

Table 7-7 : Preliminary Minimum Cut Slope Designs for the Diversion Channel

Material	Slope Category	Stabilising Measures	Slope Profile
Soil and completely weathered rocks	Temporary cutting up to 5m high	Lined open ditch drain at crest of slope	1(vert) : 1.7(horiz) No berms
	Temporary cutting up to 10m high or permanent cutting requiring significant maintenance	Lined open ditch drain at crest of slope	1(vert) : 2(horiz) No berms
	Permanent cutting requiring little or no maintenance	Lined open ditch drain at crest of slope. Surface vegetated	1(vert) : 2.7(horiz) No berms
	Permanent cutting requiring little or no maintenance	Lined open ditch drain at crest of slope. Surface vegetated. Counterforte drains within the slope	1(vert) : 2(horiz) No berms
Highly weathered rocks	Temporary cutting up to 14m high	Lined open ditch drain at crest of slope and on each berm.	1(vert) : 0.7(horiz) Slope height 7m Berm width 1.5m
	Permanent cutting requiring little or no maintenance	Lined open ditch drain at crest of slope and on each berm. Shotcrete, mesh, rock bolts, and drainage holes	1(vert) : 0.7(horiz) Slope height 7m Berm width 1.5m
Moderately weathered, slightly weathered and fresh rocks subject to degradation by weathering	Temporary cutting	Lined open ditch drain at crest of slope and on each berm.	1(vert) : 0.5(horiz) Slope height 7m Berm width 1.5m

Material	Slope Category	Stabilising Measures	Slope Profile
	Permanent cutting requiring significant maintenance	Lined open ditch drain at crest of slope and on each berm.	1(vert) : 0.7(horiz) Slope height 7m Berm width 1.5m
	Permanent cutting requiring little or no maintenance	Lined open ditch drain at crest of slope and on each berm. Shotcrete, mesh, rock bolts, and drainage holes	1(vert) : 0.5(horiz) Slope height 7m Berm width 1.5m

In terms of the 3.21km long trailing embankment this has a maximum proposed height of 9.0m, ie., not an insubstantial fill. With its proximity to the Diversion Channel and its purpose, stability of this embankment will be important to maintain. The proposed zoned embankment could be sourced from project identified construction material locations (see Section 7.11). The earthfill could be won from the Diversion Channel cuttings, but will need to appropriately moisture conditioned and compacted. Likely overall embankment slopes may be 1v:2h.

7.9 Durability Assessment

Tables 5-11, 12, 15 & 17 provide a record of the soil and groundwater chemical related test results from samples recovered for this project. Based on these results and with due reference to AS2159, the following summary conclusions are made :-

- For buried concrete non-aggressive conditions exist; and
- For buried steel moderate conditions exist.

7.10 Water Quality

Tables 5-11 & 5-12 provide summaries of the field and laboratory testing undertaken from samples recovered from the groundwater, Flinders River and from a local farm storage. In reviewing these test results the following conclusions in terms of water quality and associated water use can be made :-

1. ORP - measures the ability of a lake or river to cleanse itself or breakdown waste products; a high value = high levels of oxygen in the water; also this parameter is an indirect measure of the levels of chlorine in water - if no chlorine is free in the water no millivolts are generated; negative ORP values of -200 to -800 = alkaline water - this may explain pH values obtained due to gypsum ($CaSO_4$) being prevalent through the soil profile;

2. SPC for freshwater = 0 to 1,500 (0-800 for drinking water & suitable for livestock (can go up to 2,500); 0-300 good for irrigation - if > 300 care needed especially for sprinklers as this could cause leaf or scorch on some salt sensitive plants; 800-2500 irrigation requires special management - soils, drainage & consideration of salt tolerance; >2,500 not normal good for irrigation and for seawater = 50,000 - HSBH02 & 03 (main dam western section elevated levels could be due to gypsum) with the same for HSBH14 (eastern part of main dam);

3. Salinity - freshwater typical value $\leqslant$ 0.5ppt while seawater is approximately 35ppt - again high levels in the groundwater could be due to probable gypsum; and

4. Conductivity - most major rivers mid-range values are between 200-1,000µS/cm; high conductivity 1,000-10,000µS/cm is an indicator of saline conditions - again groundwater in certain locations may be affected by gypsum.

7.11 Construction Material Review

A variety of construction materials will be required for the proposed construction works. Table 7-8 provides volume requirements for the main earthen embankment dam. Note that additional significant volumes of aggregates will be required including aggregates for the concrete stilling basin of the Spillway and the concrete weir structure.

Table 7-8: Proposed construction material volumes required for Saego Dam

Material Type	Dam Type Code *	Estimated Material Quantities (m^3)
Clay Core	1A	927,774
Earthfill/Miscellaneous Fill	1B	1,157,894
Downstream Fine Filter	2A	349,501
Downstream Coarse Filter	28	248,923
Upstream Coarse Filter	2C	342,681
Rockfill	3A	770,068
Upstream Rip-Rap	3B	60,851
Downstream Rip-Rap	3C	255,298
Pavement	-	17,505

Note :- * See Drawing No.IS362200-0000-CI-DG-0201 (Appendix A) providing typical details of the embankment dam

A number of construction material sources have been investigated for the proposed works both locally and within the region. The potential location of these resources are presented in Figure IS362200_G-006 in Appendix A. The possible construction material sources have been identified through discussions (see Tables 7-9 to 7-14 respectively) with local earthworks contractors, quarry operators, the Flinders Shire Council, from walkover surveys & geotechnical investigations with recovery of associated sampling coupled with laboratory testing.

The optimum situation in terms of the economics and environmental impacts (with associated the gaining of associated DA for new quarries/borrow pits) for the project, dictate that as many construction materials sources required for the proposed scope of works should be obtained within the inundation area of the dam and as close to the proposed works sites as possible.

Based on the assessments to date some of the construction materials identified may meet these two main criteria, but others may not.

7.11.1 Council and Local Contractor Discussions

Over the initial weeks of the DBC Jacobs contacted and discussed, at a high level, the HIPCO project and local ground conditions with the Flinders Shire Council, local Earthworks Contractors, Concrete Suppliers and Quarry Operators.

The main purpose for these discussions was to :

- Identify potential construction material sources in terms of location, type, character and volume;
- Discussions on local large scale bulk earthworks/water storage construction – best practice;
- Identification of possible unit rates for earthworks/concrete scope of works for the project; and
- Potential plant and wet hire services for the DBC for the project, & possibly if successful local contractor capabilities for a Tier 1 or 2 Managing Contractor for a D & C contract.

Discussions were held with the following entities :-

- Flinders Shire Council
- SJ & SC Reddie Earthmoving
- Cheshire Contractors Pty Ltd
- Gromac Quarries
- Mick O'Sullivan (Quarry Consultant)
- Weiben Earthmoving

The following tables in this section set out the notes from the discussions undertaken.

Table 7-9 : Flinders Shire Council Discussion Notes

Entity	Contacts	Date of Discussion	Summary Points of Discussion
Flinders Shire Council	Hari Boppudi – Director of Engineering Department (Hughenden), but now Council CEO	22/1/21	• All rockfill, rip-rap and sands & gravels for the Alderley Bridge crossing were sourced locally • Council stated that NQCC (the Alderley Bridge construction contractor) did a good job on the bridge construction – generally concrete and structural contractor

Entity	Contacts	Date of Discussion	Summary Points of Discussion
			• Council runs its own concrete batching plants – 1 fixed and 1 mobile – they are also accredited concrete suppliers in the area • General cost of 40N concrete m^3 is around $350-450 but for large quantities could be as low as $200-250/$m^3$ • Council are currently doing $100M worth of work for DTMR in the region – this is where they get their revenue to run their work crews and their council • Council are more than happy to undertake civil/culverts/road projects for both government and commercial clients, as well as supply concrete to the same • Council indicated there are a number of locations within the region that sand and gravel can be found – generally for their own projects they utilise the road corridor to win materials, but also come to commercial arrangements with local landowners where it is relatively easy for them to get approvals to obtain 5,000 tonnes of materials (CBR 20-50% sands and gravels) • Council stated that their experience in the area suggest that there are non-aggressive subsurface conditions for buried concrete and reinforcement

Table 7-10: SJ & SC Earthmoving Contractors Discussion Notes

Entity	Contacts	Date of Discussion	Summary Points of Discussion
SJ & SC Earthmoving Contractors	Steve & Sally Reddie (Hughenden)	19/1/21	• Generally they do earthworks projects in the region including small dams • They are also interested in perhaps purchasing water, and they did attend the February 2021 talks for prospective irrigators • They are building at present a 200ML ring tank for the 15 Mile Project

Entity	Contacts	Date of Discussion	Summary Points of Discussion
			• They generally use clay liners on their dams and not HDPE • They stated that the alluvial deposits in the area are very variable and change in character vertically and horizontally significantly over short distances – not unsurprising based on the nature and geomorphological history of the Flinders floodplain • They often use black soils as a clay liner for dams – they didn't really recognise the issues with expansive clays • As an alternative to above they also use local shales in the area – unfortunately they don't have any laboratory test data for these materials and placement specification is based on local experience only – they do need to moisture condition the shales as if they don't they turn into powdery bull dust • To the question of how do you spot a shale worth using for a clay liner they indicated in the end this was based on experience, but added that don't use the shales with presumed gypsum needle crystals as the dams built of this leak • They have used local sandstone of the older sedimentary sequence (below the basalt lava flows) as sheeting for roads – they said that its interbedded with shales and siltstones but could be up to 2m thick in band width, but generally you have to chase the material when encountered • They use both the Xtraco (Miranda Greenwood) and Gromac (Peter Gross) basalt quarries for sheeting metalled roads • Weiben and Darren Beeton (Glendalough) have licenses sand pits on the Flinders, but there are multiple sand pits in the area

Table 7-11 : Cheshire Contractors Pty Ltd Discussion Notes

Entity	Contacts	Date of Discussion	Summary Points of Discussion
Cheshire Contractors Pty Ltd	Barry & Shannon Cheshire (Cairns Head Office & local office/depot in Hughenden)	28/1/21	• Cheshire constructed the Recreational Lake for The Council in Hughenden in 2018 • Cheshire has a depot/office in Hughenden that is manned generally all the time • Cheshire's work covers NQLD and they have done various projects for both government and commercial clients for 40 years now • In early 2021 they built access tracks for a 140,000 acre local cattle property through rock areas for cattle access and also for water well drilling rig exploration purposes • Apart from the Lake in Hughenden they have done various sized turkey nests and ring tank dams for farmers outside the local Hughenden region • They don't have a specific safety certification for dam walls over 8m in height • They do plant and wet hire and have built camps up to 20 man capacity and supplied electricity but not water and sewerage – for the Lake construction they set up a 20 man camp in Hughenden, supplied generator power but the Council supplied potable water and allowed them to use the local sewer network – could be an option for the HIPCO project in terms of establishing a larger camp in Hughenden to utilise town water and sewerage; they have also recently set up another 20 man camp for the Council on the Aramac/Torrens Creek Road • Cheshire have constructed pumping stations before but only gas pipeline work and not water pipelines • For the Hughenden Lake project all concrete and construction materials were supplied by the Council • Notes on the discussion on the Hughenden Lake project construction included :- o Liner for the Lake – it was reworked shale/mudstone from the excavation – the Lake site is probably beyond the influence of

Entity	Contacts	Date of Discussion	Summary Points of Discussion
			the Flinders River floodplain and they excavated and diverted Station Creek as part of the works; the shale when immersed in water is very sodic/dispersive but this was apparently caused by seams in the rockmass – when they crushed the shale and mixed it up it performed satisfactorily as a clay liner but required constant moisture conditioning for compaction purposes (2 water trucks moisture conditioning constantly during the day and 1 constantly during the night) with the clay placed at Optimum Moisture Content; initially gypsum and lime were thought about as additives but was not trialled; they had no geotechnical information before construction commenced but then Douglas Partners did some work for Premise (who were apparently the Construction Managers for the Council) and at one point Golders got involved in the conversation ○ Cheshire did undertake a number of clay sourcing investigations before construction commenced in the area for the Lake liner, but could not find anything suitable; they also confirmed black soil was not the best material to use ○ Cheshire added that when they had to clear fence lines with star pickets for the Lake construction there was literally nothing below ground, i.e.., the ground was seemingly very aggressive in nature • Cheshire have constructed a pumping station in Townsville on the Flinders River – laid 600mm diam. slotted and fabric wrapped intake pipes 12m below river bed level – first flush from the River in flood carried a huge silt bedload such that it clog the intake pipes and the associated lifting pumps such that they had to pump water out for the River directly

