Technical Memorandum

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Company:	**SLR Consulting Limited**
cc:	**Date:** 10 February 2026
	Project No. 777.V02406. 00001

RE: Tashkent International Airport
Hydrogeological Conceptualisation – Initial Technical Note (Draft)

1.0 Introduction and Purpose

This document comprises an initial hydrogeological conceptualisation for the proposed Tashkent International Airport development. The work has been undertaken to support the ESIA, it comprises a combination of scoping, baselining and preliminary impact assessment information, in response to IFC comments requesting a Conceptual Groundwater Model.

The scope of this work is an initial, high-level hydrogeological conceptual review that contains GIS-data and is based on available information and professional judgement. Limited site-specific field data is available. This review is intended to:

- Develop hydrogeological scoping into an initial baseline appraisal and preliminary evaluation of potential aspects, impacts and risk-mechanisms.
- Establish a preliminary understanding of hydrogeological conditions.
- Identify key groundwater-related aspects of the proposed project, particularly addressing risk associated with large-scale dewatering.
- Highlight data gaps and recommendations to reduce uncertainty.
- Set out a defensible basis for further analytical and/or numerical modelling that may be required at later stages.

The review does not constitute a full hydrogeological baseline or water resource impact assessment. Key aspects of dewatering design information and site-specific hydraulic test information is still pending. As such only conceptual modelling is undertaken herein. The assessment is based on:

- Review of limited client-supplied information such as GIS and geotechnical summaries where available;
- High-level interpretation of available data, and
- Assumptions informed by professional judgement and comparable alluvial settings.

Key points at this stage are:

- There is, as yet no confirmed hydraulic testing data (permeability or storage coefficients) available (e.g. pumping or slug tests);
- There are, as yet no confirmed construction-phase or operational dewatering rates;
- There are limited water level measurements across a network of eight (8) piezometers installed under the Haskoning/O'zGashlikti, Aug. 2024 geotechnical mobilisation which were dipped four time between May and June 2024 but no further groundwater level monitoring is currently available and a qualitative description of intra-annual groundwater level fluctuation has been adopted from the above report

(but not verified). Discussion of potential groundwater flooding in the above report indicates fluctuations may at times be adverse and greater than presented.

- There is no detailed receptor inventory (e.g. private wells, irrigation abstraction quantities and channel and water depths), although irrigation channels have been mapped.

All findings in this review should therefore be regarded as indicative for the purpose of impact assessment (not design) and are subject to refinement as additional information becomes available. The information provided has been based on available information and professional judgement, following the methodology outlined but no assurance can be given in terms of any third-party use or reliance on the work.

2.0 Site Setting (Overview)

Site Location

The future airport site comprises a 13 km^2 project area located on the cultivated flood plain approximately 2km southeast from the Chirchik River channel (at its closest point). The development footprint is 6km wide within the territory of Tashkent Province, approximately 20km from the existing Tashkent International Airport.

The development area is rural comprising cultivated, irrigated agriculture land-parcels associated with.

