HYDROGEOLOGICAL ASSESSMENT FOR A LAND RECLAMATION DEWATERING OPERATION

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ABSTRACT

The Kingston Foreshore Redevelopment was an urban renewal project in the suburb of Kingston on the southern shore of Lake Burley Griffin in Canberra. It involved reconfiguration of the foreshore by the excavation of part of the existing foreshore and the reclamation from the lakebed to form a harbour. Conventionally, reclamation process for "wet conditions" involves end dumping of granular fill, with subsequent vibro-compaction or dynamic compaction stabilisation. However, this method would not be possible since local sources of granular fill were not economically viable. Moreover, while the excavated materials from the existing foreshore were expected to be firm to stiff clays, the dumping of these excavated "lumpy" clays into the ponded reclamation areas without compaction, and the reliance upon subsequent preloading for stabilisation, would be difficult and problematic. The more feasible option in the geotechnical context was the "dry reclamation", where the reclamation areas was dewatered and fill placed in the dry. This approach required that the reclamation areas to be enclosed and water be pumped out and discharged into the lake. The selection of a dewatering system would in turn be a function of the hydrogeological model and the level of drawdown required.

This paper focuses on the hydrogeological model and properties of the site that were considered to be critical for the assessment and design of the construction options. The assessed hydrogeological model in the foreshore areas comprises an upper fine-grained alluvium, which acts as an aquitard to restrict the flow of the water from the lake to the underlying gravel/sand aquifer. The gravel/sand aquifer is therefore likely to be semi-confined. Laboratory tests and full scale pumping/recovery tests were undertaken to estimate the permeability and storativity of the gravel/sand aquifer. Back-analysis of the pumping test results indicated that the permeability values derived from various analytical methods such as the distance drawdown analysis and the recovery analysis compared reasonably well with those estimated using the more simplified Hazen's (1911) empirical method, which was related to the particle sizes of the gravel/sand materials. No in-situ permeability test was conducted in the upper alluvial aquitard. The hydrological conductivity of this layer was instead inferred from the dissipation test results obtained from CPTU soundings. It had been shown that the inferred permeability of the upper alluvial aquitard compared reasonably well with published correlations.

A number of dewatering schemes were assessed by undertaking 2D Finite Element Analysis (FEA) to check the viability of various options and sensitivity to changes to soil permeability properties. A summary of the various FEA results is presented.

1. INTRODUCTION

The Kingston Foreshore Harbour Development involves the staged redevelopment of a previous industrial area for residential and commercial uses on the foreshores of Lake Burley Griffin. The redevelopment of an existing boat harbour forms a key part of the foreshore project. The proposed scheme involves the reconfiguration of the foreshore and boat harbour by the excavation of part of the existing foreshore and the reclamation of a lake and boat harbour area. The preferred dry reclamation approach requires that the reclamation areas be enclosed and separated from the adjoining lake. This paper focuses on the hydrogeological model and properties of the site that were considered to be critical for the assessment and design of the construction options. A number of dewatering schemes were assessed by undertaking 2D Finite Element Analysis (FEA) to check the viability of various construction options and sensitivity to changes to soil permeability properties. A summary of the various FEA results are presented.

2. SITE CONDITIONS

The 'Kingston Foreshore' is a 37 hectare development located adjacent to the East Basin of Lake Burley Griffin, Canberra. Lake Burley Griffin was created artificially in 1963 by the construction of Scrivener Dam on the Molonglo River, west of the Canberra CBD. The geometry of the East Basin is defined by the pre-lake course of the Molonglo River. Lake walls were constructed on the southern edge of the meandering river course and backfilled to form a relatively flat foreshore. As part of the lake construction, a boat harbour was built in the north-east parts of the foreshore area, adjacent to the confluence between Jerrabomberra Creek and the Molonglo River.

The Australian Government Printing Service (AGPS) was established in the south-west corner of the site in the early 1960’s and served as the government printer until the mid-1990’s before being demolished in 2002. The remaining foreshore area was occupied by transport and bus depots, general storage and associated infrastructure for much of its history. The northern and eastern parts of the foreshore area were less developed, with only limited structures being built around the boat harbour.

At the time of the investigation stage prior to construction, the foreshore area was relatively flat with the exception of certain fill stockpiles in isolated areas of the site. Typically, the foreshore ground surface level was at about RL 556.7 – 557m. The nominal water level in the lake was about RL 556m with some minor variations (± 0.2m approximately).

3. GEOLOGICAL CONDITIONS

3.1 REGIONAL GEOLOGY

The 1:10,000 scale Engineering Geology map for Central Canberra indicates that the site is underlain by Quaternary alluvium overlying weathered sedimentary rocks of the Canberra Formation. An extract of the geological map is shown in the attached Figure 1.

Henderson (1986) describes the Quaternary alluvium as a sequence of clayey sand and gravel covered by clean sand and gravel, which is overlain by clay and silt. The Canberra Formation generally comprises siltstone, sandstone, limestone and shale.