Table 7-12: Gromac Quarries Discussion Notes

Entity	Contacts	Date of Discussion	Summary Points of Discussion
Gromac Quarries	Peter Gross (7 local quarries within Hughenden area) Mt Beckford quarry is 12km east of Hughenden – 59km from Saego Dam	23/1/21	• Within the exposed quarry faces at the Mt Beckford Quarry the basalt lava flows have a considerable variability, however other quarries in the area (including 80km west of Hughenden – Barabon Quarry (was not visited by Jacobs)) the basalt is much less variable • At Mt Beckford the basalt is only a single bench ranging between approximately 8m to 15m thick • The Mt Beckford Quarry products produced are typically 2.1 roadbase, flood rock, gabion rock, mattress rock and concrete aggregates • Peter was unsure about the exact extraction lease covering the Mt Beckford Quarry • Mt Beckford quarry is 1 of 7 quarries Gromac own and it is operated on an itinerant basis depending on demand in the area; their crushing and earthmoving fleet move between these sites as required • Gromac's extraction license is currently capped at 100,000 t/year due to limited demand in the area; a 100,000 t campaign takes them approximately 3 months to complete • Gromac has a small fleet of triple road train side tippers and trucks, quad dogs and a large group of haulage subcontractors that they use regularly for delivery; they are able to deliver large volumes when required • Result of the following aggregate laboratory tests for Mt Beckford quarry were provided post-meeting : o Petrographic analysis o Density & Porosity o % Absorption

Entity	Contacts	Date of Discussion	Summary Points of Discussion
			○ Weak Particles % ○ Wet & Dry Strength & Variation ○ Unconfined Compressive or Point Load Strength ○ Washington Degradation ○ Los Angles Abrasion ○ 10 % Fines/Sodium Sulphate Soundness ○ Acid Akali Reaction ○ Acid Silica Reaction

Entity	Contacts	Date of Discussion	Summary Points of Discussion

Table 7-13: Mick O'Sullivan Discussion Notes

Entity	Contacts	Date of Discussion	Summary Points of Discussion
Mick O'Sullivan	Mick O'Sullivan (Emerald)	07/1/21	• Mick is an ex-DTMR geologist and consultant to extractive industries in the region & NQLD • Mick consults for Gromac quarries in the area • Gromac has 4 quarries that they work in the area, all are basalt and within the dip slope of the escarpments • Closest to the proposed project area would be Mount Beckford and Barabon quarries – Mt Beckford quarry is essentially a pyroxene rich coarse grained basalt while Barabon is a better material of higher strength; the local basalt is used for conventional concrete batching in the area as it is silica free, hard and dense – the basalt does not have any issues with Acid Alkali Reaction or Acid Silica Reaction • SGS in Townsville do laboratory testing for Gromac • Mick believes that the weathered Mudstones of the Wallumbia Formation (this is shown on the published geology maps to underlain the present proposed irrigation area and probably the PBC Saego Dam layout) when exposed (perhaps for a week) and reworked may be suitable as a clay core (but he doesn't

Entity	Contacts	Date of Discussion	Summary Points of Discussion
			have any laboratory test results); he did say that QR avoid these materials in their embankments because of their low CBR's • He didn't believe that collectively the Rolling Downs Group of sedimentary rocks in the region were suitable for aggregates, but the sandstones could be a source of sand filters or sand for concrete (again he hasn't really looked at this) • He has done some exploration in the areas around Hughenden for sand for concrete including an alluvial source about 12km N of the town perhaps on Parrot or Perth Creek – he said that getting DA's and leases approved for sand borrow pits was challenging, but he did agree that potentially sand sources in the Flinders alluvial floodplains could be used as construction materials and also agreed it may be difficult to find them due to the reworking of alluvial deposits by the river • Xtraco Marock quarry was on Peter Spears land and that the Warwombie quarry was on the Deputy Mayor's land (Clancy Middleton)

Table 7-14: Weiben Earthmoving Discussion Notes

Entity	Contacts	Date of Discussion	Summary Points of Discussion
Weiben Earthmoving	Terri & Jeff Weiben	21/1/21	• Only have a small fixed batching plant for concrete with an approximate 250m^3/day capacity • Recently sold two batching plants (1 new one – Thomas type) to Bell Rural (Cloncurry based) – Alan Bell (0458 355 275) • Standard \$365/m^3 rate for 40N concrete • At the Hughenden site (62 Hann Highway/Kennedy Development Road) there are two worked out sand pits, with the main source of sand and gravel now coming from a pit on Sussex Park (the road out to the Saego Dam site) – this has an processing license of

			200,000m^3/year but they do not have any laboratory test results for this source- stated that has <10% fines content and did go CBR 100% (as tested by Xtraco) – can produce a gravel out of the deposit • Stated that there was no shortage of sand and gravel in the region

7.11.2 Coarse Filter, Rockfill, Rip-Rap, Concrete Aggregate & Pavement Materials

From discussions with local quarry operators (and review of associated aggregate laboratory test results), regional quarry consultants and Jacobs observations of commercial and existing basalt quarries in the area, it is interpreted that the inherent lava flows can be utilised for a variety of construction material sources.

Jacobs observations of both the Gromac Mt Beckford Olivine Basalt Quarry, and the assumed Council Alstonvale basalt borrow pit (see Photograph 7-2) shows a very variable composition of the lava flows. In order to use this rock type as an aggregate source not all sections of the lava flows may be used. Many sections exposed in these quarries suggest areas are extensively vesicular in nature, very closely jointed, and may contain zones of

ash/scoria. This may mean that if a new quarry was to be developed in the basalt, care would be required to ensure the more blocky/widely jointed sections of the lava flows were utilised.

From typical laboratory test results received from Gromac, from aggregates produced from the Mt Beckford quarry, some wet strengths were just below threshold values, while chemistry results were slightly above that typical for concrete aggregates. However, the aforementioned wide and historic use of these aggregates for concrete production in the region suggests that these characteristics are acceptable.

The distance to Gromacs Mt Beckford quarry is >50km from the dam site and therefore commercial hauling significant quantities of aggregates to site is deemed to be uneconomic.

The upper stratigraphy of the escarpments within close proximity of the proposed dam site are composed of potentially the same basalt lava flows, which may have similar thicknesses and character to the Mt Beckford source. However, the basalt could actually better in nature (more massive and less vesicular) if similar to the Gromac Barabon Quarry (located towards Richmond and approximately 90km from the proposed dam site).

Figure IS362200_G006 (Appendix A) indicates a number of interpreted economic and practical escarpment areas where a possible or multiple basalt quarries could be established specifically for the project, being in close proximity to the works. Sufficient quantities of all required hard rock aggregates, as well as possibly manufactured coarse dam filter materials, armour & drainage rock, as well as road pavement materials, are believed to be available to meet project needs from these source locations.

Specific considerations in terms of environmental impacts (as these potential quarry locations lie above the dam FSL), community/stakeholder sentiment, and access/haul road construction will need to be considered as part of the identification of the optimum quarry(s) sites.

Photograph 7-2:: North side of Mesa Alstonvale valley – probably Council source for repairing the sealed road section and slope down into the valley

7.11.3 Fine Filter Material & Sand for Concrete

Fine filter material composed of sand is required to alleviate seepages through the dam on both upstream and downstream (to cater for flood events) sides. Similar materials will be required for the Diversion Channel embankment and concrete production.

Sand could be generated potentially from quarry processing of basalt, but its crushed particle shape is not advantages for both dam filter and concrete production. Alluvially derived sand is far better as a source for these material types.

Again from discussions, walkover surveys and project specific geotechnical investigations a number of existing sand pits and possible additional source locations have been identified.

The Flinders River and its associated floodplain does have a significant potential for sources of sands within an economic hauling distance from the proposed dam site (see IS362200_G006, Appendix A). Unfortunately at the present limited actual sites have not been sampled or tested, but based on the likely historic dynamic nature of the River the sand deposits that do occur could be variable in composition in terms of not only sand grain sizes, but fines content and the amount and size of gravel clasts.

A Council used alluvial sand and gravel pit on the Torver Valley Road was identified and sampled. Jacobs observations suggest that this a reasonably laterally extensive deposit close to Hughenden Airport, and has generally a low fines content (see Figure 7-7, and Photographs 7-3 and 7-4). The majority of the deposit is sand with in parts sections of gravelly sand and sandy gravel. A particle size distribution was undertaken on a recovered sample from the pit and revealed the following :-

- Fines content – 3%
- Fine Grained Sand content – 2%
- Medium Grained Sand content – 25%
- Coarse Grained Sand content – 43%
- Gravel Size Clasts – 27%

A potential sand borrow pit close to the existing Alderley Bridge Crossing has been identified during the project specific geotechnical investigations. A particle size distribution was undertaken on a recovered disturbed bulk sample from test pit HRTP03 and revealed the following :-

- Fines content – 16%
- Fine Grained Sand content – 45%
- Medium Grained Sand content – 36%
- Coarse Grained Sand content – 3%
- Gravel Size Clasts – 0%

Gradings were also obtained from Gromac on exploration sites to the north and northeast of Hughenden within the Flinders River floodplain. From the three particle size distributions undertaken the following grain sizes were revealed :-

- Fines content – 2%
- Fine Grained Sand content – 15-19%
- Medium Grained Sand content – 27-71%
- Coarse Grained Sand content – 7-47%
- Gravel Size Clasts – 1-9%

The collective gradings from Alluvial Sands are presented in Figure 7-7. It is likely that screening and possibly blending of different sand source locations may be required to achieve the appropriate grading for fine dam filter and concrete.

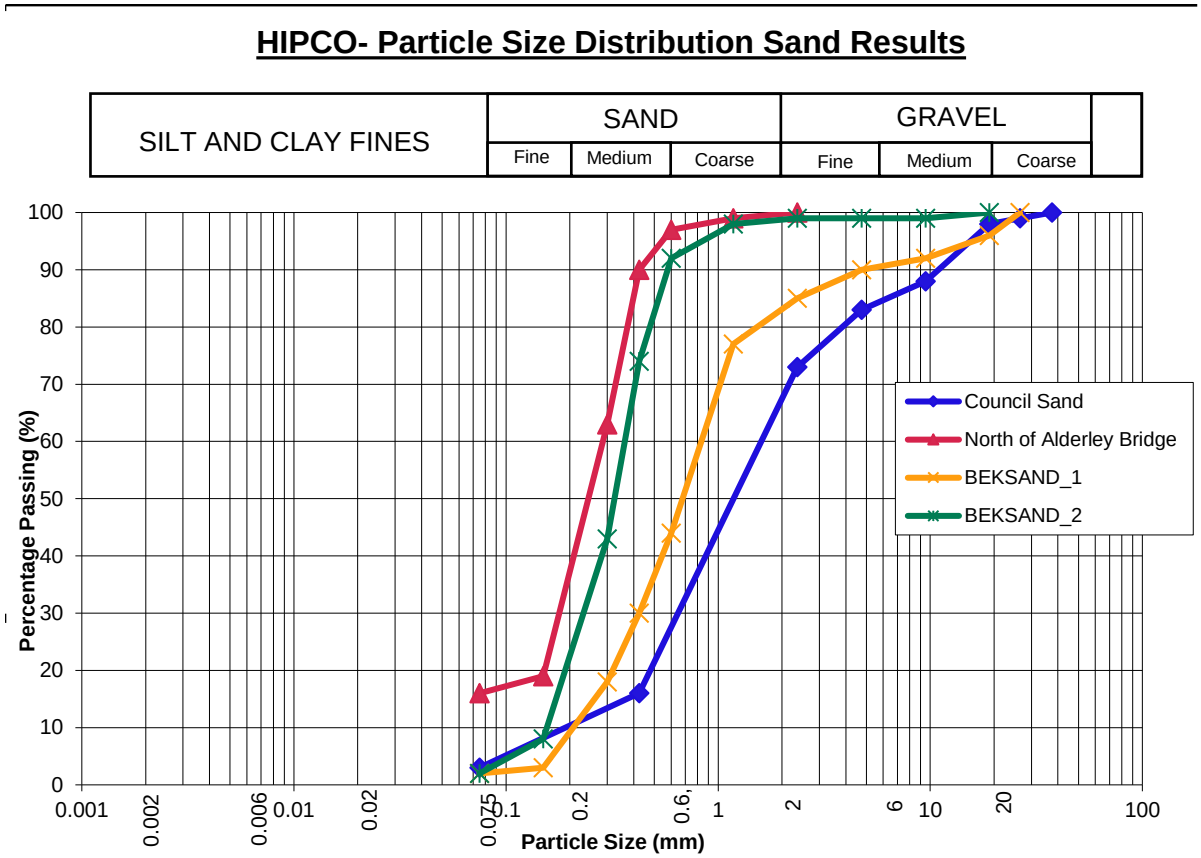


Figure 7-7: Collective Gradings from Flinders River Alluvial Sands

Photograph 7-3:: Council Sand & Gravel Pit Torver Valley Road

Photograph 7-4:: Council Sand & Gravel Pit Torver Valley Road – Stockpile

7.11.4 Clay Core Material

Ideally a suitable clay deposit for dam core use should have the following typical properties :-

- ≥35% Fines passing 0.075mm sieve
- Low plasticity clay (Liquid Limit ≤35%)
- Absence of any reactive minerals such as Montmorillonite (Smectite) or Halloysite (X-Ray Diffraction Analysis)
- Linear Shrinkage ≤7.5%
- Shrink/Swell Index <2%
- Emerson Class No. ≥3
- 'Non-Dispersive' classification (Pinhole Dispersion)
- Dispersion Index ≤30%
- Permeability ≥ 1x 10^{-9} m/s
- Maximum Dry Density 98%, Optimum Moisture Content -1 to +3%

Discussions with Engeny has indicated that their suggested clay borrow area within the PBC Saego Dam site was based on the limited laboratory testing on recovered samples from a relatively small test pit investigation campaign.