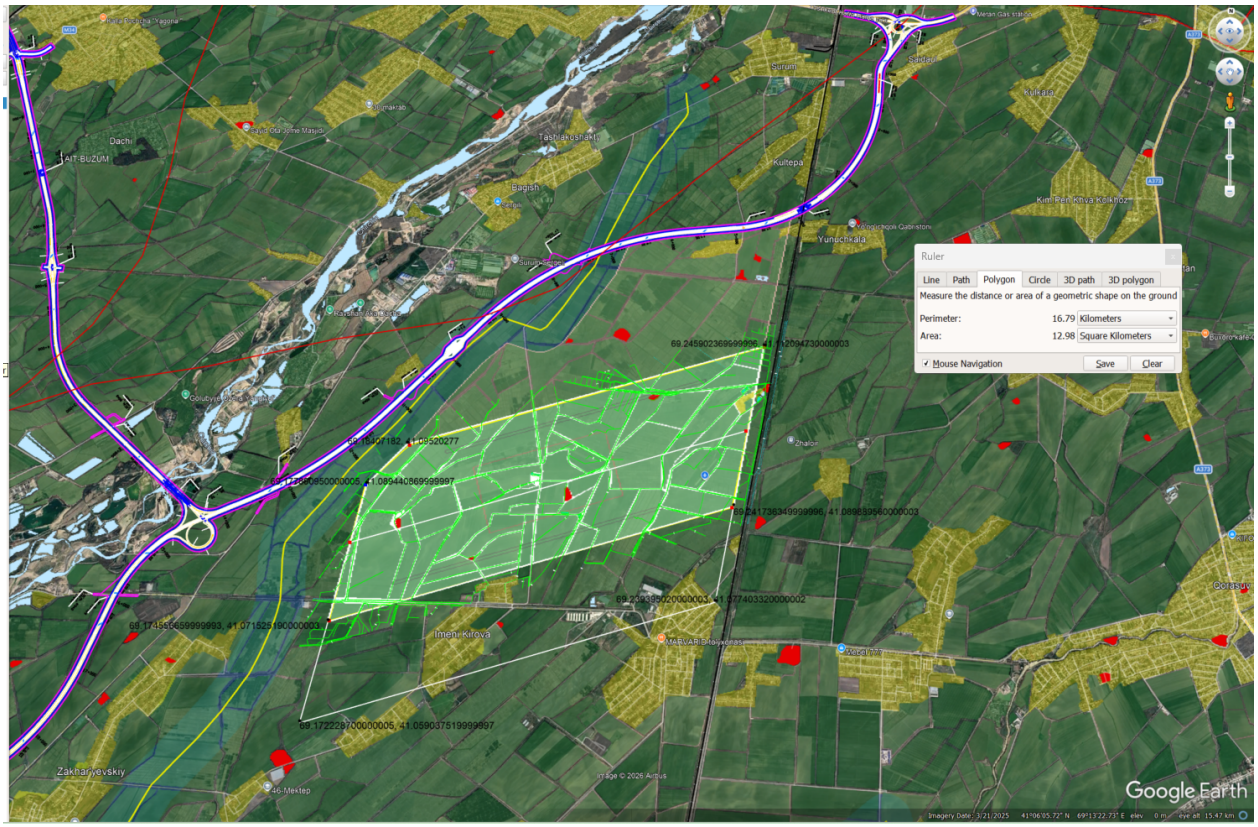


Figure 2-1: Site location map

Topography and Land use

The project site topography is relatively flat with ground elevation between 328 to 396 mASL within an intensively managed agricultural landscape. Surrounding land use is dominated by irrigated cultivation, supported by a dense network of surface irrigation and drainage channels. Rice cultivation is understood to be widespread in the area, with fields typically seasonally flooded during the growing period (generally April–October), implying sustained water application across large areas.

Topographic relief across the site is limited, with no significant local features expected to materially influence groundwater flow patterns.



Figure 2: Artist's Impression : Tashkent Airport Visualisation (Haskoning, 2024)

Meteorology

The climatic characteristics of the area (Haskoning, 2024) are derived from the Tashkent Meteorological Station Data (SHNK 2.01.01-22 "Climatic and Physical-Geological Data for Design"). The development area belongs to subzone IV G according to climatic zoning (SHNK 2.01.01-22). The region is characterized by a sharply continental climate with significant daily and seasonal temperature variations, low precipitation, and uneven distribution throughout the year. Summers are hot and dry. Winter average temperatures are below freezing, and seasonal soil freezing can occur. ***Note: no climate change predictions or amendments have been applied in this review.*** Wind characteristics are provided as are temperature and humidity variables, no further evapotranspiration analysis has been undertaken at this stage although evapotranspiration in this continental setting is expected to be high and a significant modifier to effective rainfall and infiltration rates. Humidity is relatively low ranging from 22% to 55% (hottest to coldest month) and average monthly summer temperatures from May to September all exceed 20°C and reach maximum in July of 28°C.