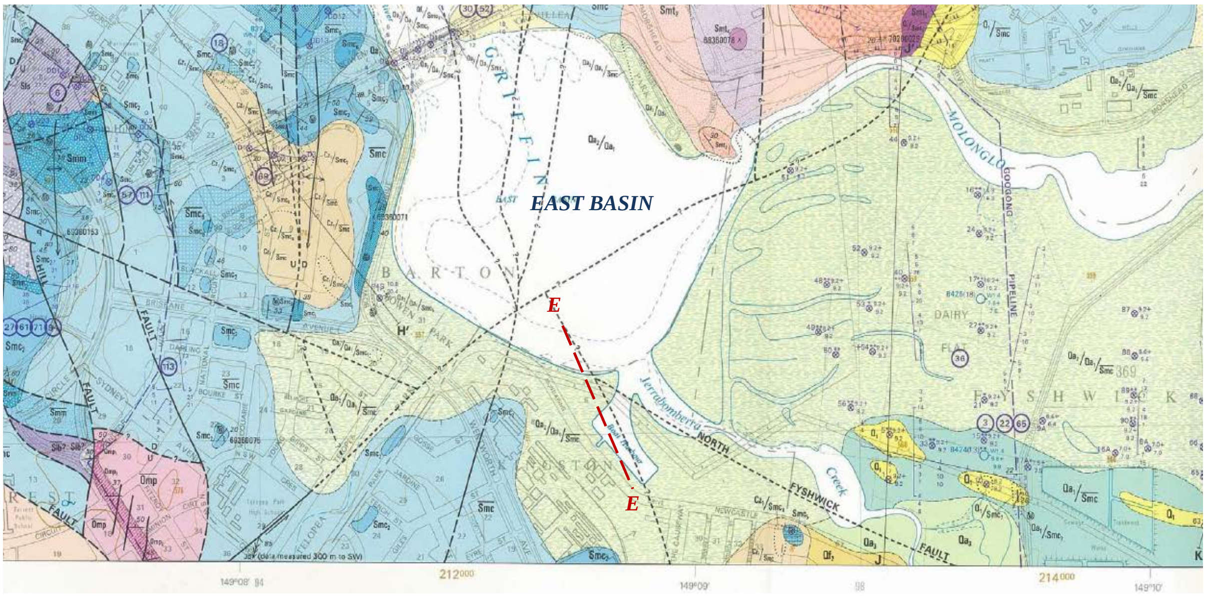

Figure 1: Regional geology

3.2 STRATIGRAPHY

In summary, the basic stratigraphy over the existing foreshore areas comprised the stockpile materials in localised areas and existing foreshore fill underlain by alluvial soils over weathered siltstone bedrock. The alluvial soils could be sub-divided into an upper stratum of predominantly clays and sandy clays underlain by a sandy profile of silty sand, well graded sand and gravely sand / sandy gravel.

The upper alluvial horizon generally consisted of low plasticity clays and sandy clays. However, the composition of the upper alluvial soils could be variable and included sandy layers in various localised areas. Some soft clay lenses were also encountered near the existing boat harbour, but appeared to be of limited thickness.

The thickness of the upper alluvial horizon was also variable, ranging from about 2.5m to about 7.2m. However, most importantly, this stratum is absent in the proposed reclamation area of the East Basin, as shown on Section E in Figure 2 (Refer Figure 1 for location of Section E).

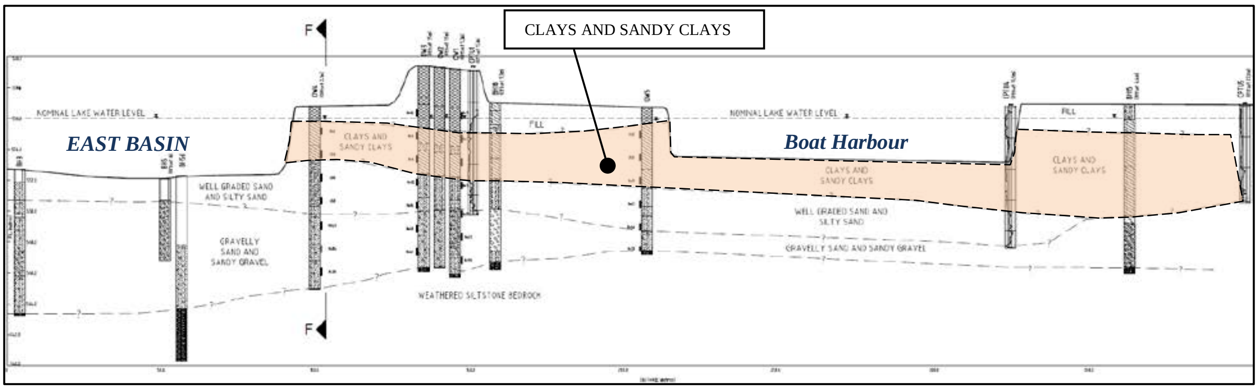

Figure 2: Stratigraphy along Section E

The upper fine-grained alluvial horizon was underlain by a sandy alluvial soil profile over the subject site. The presence and thickness of this lower sandy horizon was important for the proposed development during construction because of its relatively permeable nature. The interpolated thickness isopachs of the combined gravel/sand horizon are shown in Figure 3. It is considered likely that the permeability of the “gravel” layer will be dictated by its sandy matrix, hence the combining of these layers is considered most relevant.