From the limited laboratory testing undertaken on alluvial clay materials during the PBC these soils in themselves may not be appropriate sources of clay core materials without blending or some form of additives, or be catered for in an engineered solution by the application of appropriate dam filters to minimise the potential for internal erosion/piping failure.

By their nature the residual black reactive soils formed by the weathering of the Flood Basalt lava flows, even after being transported by colluvial and then fluvial processes, are still likely to have and intolerable high shrink-swell potential and for this reason they are excluded from any other assessment for possible dam clay sources.

The use of the basal Rolling Downs Group shales or mudstones as a possible clay core source (through crushing) may be applicable, however present information suggests that these materials may have a sodic nature, and require either considerable processing and moisture conditioning, as well as the perhaps the use of lime dosing.

It is obvious from the present knowledge base that the occurrence and identification of a suitable quality clay (not requiring major reprocessing or amelioration) of sufficient volume for earthen dam core embankments, may be difficult to find in the area. Laboratory test results (see Tables 5-15 to 5-17) from both the Holocene and Pleistocene Alluvial Clays, Residual Soils and Extremely to Highly Weathered sedimentary rocks have revealed in their natural untreated state these clay dominated soils are both reactive, erosive and dispersive.

Clays of this type have been successfully used as dam cores in the past and seminal literature also supports the use of highly plastic materials (see Figure 7-8), but with suitable dam filters designed to aid with the drainage of seepages through dam embankments.

For this project a suite of laboratory testing was undertaken on the alluvial and weathered sedimentary sequences by the addition of lime in order to establish if improved properties of these clays could be ascertained. Based on lime demand tests undertaken on natural/untreated samples of the same materials, it was clear that in general no increase in pH (and therefore improvement in physical properties of the natural soils) occurred beyond the addition of either 1 or 2% hydrated lime (Sibelco source). Therefore, the additional laboratory testing campaign was based on these 2% additions of hydrated lime. After lime addition the samples were allowed to cure at room temperature for 2 days before being tested.

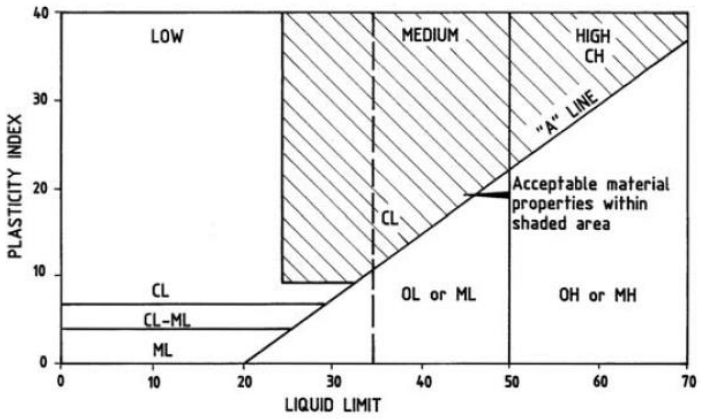


Figure 14.1. An example of specification of Atterberg limits for earthfill.

Figure 7-8: Potential Clay Dominated Plasticity Chart Dam Core Materials (Fell et al, 2015)

The results of the amelioration suite of tests of the alluvial and weathered rock samples indicate the following main summary points :-

- Soil improvements were obtained and there was no significant improvement in properties from 1 to 2% hydrated lime additives, and in some cases a detrimental effect was experienced with 2% additives. All samples tested were completely erosion resistant (ND2), recompacted samples had permeabilities between E-08 & -09 m/s. What the lime additive results (irrespective of 1 or 2%) also show are the following :
    - Plasticity is still generally high, but the lime addition has resulted in most of the results plotting in the MH sector of the plasticity chart, ie., a high plasticity SILT rather than the field description and no additive results indicating a silty CLAY – 1% addition gave a 50/50 spilt between a clay and a silt based on Atterberg Limits and 2% lime was all silts;
    - Linear Shrinkage has been reduced, but all test results are still over the 7.5% threshold and in most cases >11.5% with many samples showing signs of cracking; and
    - The Electrical Conductivity tests results all show that the samples are still potentially sodic or unstable in nature.

The conclusions that can be drawn from the investigations suggest that the potential clay dam core sources within practical distances from the proposed dam are :-

- Natural clay sources within the proposed dam inundation areas and its immediate environs are of sufficient quantities for the required Saego Dam construction, however in an untreated state they are not suitable for standard earth embankment dams;

- Suitable blending with other locally available no reactive and granular dominated soils could be undertaken to produce a suitable clay core material. However, this would involve a probable significant trial to prove this was technically possible and at an acceptable cost;

- The use of suitable dam filters can be employed to cater for the use of high plasticity clays as dam clay cores;

- There may be merit in adding 1% hydrated lime to these clay soil types as the pinhole results are encouraging, however perhaps wider filters may be still warranted because of the tendency for high linear shrinkage and the potential unstable nature of the materials – perhaps the greatest risk would be in the higher parts of the dam core where dam water levels could go up and down (depending on supply requirements/flood inputs) providing an avenue for shrinkage cracks to develop and therefore possible piping failure probabilities increasing accordingly. It may be the case that a cost-benefit assessment based on the cost of adding 1% lime and the extra time and costs associated with doing this in terms of plant/fuel etc. , against building wider filters without amelioration, may be warranted; and

- If blending or the availability/cost of amelioration is not deemed acceptable then potentially a grout enriched or concrete upstream facing (CFRD) to the proposed embankment dam could be considered, or a rockfill CFRD dam or a concrete dam (mass or Roller Compacted (RCC)). However, these forms of dams may be considerably more expensive than earth embankment types and require deeper foundations to perhaps weathered or competent rockhead, which in itself may render this project economically unfeasible.

7.11.5 Construction Material Sourcing Conclusions

From the present discussions the following overarching points of significance have been interpreted:-

- Sufficient sand and gravel deposits for project construction needs have a good likelihood to be available within the region at probable economic processing and haulage costs; coarse filter materials (sandy gravels) could also be generated from specific hard rock crushing/processing circuits;

- The local basalt can be used for a variety of aggregate sources including rockfill, rip-rap, drainage, gabion, road base and concrete, although it may be variable in composition and thickness over the district; there are no issues with the volume available of these possible aggregates in terms of proximity to any potential scheme options;

- The location of suitable quality and quantity of low plasticity clay deposits for possible dam clay core sources within the widespread alluvial deposits, may be problematic to identify. Upper Holocene and Lower Pleistocene clays are both reactive, dispersive and sodic in nature. The weathered basal sedimentary sequence of siltstones/sandstones/shales could be a possible source of clay for dam cores if processed, however their sodic & dispersive nature does not lend it to advantageous clay core sources. To alleviate these presently interpreted poor clay core sources may require appropriate amelioration with lime, use of more substantial dam filters, blending with inert local materials, and/or consideration of concrete or grout enriched dam upstream faces; in addition, careful moisture control of placed core materials will be required; the present base case for design and associated CAPEX estimation is based on the use of untreated near surface alluvial and residual soil clays as dam clay core (Type1A) & Earthfill (Type 1B) materials, with wider dam filters being employed;

- A number of options exist for local plant and wet hire either for the DBC investigations or for a potential Tier 1 or 2 Management Contractor for the D & C contract if the project was to go forward;

- Local concrete batching is available but may not meet a D & C contract needs.

The most cost-effective approach for construction material sourcing relies on the least possible processing required and the requirement for no additives/treatments if possible. For this project it is possible that additives and secondary processing may be required for clay core material and possible certain types of concrete.

Based on a review of the available published sources, discussions with the Flinders Shire Council, local earthworks contractors and local quarry operators & their consultants, Figure IS362200_G_006 (Appendix A) sets out potential construction material sources they may be utilised for the proposed scheme. It is felt that there are sufficient volumes of these construction materials within economic distances for any scheme option within proposed area in and around Hughenden within an approximate 50km radius.

7.12 Distribution Network Alignment Considerations

7.12.1 Pipelines

Two pipeline sections will be required to feed irrigation water from the dam outlet to the distribution network. An approximate 460m length from the outlet to the northside of the Flinders River and then approximately a 5km alignment from the southside of the river to the distribution network. The pipeline will be HDPE at a 1200mm diameter, with a minimal excavation depth 0.70 mbgl and will be installed on a sand or rockfill bedding material.

The decision to install a pipeline over an open channel, from the south of the Flinders River to the distribution network, was based primarily on the following factors :-

- The geomorphology of the alignment is an alluvial floodplain with numerous evidence of abandoned channels and shoots (see Figure 7-9). In addition, hydraulic modelling has shown that in a large flood event there is a potential for the Flinders to switch course and flow through this current floodplain;

- The inherent potentially sodic nature of the residual soils, would require significant channel lining; and

- For water security reasons.

The Distribution Network itself will also deliver water via a pipeline system of approximately 1200 to 630mm HDPE pipe, installed at a minimum depth of between 0.60 to 0.70mbgl.

The following sections provide geotechnical considerations for the proposed pipeline.

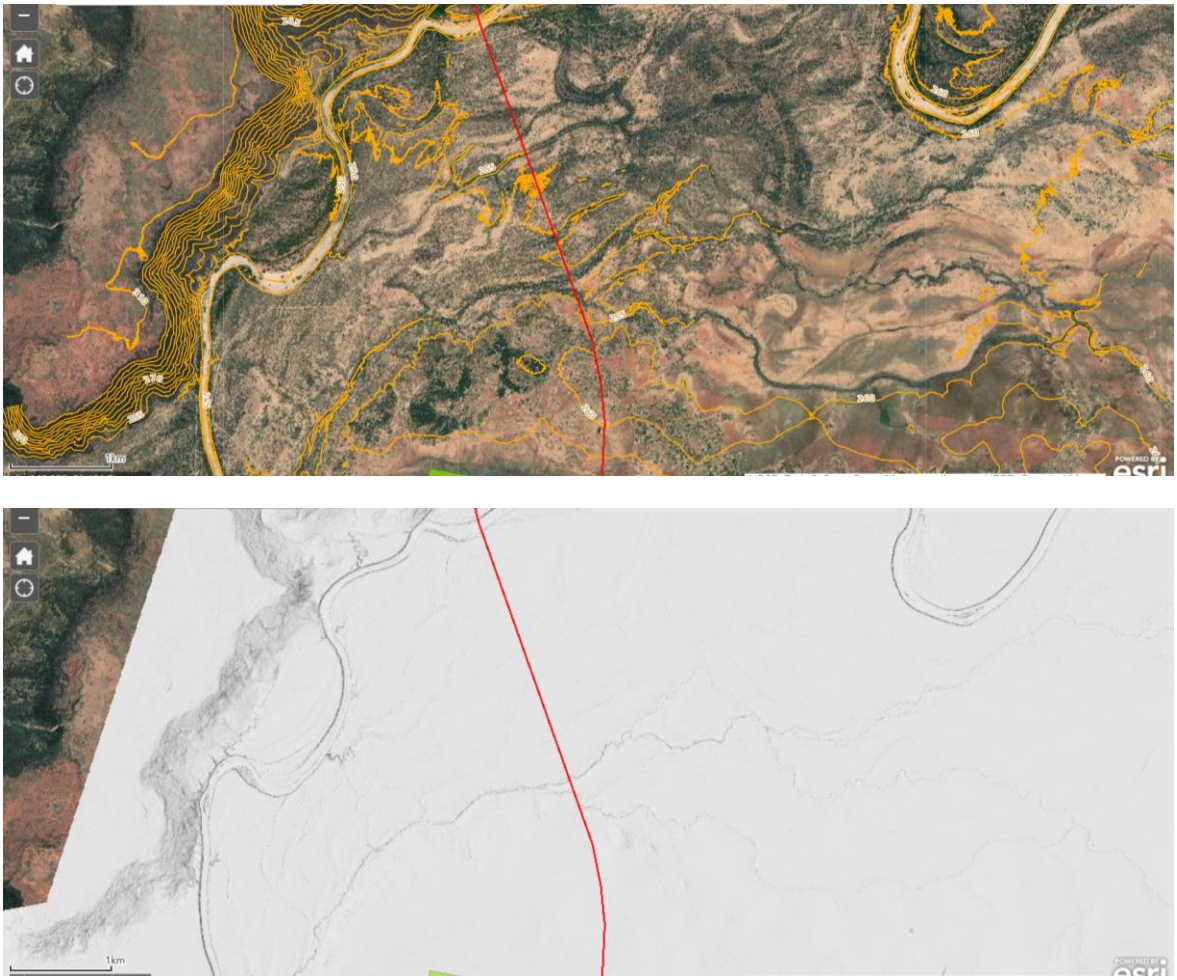


Figure 7-9: Floodplain Geomorphology of the Pipeline Alignment South of Flinders River

7.12.2 Pipeline Excavatability

Pipeline excavatability is often a subjective viewpoint, with either different interpretations of similar criteria used and/or differing approaches employed.