Annual precipitation is moderate and strongly seasonal, typically of the order of 400 - 600 mm per year, with most rainfall occurring in late winter and spring and very limited rainfall during the summer months. Winds are generally moderate and seasonally variable, with no dominant extreme wind regime influencing site conditions.

Hydrology

The Chirchik River is a major river with a basin area of 13,112 km^2 in the Tashkent Region which has an open channel, as part of a highly braided central flow-line approximately 1900m northwest from the boundary of the development site. The Chirchik is gauged and open source hydrometric records exist for Gauge Point 16275 located approximately 41km downstream at the confluence with the Chinaz River and Gauge Point 16262 (Gazalkent) located 90 km upstream just above the Ata tributary confluence. The section of the Chirchik that flows through Tashkent is identified[1] as the zone of water distribution and consumption.

The area of the development lies within the alluvial flood plain of the river, whose active width is nearly 1 km wide. The myriad network of irrigation drainage courses connects and drains to the Chirchik river. The network comprises primary and secondary irrigation and drainage canals. Based on survey data and site observations, individual channels are typically approximately 3 to 5 m wide and 1.5 to 2m deep (shown in Figure 3 and Figure 4).

Local enquiries and site reconnaissance indicate that:

- Most irrigation canals are unlined, with some locally lined using concrete panels;
- Where lining is present, joints are often open or poorly sealed, and canals are not water-tight, resulting in seepage;
- Stormwater drains are generally unlined and routed towards the Chirchik River, commonly along road corridors;
- Irrigation flows are seasonally elevated, coinciding with the rice-growing period.



Figure 3: Pictures of irrigation drainage courses

[1] Ch.3.2.1 Chirchik River Basin Characterisation, Applied Modeling of Hydrological Systems in Central Asia, Siegfried, Marti, 2021.