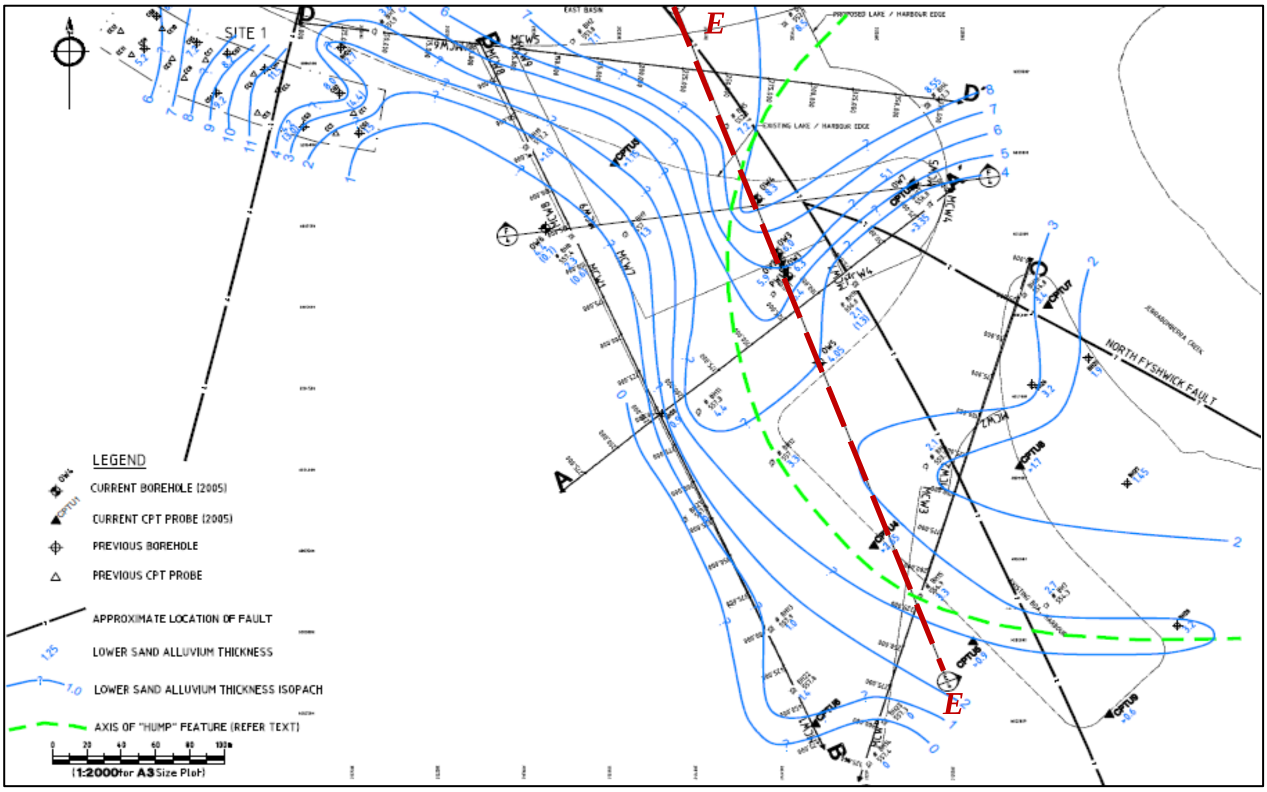

Figure 3: Thickness isopachs of lower san alluvium

As can be seen from this figure, the thickness varied from less than 1.0m along the western boundary to over 8.5m in the existing East Basin lake. The thickness isopachs depict an apparent “hump” feature with associated gullies. A localised (deeper) sand/gravel feature also existed to the west, this time following an eroded bedrock profile along an apparent fault feature.

This sandy profile also seemed to comprise silty sand and well graded sand at the top underlain by gravely sand and sandy gravel. The variation of the various horizons is demonstrated in Figure 2.

The siltstone bedrock was assessed as generally being slightly weathered and strong. It is noted that extremely weathered, very low strength siltstone was also encountered in some boreholes.

3.3 HYDROGEOLOGICAL MODEL

One of the key considerations for the selection of appropriate construction techniques for the excavation and reclamation work was the ability to dewater the site during construction. The hydrogeological model of the site was thus critical for the assessment and design of the construction options.

It is clear from the stratigraphic model that the lower gravel/sand alluvium was acting as a permeable aquifer with expected high water flow rate. Furthermore, it can be seen from some standing groundwater monitoring results that the gravel/sand aquifer was hydraulically connected to the lake.

However, the upper fine-grained alluvium in the foreshore areas, and the existing Boat Harbour, was acting as an aquitard to restrict the flow of water from the lake to the gravel/sand aquifer. The gravel/sand aquifer was therefore likely to be semi-confined (i.e. leaky aquifer).

Similarly, any recent deposits on the lake floor were expected to be fine grained and had a relatively lower permeability than the underlying gravel/sand alluvium. Depending upon the continuity and thickness of the fine-grained mud, the lake floor deposit could also act as an aquitard.

Very limited data was available on the weathered siltstone bedrock. However, it is considered that the permeability of the siltstone bedrock was likely to be governed by the characteristics of any joints within the rock, but should be significantly lower than the gravel/sand aquifer.