As indicated in Section 5.1 a test pit campaign was planned for the Dunluce property including the main southern pipeline alignment, but due to the COVID-19 pandemic was eventually cancelled. However, based on the interpreted ground conditions (see Section 6.4) it is likely that for the required excavation depths the pipelines will be installed by open trenching techniques with the use of free digging by large excavators.

Taking into consideration the possible variable alluvial soil ground conditions, Figure 7-10 sets out the general approach of the open excavated trench. This design incorporates much deeper installation than presently envisaged and includes three single batter sections (laid back on 1:1) with intermediate benches. The stability of this design may require effective dewatering of the excavation.

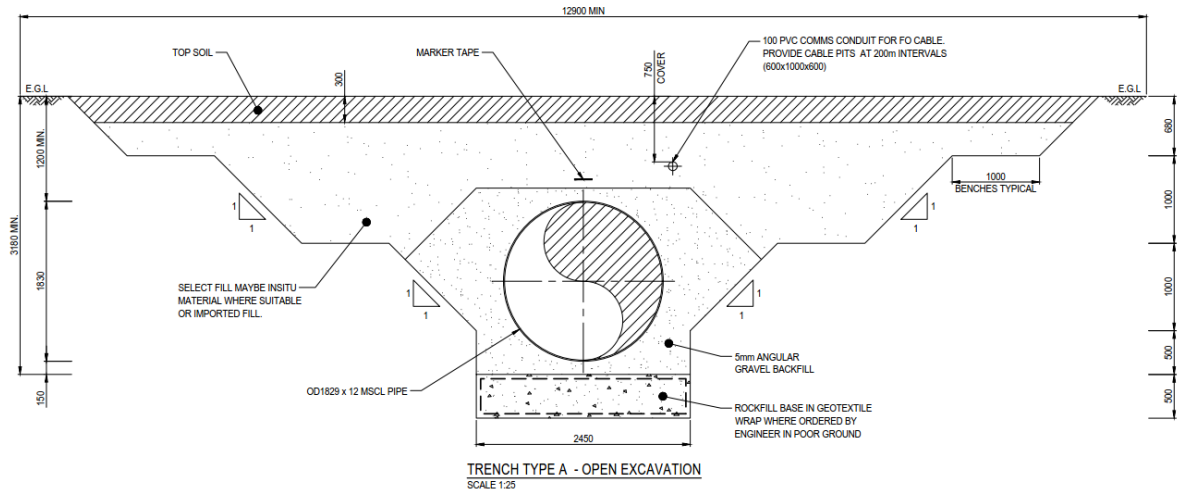


Figure 7-10: Typical Open Trench Pipeline Installation Excavation

It is envisaged that the pipeline excavations would be over a number of fronts with a limited open trench length to reduce overbreak and a feasible effective dewatering strategy.

The use of a large pipeline trenching machine (see Photograph 7-5) could be considered but may be cost prohibitive.

Photograph 7-5: Typical Trenching Plant for Open Trench Installations for Large Diameter Pipelines

For the Distribution Network pipeline installation, a good idea of the likely ground conditions that may be encountered can be gauged from Council borrow pits that exist adjacent to the public roads network on the current Dunluce property (see Photograph 7-6). Again, due to the depth of the proposed excavation required, free digging conditions for excavators are the most likely method of pipeline installation.

Photograph 7-6: Council Borrow Pit of Residual Soils with Gypsum Bands, Dunluce Property

7.12.3 Potential Pipeline Trench Temporary Works

Depending on the time of year that the proposed pipelines are to be installed, perched groundwater tables could be encountered in open trench excavations outside the immediate Flinders River channel. Within the river channel, if an open trenching methodology is to be utilised then effective dewatering will be critical in both achieving acceptable temporary stability, and the overall construction progress.

These excavations would probably require a well point system, potentially coupled with local sump pumping, to efficiently draw down the groundwater table. The derivation of an effective well point dewatering system may rely on trials due to the variability of the soils within the proposed pipeline excavations.

It is assumed that the pump groundwater is not contaminated, and it can be discharged into the existing surface courses through a v-notch weir system to remove the required sediment load.

It could be the case that on installation of the bedding material the dewatering could be backed off to allow the groundwater table to recharge and rise within the excavation. This would provide the potential advantages of:

- Providing a self-densification of hydraulically placed granular backfill material; and
- Potentially allowing the easier installation of stringed pipe sections by towing within the trench.

It is believed that even if the pipeline excavations are benched, that some overbreak and slope instability may still occur within the alluvial soils. In this case the use of sheet piling may also be required.

7.12.4 Potential Pipeline Buoyancy

Buoyancy of pipelines within the alluvial floodplains with high groundwater tables can be a significant issue, but is dependent on also the depth of pipe below ground level, backfilling material type and specification and the

actual pipe material type. The use of steel pipes negates the need for such restraints; however, the proposed HDPE pipe could require to be restrained at section valves. Here the alternatives that could be used include:

a) Concrete linings (see Photograph 7-7);

b) The use of sand bagging; or

c) The use of geotextile wrapping (see Figure 7-11).

7.12.5 Potential Scour Considerations

Within the pipeline traverses of the alluvial floodplains (and potentially within the Siphon Crossing of the Flinders River if this is to be installed by open trench), an upper protection layer of rip-rap rock over the pipeline crown has been designed to minimise any detrimental effects of scour from flood events (see Figure 7-12).

This is seen as the most cost effective and environmentally sensitive approach to potential scour issues. The other alternative considered was that of concrete lining (see Photograph 7-7), but this was dismissed generally based on cost impacts.

Photograph 7-7: Typical Concrete Lined Steel Pipes for Scour Protection Purposes

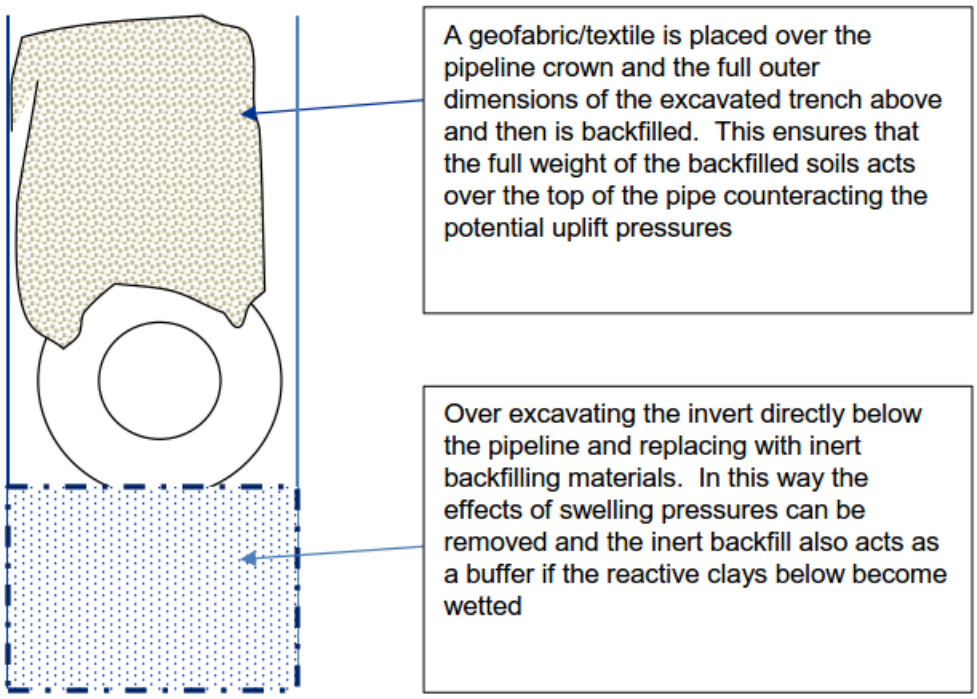


Figure 7-11: **Potential use of Geotextile Wrapping/Soil Pillows to Restrain Pipeline Uplift**

7.12.6 Dealing with Reactive and Sodic Soil Potential Issues

Laboratory testing of the alluvial and residual soils indicates that reactive cracking clays could be encountered in this area, which may have a high potential for shrink-swell, as well as being extremely difficult for plant to traffic over when wet. Again, moisture control of these soils will be extremely important for successful backfilling with possibly the need for the addition of lime. The high swelling pressures that could generate on wetting of these soils beneath the pipeline invert (see Photograph 7-8), could require either a deeper dig out and replacement with inert soils or the use of sandbags, concrete linings or geotextile/fabric wrapping/the use of soil pillows (see Figure 7-11). This situation could be worse for the Distribution Network, because of the relatively shallow depth of installation and being of a generally smaller diameter plastic type.

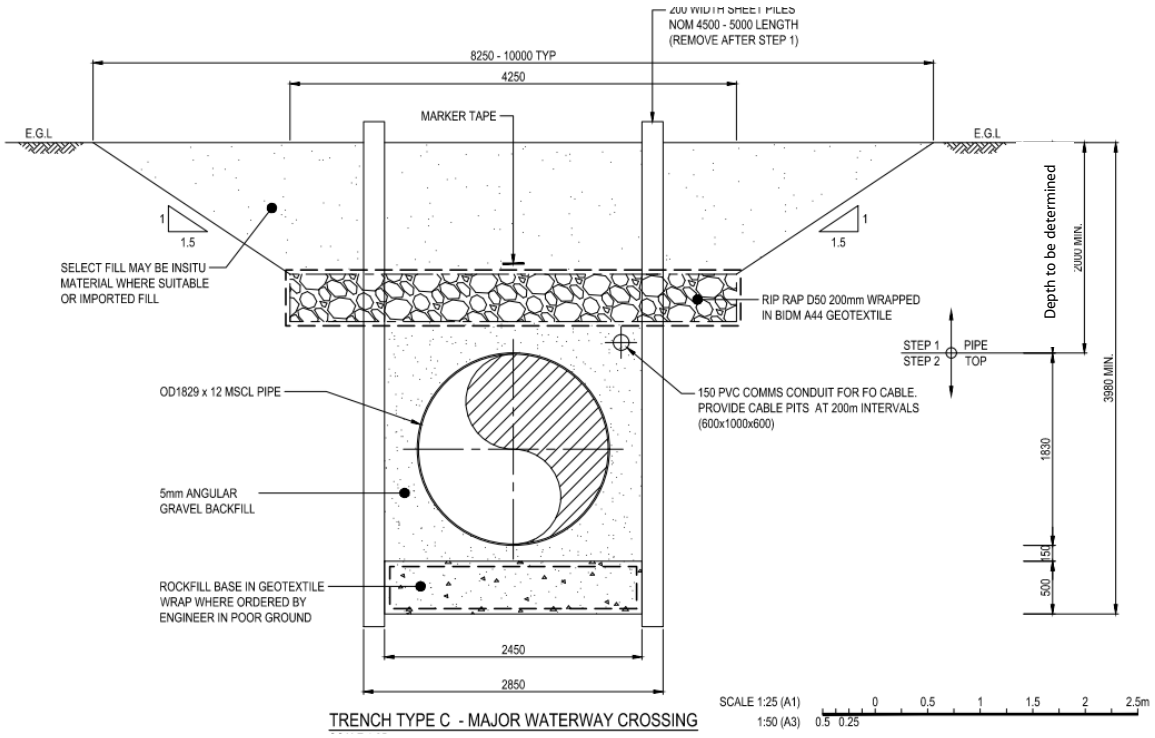


Figure 7-12: Potential Open Trench Rip-Rap projection of Pipelines within Alluvial Floodplains

Section 2.2 & Figure 6-4 indicate the potential high possibility of encountering sodic soils over the proposed pipeline alignments. The effects of such sodic dispersive potential on pipeline temporary and permanent works, as well as the possible measures required to reuse these type of soils for fill materials are significant considerations (see Photographs 7-9 to 7-10).

Photograph 7-8: Reactive Clay (Cracked) at a Pipeline Invert Level

Photograph 7-9: Piping of Sodic Soils in an Open Trench Pipeline Excavation

Photograph 7-10: Typical Tunnel Erosion of Over Pipeline Backfilled Alignments

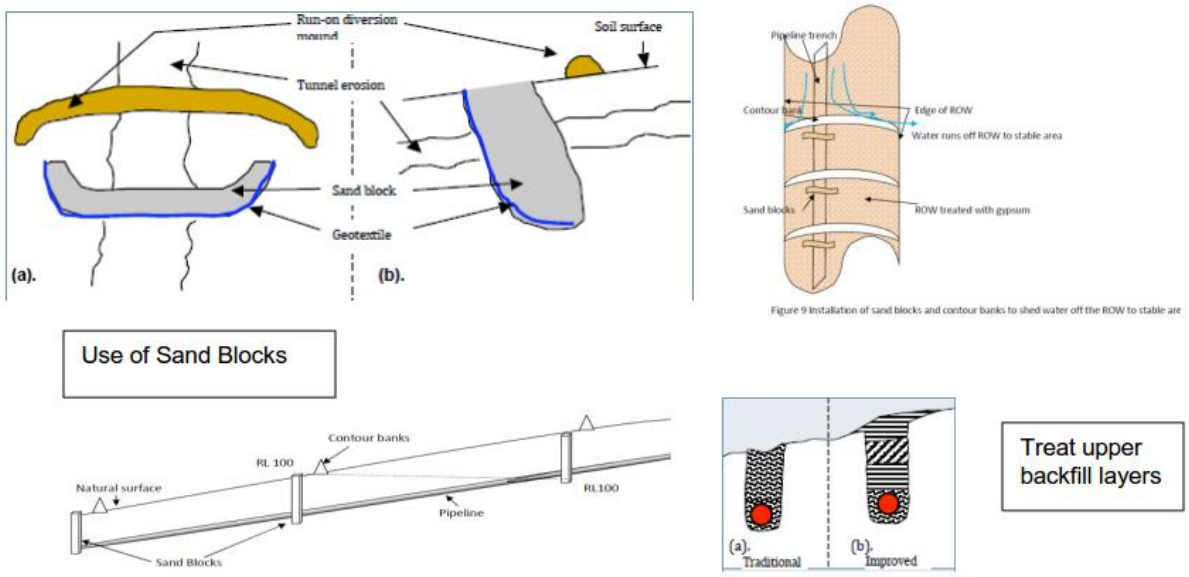


Figure 7-13: **Potential Control Measures for Pipeline in Sodic Soils**

Current indications from Jacobs observations, along and within the vicinity, of the proposed pipeline alignment, as well as laboratory test results from the site specific investigation, suggest that exposed soils may have a dispersive characteristic.

If the type soils were considered as reuse within project works a variety of approaches may be required for this to be enabled, including:

- Additional drainage control measures;
- Using a non-reactive covering over the surface of the pipeline;
- For cohesive dominated backfill soils treat the upper 300-500mm by mixing with potentially between 1 to 2% by weight of hydrated lime - the calcium will replace the sodium minerals and improve soil permeability and allow water to pass through this layer without erosion occurring – in addition the mixed zone needs to be carefully compacted;
- For granular dominated backfill soils install a geotextile 300-500mm below ground surface and then replace with the excavated soil – the geotextile will act as a barrier in this case; and
- Installing sand blocks or barriers across/around proven tunnel prone areas (see Figure 7-13).

To minimise the potential effects of piping erosion within the exposed pipeline trench it is recommended that backfilling to ground level should occur as soon as practically possible. This approach should fit in with the proposed multiple fronts and short lengths of trench excavation, allied with dewatering, and backfilling strategy previously indicated.

7.12.7 Ground Aggressivity

Aggressive (acidic) soil and groundwater conditions in terms of fabric attack to buried concrete reinforcement or steel pipes may have a significant impact on cost and durability overtime. If such conditions exist then concrete

cover to reinforcement may be required to be increased and or specific concrete mixes may be required. In addition, sacrificial thicknesses for the steel pipeline or cathodic protection could be warranted.

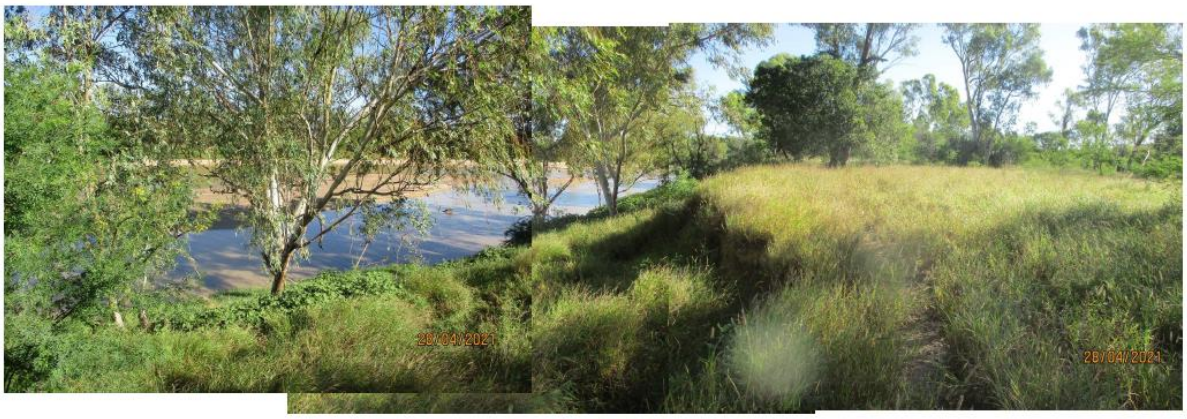
Field and laboratory testing of groundwater, local dam and river water, as well as laboratory testing of pH, sulphate and chloride on various soil samples from differing depths and origins, from the site specific geotechnical investigations, suggest moderate and non-aggressive conditions for the steel pipe and buried concrete & reinforcement respectively. Indications of infrastructure durability within the vicinity of the project do not suggest that corrosion, and therefore the presence of aggressive subsurface conditions, are a significant risk. Therefore, at present no additional measures for dealing with aggressive ground have been considered.

7.12.8 Siphon Crossing of the Flinders River

In order to deliver irrigation volumes from the outlet of the dam on the northern side of the Flinders River to the Distribution Network on the current Dunluce property, on the south side of the River, an approximate 120m siphon crossing under the River is required.

At the present time this siphon is composed of 3 x 1200 mm diam. polyethylene pipes to be installed at least 3m below the current riverbed.

Current views of the north and south bank river, as well as the riverbed are provided in Photographs 7-11 & 7-12.



Photograph 7-11: Proposed Siphon Crossing - North Bank of Flinders River

Current imagery drilled borehole locations and hillshade imagery for the proposed Siphon crossing are presented in Figure 7-14 below.

Walkover survey of the crossing point along with hillshade imagery indicates that the south side of the proposed siphon may lie within an old meander or shoot of the former course of the river. In addition, from observations on site it was obvious that both banks were composed of a composite series of sand dominated terraces.

Drilling on both sides of the river revealed a thick sequence of Holocene (interbedded sand and silt dominated strata) & Pleistocene Alluvium (gravelly clays) overlying H-MW siltstone varying between 17.90 (north bank) to 22.70mbgl (south bank). An interpreted geological section over the proposed siphon crossing is presented as Drawing No. IS362200-0000-CG-DG-0041 in Appendix A.

Photograph 7-12: Proposed Siphon Crossing – South Bank & Riverbed

The required depth of the pipeline crown level will be dependent on the required hydraulics of the delivery system to the Distribution Network, as well as the topography and geomorphology of the river & its immediate environs, the past and potential riverbed erosion rates and the possibility of future horizontal river channel changes.

Available imagery in the immediate adjacent areas to the proposed crossing (see Figure 7-15) does show a number of abandoned channels and meanders on the south side of the river.

There remains the risk that during the design life of the project that during flood events further horizontal changes in the river channel course may occur, while significant riverbed erosion is seen as unlikely. Design of the crossing and the adjacent pipelines should take this into consideration.

In terms of construction both open/trench excavations could be considered, as well as pipe bridges and trenchless methods such as Horizontal Directional Drilling (HDD) or pipe jacking.

The probable deep trenching required (taking account of future erosion/horizontal channel changes) within the upper loose to medium dense SANDS and SILTS will probably require the use of temporary sheet piling to support the excavation. Due the probable depth of the excavation wailers/props/jacks are likely to be required, which will add constructability difficulties (cost and time increases) to the pipeline installation. Trenching within the riverbed would also probably require sheet pile support, but also likely dewatering. The upper sections of the excavation will be required to be backfilled with appropriate rip-rap or rockfill as an erosional buffer zone. A possible arrangement/design for an open trench installation methodology is provided in Figure 7-12.

Open trenching has the added risk of flash flooding and may only be safely undertaken during the dry season, where river flows will be reduced. It is likely that even with these difficulties and risks open trenching is probably the cheapest and fastest form of pipeline installation methodology.

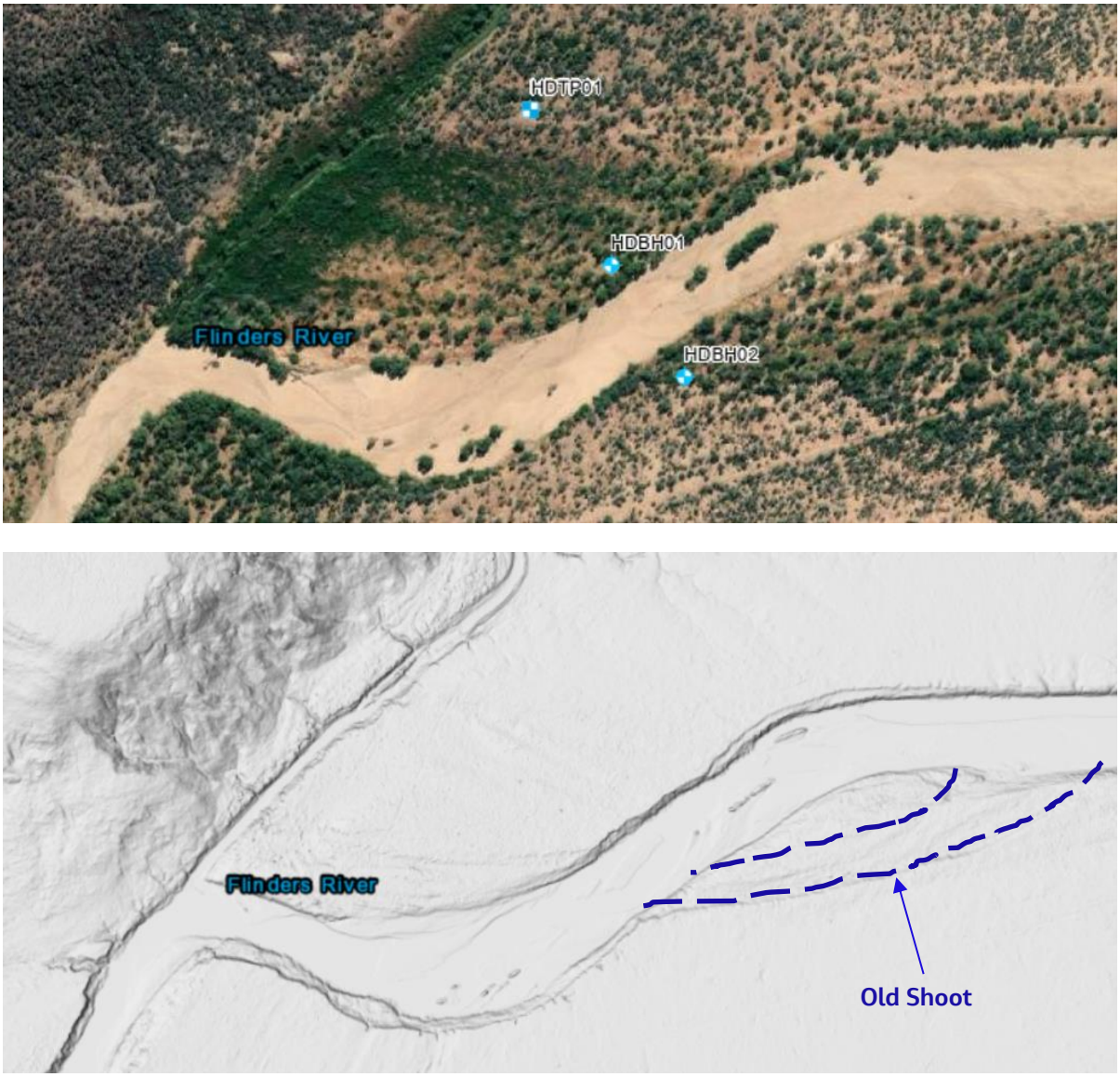


Figure 7-14: Proposed Siphon Crossing – Current Imagery & Hillshade

Pipe bridges could be a viable installation method, but risks of possible flood levels and/or dynamic lateral loading and scour on supporting columns would need to have careful consideration. Flooding risks during construction would be similar to an open trench method, while costs are likely to be higher.