With regard to the recharge boundaries, it was envisaged that the majority of the recharge should be originated from the lake, while the groundwater flow from the land (south west) side should be of secondary magnitude only. The boundaries of the lake recharge sources would be dependent upon the presence and continuity of the lake floor mud aquitard.

4 RECLAMATION OPTIONS

The Kingston Foreshore Harbour project includes the reconfiguration of the foreshore and boat harbour by excavation and reclamation. The original concept for this latter works basically involved construction of a levee bank into the Lake, with various combinations of sheet piling, followed by reclamation using site-excavated materials. This reclamation was to be in either wet or dry condition, dependent on the contractor's preference. The concept also involved preloading of the site for a fixed period to treat an underlying "soft soil" condition.

However, further consideration concluded that reclamation works in the "wet condition", can only be feasibly undertaken using granular materials, particularly given the need to create a "stable site" for roads and services. The use of excavated (or stockpiled) stiff clays, end dumped into a ponded reclamation area, with reliance on subsequent preloading for stabilisation, was not considered practical.

Conversely, hydraulic filling or end dumping of granular fill, with subsequent vibro-compaction stabilisation, is a conventional reclamation process. However, there were no economic sources of locally available granular material at the time. A graded, recycled concrete fill of the volumes anticipated was not considered economically feasible.

It was thus considered that reclamation in the "dry condition" using the locally available excavated stiff clays and clayey sands would be preferred.

5 HYDROGEOLOGICAL INVESTIGATION

The hydrogeological investigation comprised:

- One pump-well borehole;
- Seven observation well boreholes;
- A 24 hour pumping test followed by a 24 hour recovery test; and
- Nine piezo-cone (CPTu) tests.

A pump well was installed in the pump-well borehole consisting of six inch steel casing at the top with a 6m length of 6 inch stainless steel screen (2mm aperture) at the base. The screened section of the well was targeted at the likely gravel/sand aquifer within the stratigraphic profile. Washed river filter gravel (10mm) was backfilled around the screened section of the well with a 1m length bentonite plug above. The borehole was then backfilled around the well casing with cuttings from the hole with a concrete seal at the surface.

Observation wells consisted of 50mm diameter PVC pipes (Class 18) with a 3m length of machine slotted screen at the base, with the exception of one well where 6m length of screen was installed. Washed coarse grained filter sand was backfilled around the screened section of the wells covered with a 1m thick bentonite plug. The boreholes were then backfilled around the well casing with a bentonite cement grout to the surface. The screened sections of the observation wells were again aimed to monitor the groundwater levels within the gravel/sand aquifer and were separated from the overlying layers by the bentonite plug.

A 24 hour constant rate pumping test was conducted, followed by a 24 hour recovery test. Prior to the constant rate pumping test, a 4 stage multi-rate short duration pumping test was also undertaken to determine the appropriate pump rate for the 24 hour test. Based on the short multi-rate test results, a constant rate of about 1.45 l/sec was adopted for the 24 hour pumping test. The water collected from the pumping was discharged into a temporary detention basin lined with geo-membrane.

The testing and data monitoring of the 3 observation wells near the pump well (at 5.2m, 10.2m and 15.2m distance from the pump well) were undertaken. The water levels within the outer observation wells at some 50m to 150m distance from the pump well were monitored manually.

Piezocone testing (CPTu) was performed using a GeoMil 25MPa probe pushed with a portable hydraulic rig. The CPTu positions were generally biased to areas with suspect “soft” soils. However, one CPTu was located at the pump well on top of an existing stockpile.

The penetration of the probe was stopped temporarily at selected depths. The dissipation of excess pore water pressure generated by the probe penetration was monitored with time to assess the permeability characteristics of the encountered soil.

6 HYDROGEOLOGICAL PROPERTIES

6.1 GRAVEL/SAND AQUIFER

The assessment of the hydrogeological properties of the gravel/sand aquifer is critical to the design of any dewatering systems for the subject site. Both the limited laboratory test results and the full scale pumping/recovery test data had been analysed to estimate the permeability of this gravel/sand aquifer.

Laboratory particle size analysis had been conducted on a number of samples on the gravely sand material during the investigations. The permeability (k) of the tested samples was estimated using the empirical method proposed by Hazen (1911) as follows:

$$k = D_{10}^2 \qquad (1)$$

Where D_{10} is the particle size in mm with 10 percent finer materials and k is the estimated permeability in cm/sec.

Using this empirical method, the permeability of the gravely sand material was inferred to be between 1×10^{-3} cm/sec and 1×10^{-2} cm/sec generally, with one possible higher result at 1×10^{-1} cm/sec.

It is the authors’ experience that the above empirical correlation is reasonable in order to estimate the permeability of the tested samples. However, it should also be noted that the global permeability of the gravely sand (or sandy gravel) can be different from the sample permeability due to sampling bias and likely heterogeneity and layering of the sand profile. The “actual” global permeability can only be reliably estimated from back-analysis of an in-situ pumping and recovery test.