HDD would require a significantly greater length of pipeline to reach the required crown levels under the river channel and to connect to the delivery pipelines either side of the riverbanks. Pipe jacking could reduce the lengths of pipelines required, but temporary works in terms of construction of appropriate deep launch and receival shafts in the inherent loose to medium dense near surface alluvium, with a high groundwater table, would require a considerable effort. The advantage of trenchless installation methods would be the reduced risks of flooding during construction and in the operational phase, however it is likely that would be considerably more expensive than open trenching methods.

At the present time HDD has been chosen as the preferred method of pipeline installation for the Siphon Crossing.

Figure 7-15: Proposed Siphon Crossing – Adjacent Abandoned Channels & Meanders

7.12.9 Reuse of Excavated Material and Pipeline Backfill Materials

At present it is interpreted that the vast majority of excavated materials will be used as pipeline backfill materials for the following reasons:

- Sustainability.
- Cost; and
- Construction program.

Secondary processing (drying out) of excavated materials below the groundwater, as a result of encountering perched water tables within the alluvial floodplains, could be negated by installing pipelines within the dry winter season, and by use of a trenchless method for the Siphon Crossing.

Bedding sands for the pipeline alignments are likely to be able to be sourced from local excavations, or from borrow pits within economically viable haulage distances to the work sites.

The issues of reactive & sodic soils previously detailed will need to be considered in the backfill material specifications. It may be the case that a higher degree of maintenance will be required for the backfilled trench sections of the pipeline.

7.12.10 Pumping Station Structural Foundations

Pumping stations will be required at the dam outlet and within the Distribution Network to boost flows to service all the required agriculture lots.

Potential pumping station layouts/form are presented in Figures 7-16 & 7-17.

Due to the probability of encountering relatively deep rockhead at both of the proposed pumping station locations and the likelihood of improving density/shear strength of both alluvial and weathered rock sequences with depth (as indicated by the project specific investigations), 900mm diameter concrete piles driven provisionally to 15mbgl, have been considered as appropriate for both acceptable load and associated settlement criteria.

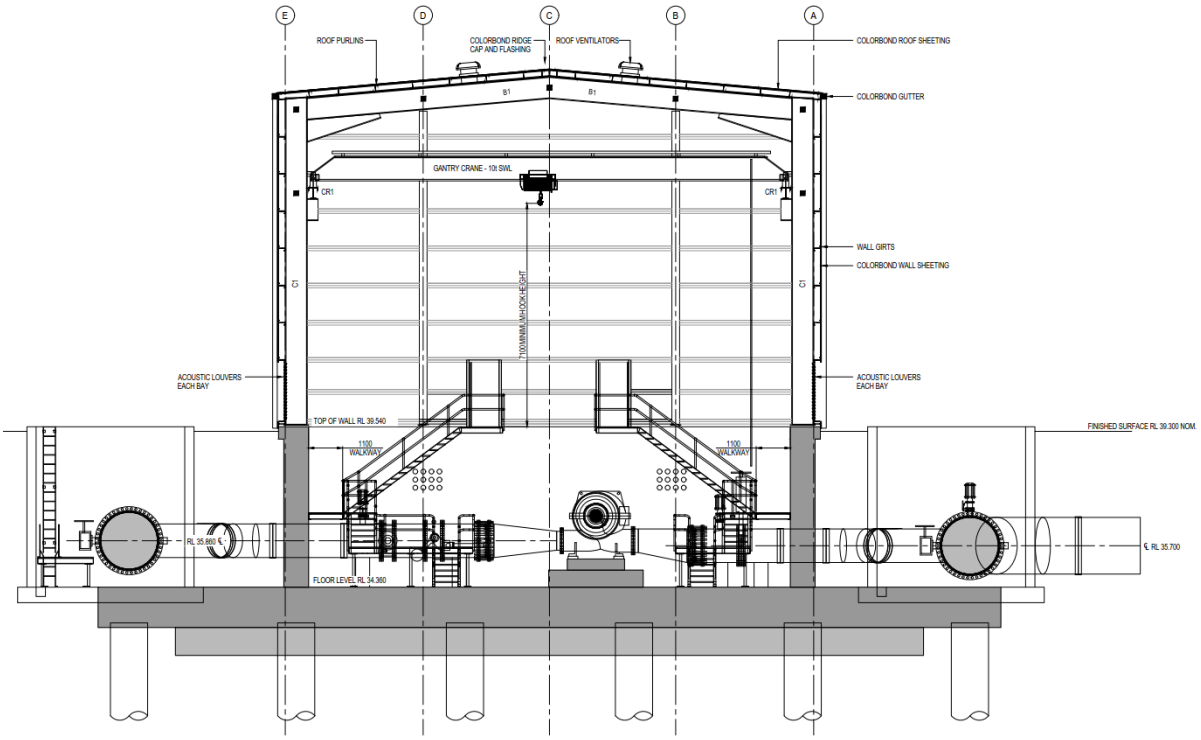


Figure 7-16: **General Layout, Elevation & Foundation Details of Proposed Pumping Stations**

Figure 7-17: Potential Pumping Station Form

7.13 Road Related Design

As part of the proposed scheme development a limited number of existing unsealed roads will be required to be realign, or reapportioned, as well as the construction of new unsealed access roads.

During construction a significant earthworks fleet, as well as cranage, concrete trucks and light vehicles will require all weather access to the different scheme elements.

The majority of the upper soils of the site area are composed of reactive soils derived from the overlying basalts. In wet weather trafficking on these road surfaces is problematic, with many areas immediately unusable for days after rain events.

Construction of both haul roads and permanent access on reactive soils is challenging to provide all weather access. The shrink-swell behaviour and alternative cracking in drier periods provides subgrade CBR values typically < 3%. In these circumstances sealed road/pavement design generally follows a "dig-out-and-replace" or stabilisation (with typically lime, but increasingly cement) strategy for poor subgrades of this type.

Table 3.5 provides some idea of the potential subgrade CBR's that could be encountered, however many of these test results are from alluvium immediately adjacent to the Flinders River (excavated for the new Alderley Road Bridge Crossing), with only one (TP05 @ 1.0-1.5mbgl, high plasticity CLAY with a CBR = 1%) perhaps reflecting the more typical subgrade conditions outside the floodplains.

Noting the usage of the current roads, and the potential operational phase access requirements, improving wholesale subgrades of existing or new roads may not be deemed appropriate, especially on a cost-benefit basis.

It is more likely that both construction and operational access is to be provided by sheeting or the provision of a sand sub-base, with a gravel sub-base & wearing course pavement structure akin to a mine lease haul/access road arrangement. The base materials should be ≤ 250mm individual clast sizes, with lift heights ≤300mm, with a minimum of a 200mm thick wearing course (NRME, 2019). Cover to the subgrade will depend on the CBR value and the maximum axle weight of the plant fleet (see Figure 7-18), however for subgrades with CBR's <3 thicknesses may increase.

Road level heights will be a point of determination for the project, but perhaps levels able to cope with a Q10 flood are appropriate. The proposed project basalt quarry(s) should be able to provide suitable materials in the required volumes to construct the proposed road network.

At major creek crossings the use of concrete floodways should be considered to provide permanent and stable crossing points for these critical locations.

The construction of the new weir structure will result in overtopping of the new Alderley Bridge Crossing. Jacobs assessment of the option for raising the bridge soffit levels suggests that this is not practical. This means that the existing bridge access over the Flinders River will have to be reapportioned on the crest of the new weir.

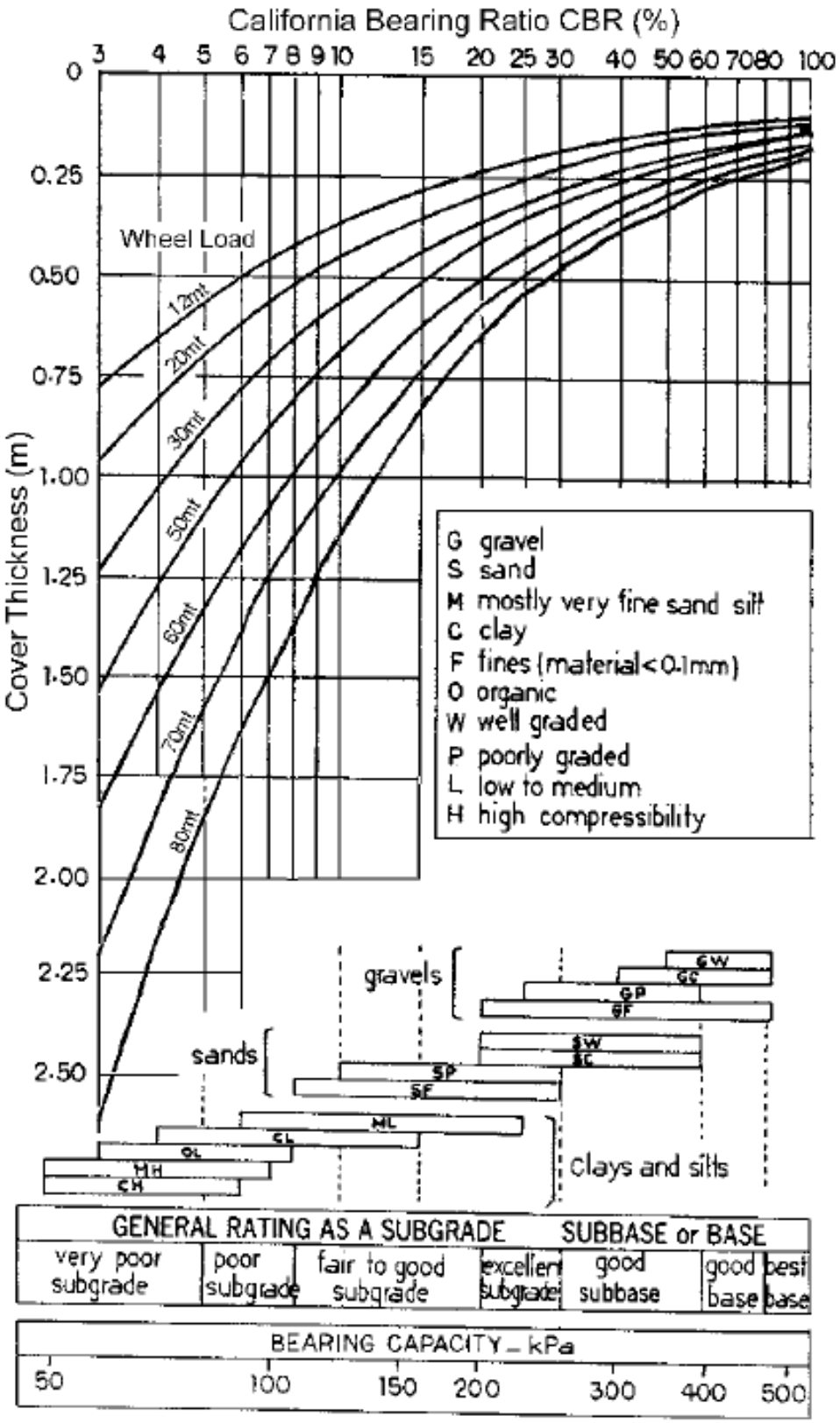


Figure 7-18: Cover to Subgrade for Construction Roads

8. Key Risks and Mitigations

Table 8.1 below sets out the major geotechnical risks and associated mitigating measures that should be considered in the project risk registers, cost estimation for the proposed works and for the next phase of this project.

Table 8-1: Key Geotechnical Risks & Mitigating Measures

Area/Work Scope	Risk	Mitigation Measure
Saego Dam Wall Foundation Founding Layer	The present design requires removal of 2.5m of existing upper alluvial/residual soils to found the dam. However, other areas may have deeper softer/less dense layers that could exist, and either be required to be removed or ground improved.	Undertake additional intrusive geotechnical investigations, along the dam wall/abutments to define where the competent foundation level is to be encountered.
Saego Dam Embankment Stability	There remains the risk of potential main dam wall stability from not only changes in the impounded storage levels (including Rapid Drawdown), but also from flooding events of the Flinders River on the downstream face (again including Rapid Drawdown).	Undertake appropriate sensitivity analysis of the proposed dam wall sections to allow for worst case plausible water and flood event levels, velocities & periods, to prove the appropriate dam wall dimensions & configurations.
Saego Dam Wall Foundation Grout Curtain/Slurry Trench	At present a proposed grout curtain/slurry trench has been defined based on available subsurface information. This impermeable barrier could be of insufficient extent (vertically and horizontally) or installed inappropriately such it becomes ineffective and allows excess seepage below the dam foundation.	Undertake further geotechnical investigations to define more accurately the vertical and horizontal extent of the high permeability alluvial soil and /or rock sequence beneath the dam footprint. During construction install additional grout holes and undertake a more stringent quality assurance program.
Liquefaction Potential below Saego Dam Wall	Present seismic hazard analysis indicates a very low potential for liquefaction to occur below the proposed dam wall, however question marks still exist on what the most appropriate Vs30 value should be used for these assessments.	Undertake site specific crosshole geophysical shear wave velocity surveys during the Detailed Design phase of the project to confirm current seismic hazard assessments.
Clay Core Sourcing	Both a site and adjacent regional study has been undertaken to identify and classify the character of potential soil and weathered sedimentary rocks as clay core sources for the proposed dam wall. Adequate volumes of cohesively dominated materials are available within economically practical haulage distances and within the inundation area, however their properties are not conducive for clay core sources without appropriate amelioration or perhaps blending. Lime additives may help but do not solve all the adverse properties of the natural materials.	Undertake additional and further investigations to confirm consistency of potential clay core materials character. Carryout trial blending mixes of suitable quality and available local materials with alluvial clays to identify optimum earthworks specification. Consider the use of other additives (eg., cement or bentonite) for inferior materials and carryout appropriate trials. Construct dam cross sections with thicker filter layers to minimise the effects of saturating clay core. Consider the use of CFRD or grout enriched upstream face or an asphaltic core.
Construction Material Sourcing	At present a limited number of actual sand and gravel source locations have been sampled (for potential filter sources). Other areas within the Flinders floodplain have been identified within the region, but none have been sampled or tested. There is still uncertainty in the volume and quality of the potential filter materials available within a practical haul distance and how much	Undertake additional and further investigations to confirm identified sites of additional potential filter sources within economic haulage distances from the site. Identify potential actual quarry sites in local basalt mesas and undertake an appropriate intrusive investigation & laboratory testing suite to confirm

Area/Work Scope	Risk	Mitigation Measure
	screening may be required to manufacture suitable materials. It is presently planned to establish at least one new local quarry within the basalt mesas close to the proposed scheme. Evidence from local quarries in the local region suggest a high variability of potential basalt rocks, which could require segregation of quarry development along certain trends and increase actual quarry extraction areas required to be identified by an appropriate DA.	quality & quantity of potential rockfill, rip-rap, drainage rock, concrete aggregate and road base available, as well as the potential extent of the quarry extraction area for DA.
Spillway Foundation Stability	The Spillway is a mass concrete structure that is required to be founded on a suitable stiff formation, that does not have a tendency for kinematic sliding failure. Weak founding surfaces that may have laterally extensive subhorizontal discontinuity sets could provide problematic ground conditions	Geotechnical investigations to date indicate that moderately weathered bedrock could be a viable founded surface, while kinematic analysis for a potential sliding failure mechanism are very low.
Spillway Base & Return Channel Erodibility	Based on visual observations of the area around the proposed Spillway, including walkover surveys of the potential Stilling Basin and Spillway Return Channel south to Back Valley Creek, zones of dispersive &/or erosive soils and weathered rock sequences are apparent. If these areas remained unprotected during the operation phase of the project during spilling events they may have a high probability of being eroded. Erosion overtime could lead to undermining of the Return Channel and the Spillway itself, as well as detrimental environmental excessive sediment loading to Back Valley Creek &/or the Flinders River	The Stilling Basin and the entire Spillway Return Channel to be protected by rip-rap material suitably sized for the appropriate flood case and return periods
Weir Foundation Construction	The construction of the proposed mass concrete weir will require a significant excavation within the Flinders River Channel. Temporary cut slope stability and an effective dewatering system will be required to be employed for health & safety and program surety. During the DBC no intrusive geotechnical investigations were undertaken over the proposed weir footprint within the River Channel.	Undertake appropriate intrusive geotechnical investigations over the proposed weir footprint within the River Channel to confirm weir structure founding depths and for appropriate design of temporary cut slopes and the need and scope for ground treatments.
Weir Structure Stability	The present interpretations of ground conditions, and associated foundation levels and designs, is primarily based on the project specific seismic geophysical survey correlated with out of channel and adjacent limited CPT's and one borehole, as well as a site wide assessment of rockmass discontinuities, and the historic Alderley Bridge Crossing investigations. The present founding levels in terms of bearing capacity, and associated additional measures to resist potential sliding may not be adequate &/or appropriate.	Undertake specific intrusive geotechnical investigations at the Weir to determine actual ground conditions and design appropriate foundation solutions
Diversion Channel Excavation	The maximum depth of cut required to form the Diversion Channel is approximately 12.6 mbgl, In this area a significant proportion of the excavation will be in	From the formulated ground model it is interpreted that the majority of the proposed cuttings can be effectively excavated by the use of standard plant

Area/Work Scope	Risk	Mitigation Measure
	slightly weathered bedrock, which could be difficult to excavate at the required and typical production rates.	(dozers, excavators and graders), with only the potential requirement for D10 &/or D11 with tyne rippers or excavators with hydraulic breakers for approximately 1.5 km of the alignment within the vicinity of the deepest cuttings. No blasting is currently interpreted to be required.
Diversion Channel Cut Stability	The required cuttings will be of varying height in varying soil to slightly weathered rock conditions. Slopes that are inappropriately designed and constructed have the risk of failing into the channel, not only requiring greater OPEX, but also creating a risk of not delivering flood waters to the dam.	Long term stability cut slope batter dimensions will be dependent on the encountered geology but based on the interpreted ground models the preliminary minimum cut slope designs provided have taken into consideration the variable ground conditions, as well as potential erodibility of the Wallumbilla Formation sequence when exposed.
Diversion Channel Cut Durability	On exposure soils may have a sodic or dispersive nature, while the interbedded siltstone and sandstone bedrock formation may well degrade overtime. Erosion and degradation may result in loss of channel dimension and could in themselves leading to cut slope instability, with similar consequences/risks as detailed above.	All cut and fill slopes and channel bases will be required to be lined. Viable lining options include rock armouring from the winning of appropriately sized clasts from proposed project specific basalt quarries; composite liners including a geofabric underlay with the use of Firestone EPDM liner; a grouted mattress arrangement; or a concrete impregnated canvas.
Pumping Station Foundations	No specific geotechnical investigations have been undertaken for the proposed 2 pumping stations. It is likely that both pumping stations may require piled foundations, but there are question marks on the ground conditions and appropriate foundation solutions	Undertake geotechnical investigations at the Pumping Station locations to determine actual ground conditions and design appropriate foundation solutions
Pipeline Scour & Dispersive Backfill Materials	Exposed alluvium, residual soils or extremely weathered sedimentary rock sequences have a tendency to be eroded when water flows over them. This could be a significant issue for pipelines in terms of loss of backfill over the crown and ultimately, potentially in high flow flood events, loss of the infrastructure in its entirety. Much of the distribution network on the south side of the Flinders River may have a sodic potential, which could lead to tunnel erosion during the Operational Phase of the project, and again damage or loss of the infrastructure.	Scour protection in the form of rip-rap over the crown is the most likely, practical and economic solution for pipeline lengths in alluvial floodplains. Dealing effectively with sodic soils for backfill materials or immediately adjacent zones either side of the pipelines, may take a variety of forms including amelioration with lime or gypsum, more compactive effort for backfill materials, or sand blocks/barriers at key locations.
Pipeline Flinders River Crossings	The Siphon Crossing of the Flinders River will require a major traverse. Open trenching could be used but significant challenges in terms of health & safety risks, scope of temporary works, losing the works in a flood event and either river ed erosion or shifting river channels overtime are apparent.	Install the required pipelines by a trenchless technology technique, most likely to be Horizontal Directional Drilling (HDD).

9. Conclusions and Recommendations

9.1 Major Conclusions for the HIP Scheme

9.1.1 Ground Conditions

The site bedrock geology generally comprises interbedded Siltstone and Sandstone of the Wallumbilla Formation overlain by a variable thickness and composition of Holocene and Pleistocene Alluvium. The alluvium is cohesively dominated, with the occurring granular stratigraphy potentially linked to former channels. Subsurface conditions have been significantly influenced by palaeochannels which eroded into the underlying Wallumbilla Formation. Changes in depositional environments were believed to have occurred as a result of a period of sea level regression, which was subsequently infilled with Holocene and Pleistocene deposits as the relative sea level rose. The alluvium is deepest/thickest close to the present perennial surface water courses of the Flinders River, Stewart Creek and Back Valley Creek.

Reactive soils were also noted to be present across the site being typically less than 1.5m thick. These soils are believed to have been derived from the original basaltic residual soils that either were transported from colluvial slopes of the previously mentioned mesas in the area, &/or from former lava flows that have now been eroded by the River and creeks meandering across the inherent alluvial floodplains.

The slopes of the basalt mesas are draped in basalt talus and scree generally consisting of gravel to boulder size clasts.

During the investigation traces of gypsum were encountered throughout the soil horizons often occurring in concentrated masses. Gypsum rich layers were identified in Flinders Shire Council borrow pits within road corridors adjacent to the proposed Irrigation Area. Significant and laterally extensive layers of gypsum crystals were identified and are thought to be associated with the Ranmoor Member of the Wallumbilla Formation (see Photograph 6-1), although as previously intimated are likely within the entire stratigraphic sequences of the project site. The gypsum may have formed as evaporite beds during significant climate change through the Quaternary during periods of higher rainfall (where gypsiferous laden water infiltrated through the soil profile – eluviation) followed by subsequent drought.

No surface or subsurface expression of evidence of major geological structural features (such as faults) has been found within or in close proximity to the project site.

The competent bedrock sequence is typically dominated by massive to very thickly bedded subhorizontal to horizontal bedding planes, however within the vicinity of the proposed Spillway discontinuity spacings reduce to very thinly bedded to closely jointed.

9.1.2 Seismic Considerations

No significant geological structural features are in close proximity to the project site. A project specific detailed seismic hazard assessment undertaken has calculated Peak Ground Acceleration (PGA) levels for the proposed HIPCo Saego Dam site for a 475-year return period (10% probability of exceedance in 50 years) of 0.0026 g and for a 10,000 year return period is 0.032 g, when considering earthquakes of magnitude 5.0 or greater with varying Vs30 values. The conclusion is that the risk of seismic events on the proposed works is considered to be very low.

9.1.3 Dam Piping Failure

As the main dam is to be founded at shallow depths, and the potential existence of permeable layers within the alluvial sequence, it is suggested that a grout curtain or slurry trench should be installed through the alluvium and be seated into the less weathered rockmass sequence over the dam Ch 2300-7825m (covering the Type B embankment). With this in mind and taking into consideration the removal of approximately 2.5m of the current superficial materials to found the dam embankment upon, the possible grout curtain depths required could be in the range of 10 to 20m.

9.1.4 Dam Stability

Based on the defined two typical sections (Residual Soil and deep Alluvium founding) for the main Saego Dam alignment both temporary and permanent slope stability FoS have been satisfied for a variety of dam levels on the upstream face and flood levels on the downstream side. Liquefaction analysis for the same typical embankment designs indicate high FoS (at least 2) against liquefaction at this site.