The full scale pump test undertaken was back-analysed using a number of different techniques. Both the pumping drawdown test data and the recovery test data were analysed in accordance with Clause 28 of BS 5930:1990 “Analysis of Pumping Tests” using the following different analytical methods:

- Theis analysis of the drawdown test data
- Cooper-Jacob analysis of time vs drawdown data
- Cooper-Jacob analysis of distance vs drawdown data
- Recovery analysis

The back-analysed properties are summarised in Table 1. It is noted that based on the authors’ experience, the values derived from the distance drawdown analysis and recovery analysis (Figure 4) are more reliable than the Theis drawdown and time drawdown analysis because of potential initial development errors. Based on the back-analysis, the values adopted for concept design purposes are also shown in Table 1. The back analysed permeability and the adopted value were consistent with the aforementioned value derived from Hazen's (1911) empirical method.

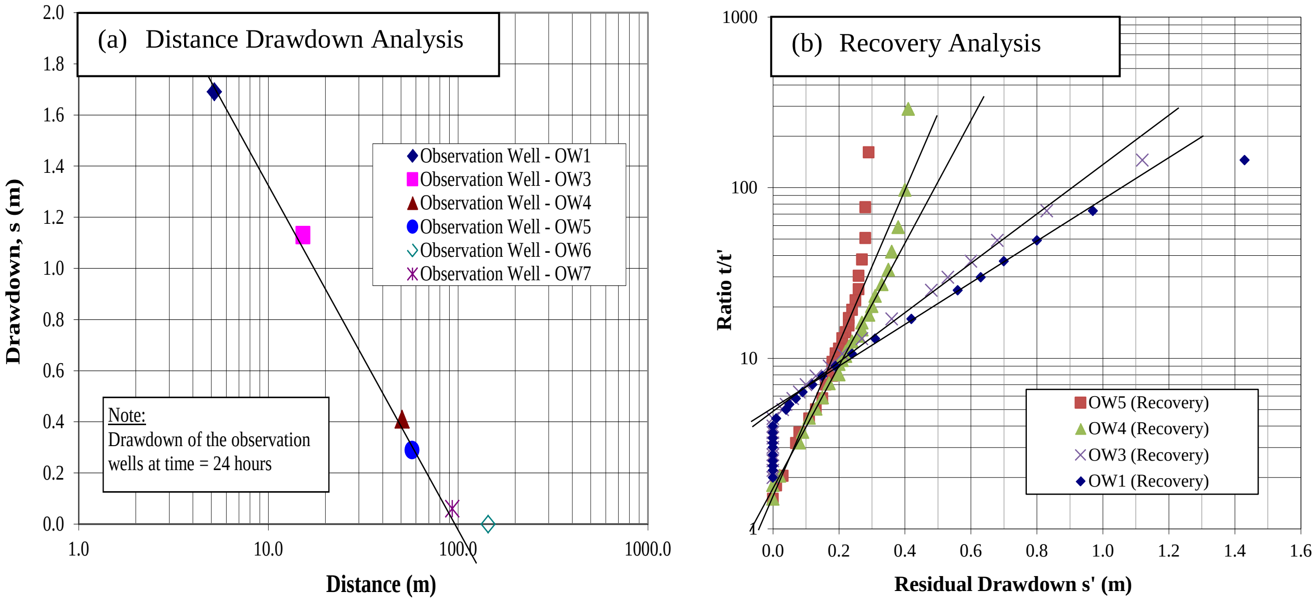

Figure 4: (a) Distance Drawdown Analysis; (b) Recovery Analysis

Table 1: Summary of Pump Test Back-analysis Results

Analytical Method	**Transmissivity (m3/sec/m)**	**Permeability† (cm/sec)**	**Storativity**
Tester Interpretation	$2.8\text{x}10^{-4}$	$4.7\text{x}10^{-3}$	$4\text{x}10^{-3}$
Theis Drawdown Analysis	$4.3\text{x}10^{-5}$ to $1.8\text{x}10^{-3}$	$7.1\text{x}10^{-4}$ to $3.0\text{x}10^{-2}$	$5\text{x}10^{-4}$ to $6\text{x}10^{-3}$
Cooper-Jacob Time Drawdown Analysis	$3.8\text{x}10^{-4}$ to $1.7\text{x}10^{-3}$	$6.2\text{x}10^{-3}$ to $2.9\text{x}10^{-2}$	$1\text{x}10^{-5}$ to $4\text{x}10^{-5}$
Cooper-Jacob Distance Drawdown Analysis	$3.9\text{x}10^{-4}$	$6.6\text{x}10^{-3}$	$8\text{x}10^{-3}$
Recovery Analysis	$2.4\text{x}10^{-4}$ to $1.2\text{x}10^{-3}$	$4.0\text{x}10^{-3}$ to $2.0\text{x}10^{-2}$	-
Adopted Values	**$3.0\text{x}10^{-4}$**	**$5.0\text{x}10^{-3}$**	**$5\text{x}10^{-3}$**

† assumed aquifer thickness = 6.0m based on observation well drilling.

6.2 UPPER ALLUVIAL AQUITARD

No in-situ permeability testing has been conducted in the upper alluvial aquitard in this area. Furthermore, the laboratory empirical method using particle size distribution is applicable to predominantly uniform sand only, and cannot be used for fine-grained material. Nevertheless, a number of pore pressure dissipation tests as part of the CPTU testing had been conducted to assess the drainage characteristics of the penetrated materials.