9.1.5 Spillway Considerations

With the presumption that the Spillway structure will be located on competent rock (defined here as moderately weathered and of low to medium strength or better), required excavation could be in the order of a maximum of 10 mbgl, or RL 254 to 257 mAHD.

The potential excavations required will be through typically very stiff to dense alluvial sediments and very low to low strength weathered rock horizons, and as such should be effectively excavated by the use of free-digging techniques by a suitable large excavator.

The Spillway and apron will be designed to withstand uplift pressures, being reservoir pressure at the upstream face varying linearly to tailwater pressure at the end of the apron. The critical feasible kinematic failure case is deemed to be sliding (rather than overturning) given the low strength of the siltstone.

Analysis indicates the probability of a kinematic feasible sliding failure mechanism below the Spillway is very low.

It would be preferable not to rely on anchors for the Spillway monoliths, due in part to the relatively weak rockmass, and it may be required that a shear key and or fattening the Spillway section could be needed to provide additional resisting force if it does not meet acceptance criteria. The Spillway apron will require anchors to withstand the uplift pressure.

The Spillway stilling basin/apron and return channel (draining to Back Valley Creek) will potentially be in erodible materials and overtime could significantly deepen with the possibility of undermining the associated downstream cuttings and the Spillway downstream apron, as well as causing detrimental environmental effects (increasing sediment load in Back Valley Creek). With this in mind the Spillway return channel cuttings will be laid back on 1v:3h with a protective covering of locally won available rock armour (from the proposed basalt quarry) sized to suit the critical flow case (perhaps in the order of D50=750mm). This would extend over the full extent of the Spillway return channel to Back Valley Creek.

9.1.6 Weir Considerations

The proposed gated concrete Weir will require excavations in the riverbed of approximately 9.00 and 12.5m to found on competent rock.

In order to install the weir foundation a significant temporary works excavation will be required in both cohesive and potentially highly permeable granular alluvium perhaps in the order of 4-10m thick, being well below the interpreted winter water table level (approximately 1m below existing riverbed level).

To provide a stable and dry excavation it is likely that the majority of the excavation would need to be dewatered probably by a mixer of well points (at the top) and deep wells. The excavation dimensions would also have to be relatively conservative in design and probably require a two-batter arrangement with a wide intermediate bench.

Analysis of global site-specific televiewer data, as well as derivation of appropriate rockmass design parameters, indicates the probability of a potential kinematic sliding failure mechanism below the Weir is very low, however anchors may still be required to resist water uplift pressures.

9.1.7 Diversion Channel

The maximum depth of cut required to form the Diversion Channel is approximately 12.6 mbgl, and a small section of fill will be required close to the interface with the main Sego Dam wall.

From the formulated ground model it is interpreted that the majority of the proposed cuttings can be effectively excavated by the use of standard plant (dozers, excavators and graders), with only the potential requirement for D10 &/or D11 with tyne rippers or excavators with hydraulic breakers for approximately 1.5 km of the alignment.

Long term stability cut slope batter dimensions will be dependent on the encountered geology, and preliminary minimum cut slope designs have been provided, which take into consideration the potential erodibility of the Wallumbilla Formation sequence when exposed.

Excavated materials could be used for both required areas of fill and for the proposed trailing embankment.

Due to the very high probability of degradation/erosion of exposed both soils and rock sequences, for stability, to maintain full diversion dimensions and to minimise operational phase maintenance requirements, all cut and fill slopes will more than likely be required to be lined. Linings for channels of these lengths and dimensions are of a considerable undertaking and cost effective solutions will be difficult to achieve. Rock armouring from the winning of appropriately sized clasts from proposed project specific basalt quarries could be considered. As an alternative, composite liners including a geofabric underlay with the use of Firestone EPDM liner could be considered, or a grouted mattress arrangement, or a concrete impregnated canvas.

9.1.8 Construction Materials

Based on a variety of local interviews with Council, quarry operators and earthworks contractors, plus remote sensing, walkover surveys and specific sampling and associated laboratory testing, the following conclusions on construction material sourcing are made:

- Sufficient sand and gravel deposits for project construction needs have a good likelihood to be available within the region at probable economic processing and haulage costs; coarse filter materials (sandy gravels) could also be generated from specific hard rock crushing/processing circuits;

- The local basalt can be used for a variety of aggregate sources including rockfill, rip-rap, drainage, gabion, road base and concrete, although it may be variable in composition and thickness over the district; there are no issues with the volume available of these possible aggregates in terms of proximity to any potential scheme options, but this will require the establishment of project specific quarries;

- The location of suitable quality and quantity of low plasticity clay deposits for possible dam clay core sources within the widespread alluvial deposits, may be problematic to identify. Upper Holocene and

Lower Pleistocene clays are both reactive, dispersive and sodic in nature. The weathered basal sedimentary sequence of siltstones/sandstones/shales could be a possible source of clay for dam cores if processed, however their sodic & dispersive nature does not lend it to advantageous clay core sources. To alleviate these presently interpreted poor clay core sources may require appropriate amelioration with lime, use of more substantial dam filters, blending with inert local materials, and/or consideration of concrete or grout enriched dam upstream faces; in addition, careful moisture control of placed core materials will be required; the present base case for design and associated CAPEX estimation is based on the use of untreated near surface alluvial and residual soil clays as dam clay core (Type1A) & Earthfill (Type 1B) materials, with wider dam filters being employed; and

- Local concrete batching is available but may not meet a D & C contract needs.

9.1.9 Distribution Network

From the interpreted ground conditions, all pipelines are expected to be installed by free digging with excavators in trenches, with the exception of the proposed siphon crossing of the Flinders Rivers which will be presently installed by HDD.

In the alluvial floodplains, perched water tables could require battering back or the use of sheet piling and dewatering for trench stability. In these areas buoyancy and scour may be required to be dealt with by the use of sand bagging and rip-rap backfill for the majority of the trench over the pipe crown.

Reactive soil excessive swelling under the pipe invert may also need to be catered for by again similar measures for pipe buoyancy. Sodic soils could also be encountered in the pipeline trench alignments and a variety of control measures could be applied to deal with associated risks, including amelioration, the use of sand blocks or differing backfill compaction specifications.

The majority of trench excavated materials can be used as backfill over/around the pipelines, although as above they may need different compaction regimes and/or additives.

Due to the probability of encountering relatively deep rockhead at both of the proposed pumping station locations and the likelihood of improving density/shear strength of both alluvial and weathered rock sequences with depth (as indicated by the project specific investigations), 900mm concrete piles driven provisionally to 15mbgl, have been considered as appropriate for both acceptable load and associated settlement criteria.

9.2 Recommendations for the Next Stage of Design

The following recommendations for subsequent stages of the HIP are provided below :

- Confirmation of Vs30 values by the undertaking of site specific crosshole geophysical shear wave velocity surveys;

- Confirm founding level and the currently very low potential for kinematically feasible sliding failures for the Spillway structure by additional geotechnical investigations;

- Appropriate intrusive geotechnical investigations are required within the proposed weir structure footprint to confirm foundation depths of the mass concrete structure & the present indication of a very low probability of sliding failure, but also for the design of appropriate temporary cut slopes and a likely dewatering system within the Flinders River channel;

- Appropriate geotechnical investigations (geophysical and/or intrusive) are required over the proposed siphon crossing of the Flinders River to confirm HDD pipe installation depths to ensure they are below the scour depths within the design life of the project;
- Confirm the need of a slurry trench for the Type B dam embankment design by undertaking additional soil and rock insitu permeability testing;
- Carryout additional boreholes at the proposed Spillway location to confirm founding level of structure, as well as the need for anchoring;
- Sample and carryout appropriate laboratory testing for existing and possible new sand & gravel borrow pits, as well as potential appropriate basalt quarry sites;
- Undertake a suite of blending alluvial clays with alluvial sands and gravels (which will require site works to obtain relevant samples) and then undertake an appropriate laboratory testing schedule to identify potential optimum clay core source mixes;
- Undertake additional intrusive geotechnical investigations to inform the scheme design development via an appropriate Gap Analysis; and
- Confirm embankment design and stability of the Diversion Channel earth embankment;

10. References

- ANCOLD : 2013 : Guidelines on Design Criteria for Concrete Gravity Dams. Australian National Committee on Large Dams, Hobart.
- ANCOLD : 2018 : Guidelines for Design of Dams and Appurtenant Structures for Earthquake. Australian National Committee on Large Dams, Hobart.
- ANCOLD : 2018 : Draft Practice Note Geotechnical Investigations of Dams, Their Foundations, and Appurtenant Structures, Unpublished
- AS 1726 : 2017 : Geotechnical site investigations : Australia Standards
- AS 2159 : 2009 : Piling – Design & Installation : Australia Standards
- Atkinson, T : 1992 : Design and layout of haul roads; *Chapter 13.4 - SME Mining Engineering Handbook.* 2nd edition. Society of Mining, Metallurgy, and Exploration, Inc. Littleton, Colorado: 1334 – 1342.
- Department of Natural Resources, Mines & Energy : 2019 : Design & Construction of Mine Roads
- Bieniawski, Z T : 1989 : Engineering Rockmass Classifications : John Wiley & Sons
- Construction Services Pty Ltd : 2020 : Geotechnical Investigation – Factual Reporting, Saego Dam – Report No. 10599E/P/106
- Construction Services Pty Ltd : 2019 : Ground Investigation - Alderley Crossing Bridge, Report No. 10599E/P/100_v2
- Caterpillar : 2016 : Caterpillar Performance Handbook 46
- Douglas Partners : 2018 : Hughenden Recreational Lake – Report on Geotechnical Investigation (Report No. 93897.00_R.001.Rev0
- Douglas Partners : 2018 : Hughenden Recreational Lake- Report No. 93897.00_R.002.Rev0
- Engeny : 2020 : Preliminary Business Case, Hughenden Irrigation Project, Volume 1 Study Report
- Fell, R, MacGregor, P, Stapledon, D, Bell, G, and Foster, M : 2015 : Geotechnical Engineering of Dams, Second edition. Taylor Francis Group, London.
- Hoek E : 2007 : Practical Rock Engineering
- Hoek E, Carranaza-Torres C & Corkun B : 2002 : Hoek-Brown Failure Criteria – 2002 Edition
- Hoek E & Brown E T : 1997 : Practical estimates of rock mass strength : Intnl. J. Rock Mech. & Mining Sci & Geomechanics Abstract 34 (8), pps 1165-1186
- Houlsby A C :1976 : Routine Interpretation of the Lugeon Water Test : QJEG Vol 9, pps 303-313
- Idriss I M & Boulanger RW : 2008 : Soil Liquefaction during Earthquake: EERI Publication, Monograph MNO-12, Earthquake Engineering Research Institute, Oakland.
- Jacobs : 2021 : Geotechnical Desk Study and Initial Site Findings Report; Report No. IS362200-GT-RPT-0001 | 01
- Jell P A : 2013 : Geology of Queensland : Geological Survey of Queensland, ISBN 9781921489761
- Lunne, T, Robertson, P K & Powell J J M : 1997 : Cone Penetration Testing in Geotechnical Practice
- Peritus Ag : 2021 : Detailed Business Case, Hughenden Irrigation Project, Soil Survey, Erosion & Salinity Management Report
- Pettifer G S & Fookes P G : 1994: A revision of the graphical method for assessing the excavatability of rock : QJEG 27, pps 145-164
- Serafin J L & Pereira J P : 1983 : Considerations on the geomechanical classification of Bieniawski : Proc. Int. Symp. on Eng. Geol. and Underground Construction Vol I (11), Lisbon Portugal, 1983, pps 33-44
- Stark T D and Idries A : 2021 : Drained Residual Shear Strength Power Function Coefficients a and b
- Vine R R , Bastian L V , Casey D J, and Chertok I : 1970 : Richmond, Queensland, 1:250 000 geological series map. Sheet SF/54-04, 1st edition. Bureau of Mineral Resources, Australia 1v Map Legend

Appendix A. Figures and Drawings

Appendix B. Site Photographs

Appendix C. Geophysical Survey & Televiewer Surveys

Appendix D. Cone Penetration Testing

Appendix E. Engineering Borehole Logs, Core and Test Pit Photography

Appendix F. In-situ Soil Permeability Testing

Appendix G. Packer Test Analysis

Appendix H. CPTu Interpretation with CPeT-iT

Appendix I. Laboratory Test Certificates

Appendix J. Seismic Hazard Assessment

Appendix K. Saego Dam Stability Assessment

Appendix L. Geomechanical Assessment

Appendix M. DIPS Analysis