The dissipation testing measures the rate of pore pressure dissipation and rate of consolidation under applied loading, and does not measure permeability of the surrounding soil directly. However, using the determined coefficient of consolidation, c_v and the inferred coefficient of volume change, m_v (and taking into consideration of over-consolidation ratio), the permeability of the surrounding soil can be estimated using the theoretical relationship:

$$k = \gamma_w \, m_v \, c_v \tag{2}$$

where γ_w is the unit weight of water

The estimated permeability values for the upper alluvial aquitard are plotted against the time to achieve 50% pore pressure dissipation, t_{50} in Figure 5. A number of published relationships are also shown in the figure for comparison purposes and confirmed that the estimated values appear to be reasonable for the t_{50} measured.

The inferred permeability of the upper alluvial aquitard ranged between 1.7×10^{-7} cm/sec and 2.1×10^{-6} cm/sec. For option assessment purposes, a value of 1.0×10^{-6} cm/sec was adopted.

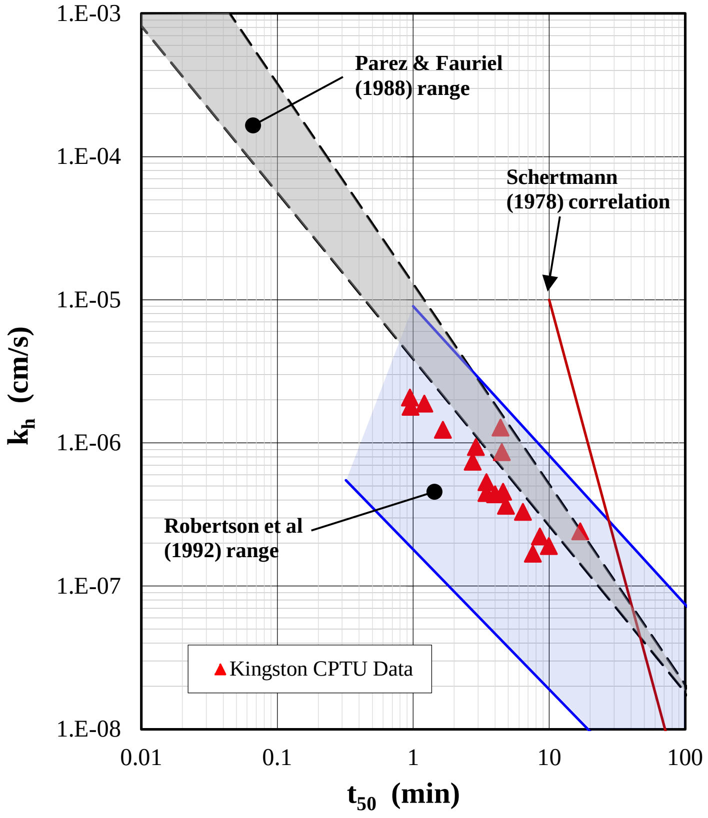

Figure 5: CPTU Estimated Soil Permeability

7 NUMERICAL ASSESSMENT OF DEWATERING OPTIONS

Limited 2D finite element modelling of the dewatering scheme was undertaken to check its viability and sensitivity to changes to soil stratum (permeability) properties. The modelling results are as follows, with reference to the output figures:

(1) The basic geometry chosen is shown on Figure 6, namely a homogeneous bund (recycled concrete with permeability, k=1x10^{-2} cm/sec), on a sand gravel aquifer (k=5x10^{-3} cm/sec) and with a lake sludge upstream (k=1x10^{-4} cm/sec).

(2) Case 1 (Figure 7) gives the basic model without any sheet pile cut-off wall, and with 2 spear point controls set 1.5m below the mud dredged surface. The maximum flow was 8.L/min/m, which theoretically can be accommodated by even a single spear point row (at 5m centres) at bund toe. It was expected that an intermediate row would be used to share the flow and remove seepage from bund downstream face.

(3) Case 2 (Figure 8) provides for a near-fully penetrating (non-leaky) cut-off 1m above the bedrock. The flow was 3 L/min/m and thus about 40% of the base case, but with a downstream phreatic surface almost at dredged surface (thus providing protection for seepage erosion of bund).

(4) Case 3 (Figure 9) is an intermediate cut-off with an expected intermediate flow (5 L/min/m) but a similar low downstream phreatic surface result.

(5) Cases 4 and 5 (graphical outputs not shown) provide for a conservative "leaky" sheet piling (equivalent to k=1x10^{-4} cm/sec) for the above cases 2 and 3 respectively. The flow rates into the main (spear point) control line increased (still less than Case 1), but the bund protection was preserved.

It is thus apparent that a partially penetrating cut-off scheme would work, for the gravely sand aquifer k of about 5x10^{-3} cm/sec as assessed from pump test results and was as expected from grading of the materials.

Furthermore, an additional analysis case had been performed with an increased bund permeability (k=1x10^{-1} cm/sec) for Case 3 (with intermediate cut off only), to ensure that the bund properties do not overly impact on the composite scheme (graphical outputs not shown). The results showed that the maximum flow to the spear points was about 5.5 L/min/m which was slightly more than Case 3 as expected, but was still less than Case 1. More importantly, the phreatic surface on the downstream side was controlled to near the dredged surface with the bund protection preserved.

A possible alternative to the sheet pile cut-off walls, in order to reduce flow, was the use of an impermeable membrane on the upstream bund face to exclude water inflow through the bund face. The numerical model for this option is shown in Figure 10. In addition, it was assumed that the lake sludge could be absent in this model.

Two cases (7 and 8) had been analysed for the impermeable membrane option using fine (k=1x10^{-2} cm/sec) and coarse (k=1x10^{-1} cm/sec) recycled concrete respectively. The predicted maximum flows to the spear points were 4.2 L/min/m and 6.9 L/min/m for the two recycled concrete permeability cases respectively. Again, the downstream bund protection was preserved using this alternative treatment approach (see Figure 11 for graphical outputs for the coarse recycled concrete case).

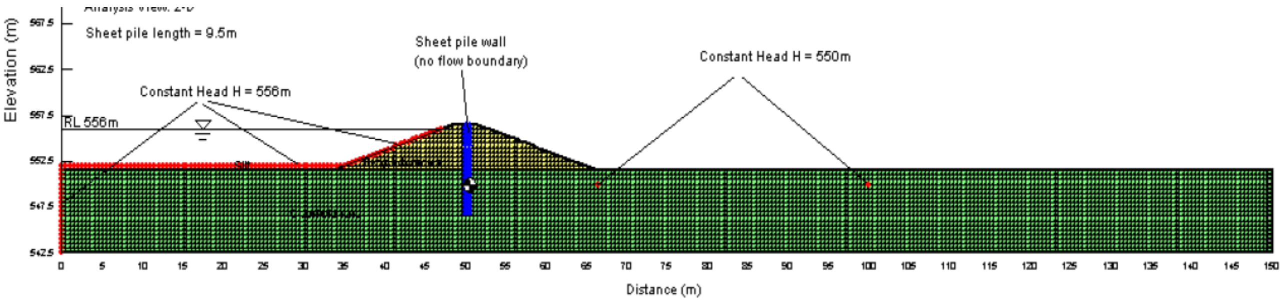

Figure 6: Finite element model and boundary conditions

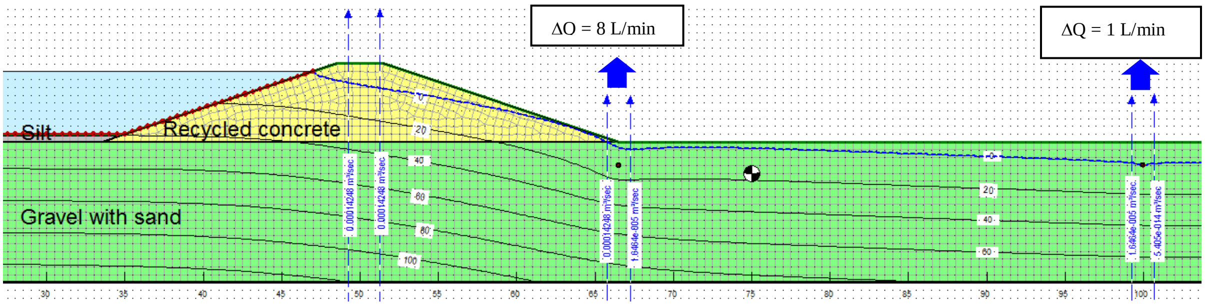

Figure 7: Case 1 – No sheet pile wall

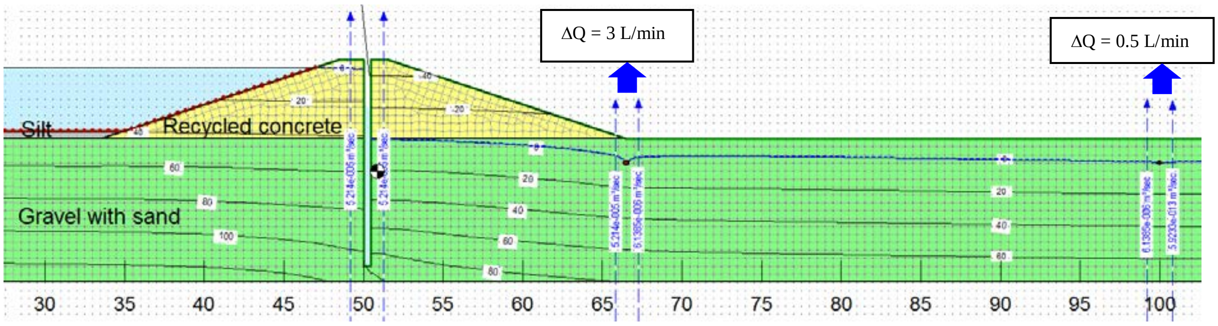

Figure 8: Case 2 – 13m long sheet pile wall

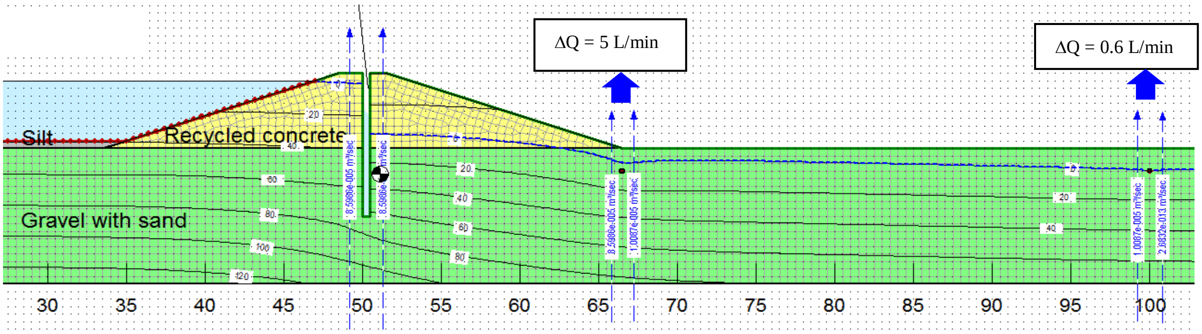

Figure 9: Case 3 – 9.5m long sheet pile wall

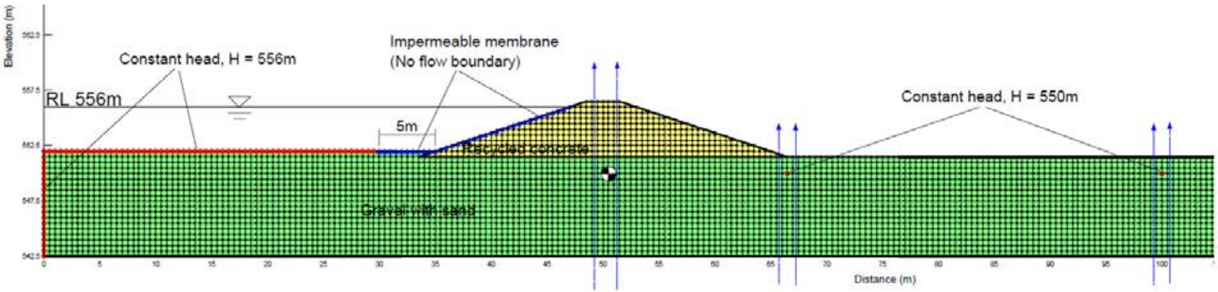

Figure 10: Finite element model and boundary conditions for upstream impermeable membrane cases

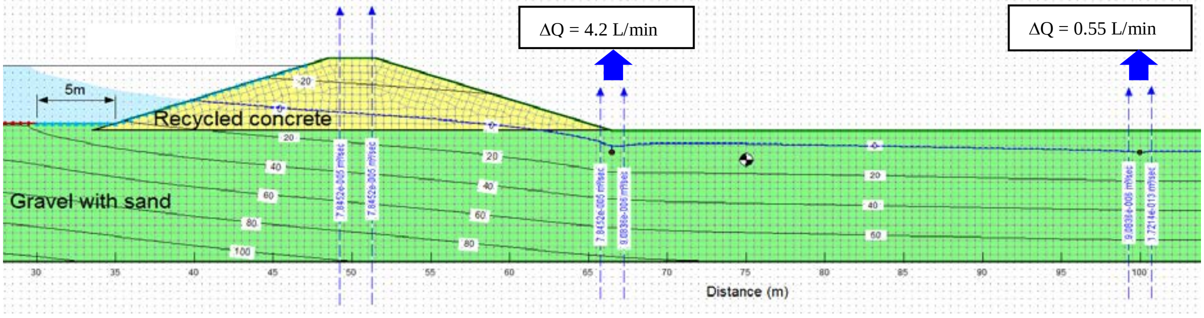

Figure 11: Upstream impermeable membrane with fine recycled concrete bund

8 DISCUSSION

The preferred dry reclamation approach required that the reclamation areas be enclosed and separated from the adjoining lake. In addition, the water within the enclosed areas would need to be pumped out and discharged into the lake. The groundwater table within the enclosed areas should also be lowered to at least some 0.5m - 1.0m below the lake floor level in order to provide reasonable trafficability for construction machineries.

A number of potential containment options were possible including temporary sheet pile walls with/without earth filled cofferdams. The suitability of the adopted containment system would be dependent upon the availability of the materials and the dewatering requirements.

The selection of a dewatering system would in turn be a function of the hydrogeological model and the level of drawdown required. The adopted permeability values of both the upper aquitard ($1x10^{-6}$ cm/sec) and the gravel/sand aquifer ($5x10^{-3}$ cm/sec) are typical values for clays and sands respectively.

It must be recognised that the water inflow volumes could be large without appropriate treatment. The water flow would predominantly be recharged from the lake. Treatment options could therefore include the fully or partially penetrating sheet pile walls or placing impermeable geo-membranes on earth filled cofferdams.

The sheet piles could be driven into the gravel/sand aquifer to a level required for reducing the flow rate to an acceptable level. Alternatively, the impermeable geo-membranes could be placed on the upstream side of the cofferdams and extended into the lake to increase the drainage path as required.

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