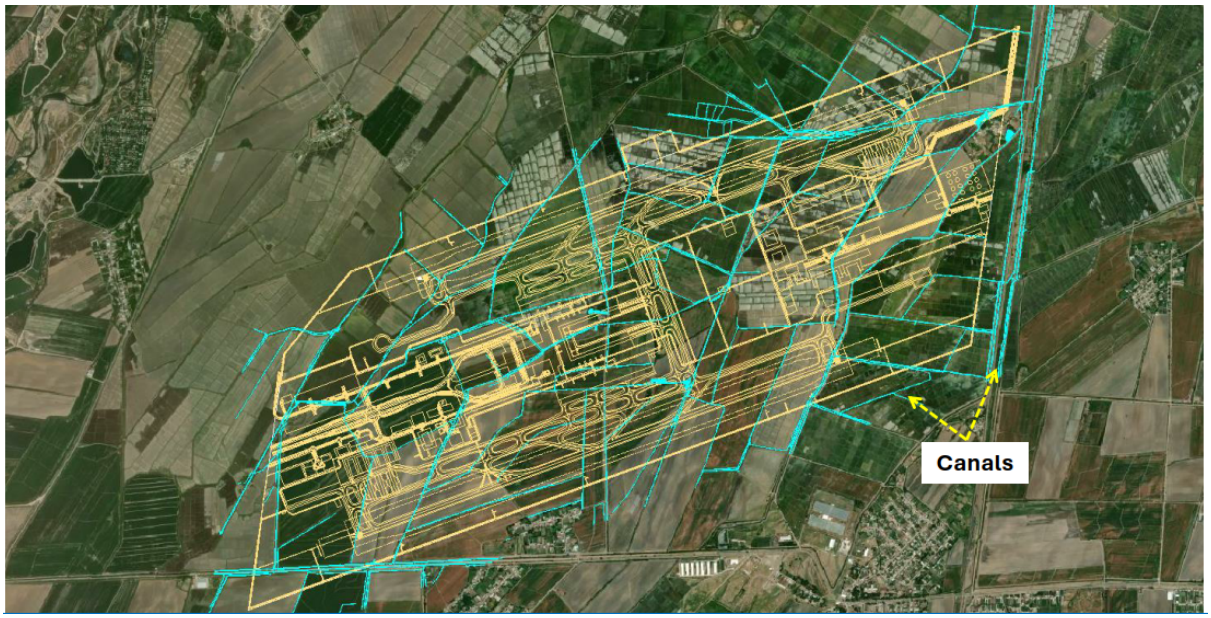


Figure 4: Iirrigation canals within project footprint

Geology

The superficial geology of the area (no bedrock geology is encountered) comprises Quaternary system deposits represented by alluvium in terrace deposits. The Chirchik alluvium consists predominantly of gravel and conglomerates, forming a series of terraces with uniform composition consisting of well-rounded pebbles interspersed with gray quartz sand. The conglomerates are typically calcareous in cementation.

The unit reflects modern period deposits (Syrdarya complex QIVsd) depositionally derived from embedded cold-steppe complex floodplain and the 1st and 2nd upper terraces of the Chirchik River. The deposits consist of gravel covered by a thin layer of fine loam. The gravel of the second terrace exhibits compaction and signs of limestone cement. Some parts of the gravel contain nests of conglomerates. The thickness of the gravels forming the first upper terrace and the floodplain ranges from 5 to 10 meters. The terraces are almost identical in litho-type and do not represent a discernible geological contact for the purposes of this hydrogeological appraisal. The clayey soils consist of loess-like clays with interlayers of loam, light-brown in color, with a hard plastic-consistent texture. Coarse-grained gravels are saturated.

Information on site geology has been obtained from Haskoning, 2024 Geotechnical review. The results of the ground investigation (GI) identified a three layer, alluvial geological system comprising an upper silt, an intermediate sand and an extensive, deep unit of sandy gravel with large cobbles and boulders. Refer to Table 1.

Table 1: Geological profile (Haskoning, 2024)

Table 22. Engineering- geological formation

Description	Short Description
Low plasticity Silt Clayey (CL) hard density	EGE-1
Average Thickness: 2.0 meters Thickness Range: Minimum 0.0 meters to Maximum 2.9 meters EGE-1 is identified as the first geological layer with a thickness ranging from 0.0 meters to 2.9 meters. This layer is described as light brown, low plasticity silty clay (CL) with hard density.	
Fine grained sand (SW-SP) of yellowish color from wet to water saturation hard density	EGE-2
Average Thickness: 1.2 meters Thickness Range: Minimum 1.0 meters to Maximum 1.5 meters EGE-2 is identified as the second geological layer with a thickness ranging from 1.0 meters to 1.5 meters. This layer consists of fine-grained sand of yellowish color, from wet to water-saturated, with hard density. This layer is found specifically in boreholes BH-103, BH-110, and BH-204.	
Sandy gravel with large boulders and cobbles. (GW-GP)	EGE-3
Average Thickness: Greater than 1.5 meters Thickness Range: Extends throughout the borehole depths EGE-3 is identified as the third geological layer. The thickness of this layer extends beyond the typical borehole depths. This layer is composed of coarse blocky materials and is classified as sandy gravel with large boulders and cobbles (GW-GP).	

Of significant interest to the hydrogeological assessment is unit EGE-3, the third geological layer, reported by Haskoning to extend to a maximum depth of 50 meters comprising sandy gravel with large boulders and cobbles and according to geotechnical classification systems (BS5930, ASTM D2487) essentially consisting of well graded to poorly graded gravels with significant medium /coarse sand, with gravels taking up more than 50% of the coarse fraction and sand taking up more than 12% of the remaining fraction. Haskoning stated this is a high permeability, heterogenous granular, matrix supported (load bearing) mix. No cohesive components (clays , silts) were reported. Haskoning reported (Table 23) that the USCS grain size distribution for the units was:

1. EGE-1 (Silt Clay): 73.1% silt, 17.1% sand, 9.8% clay
2. EGE-2 (Sand): 65.3% sand, 28.2% gravel, 4.3% silt and 2.2% clay
3. EGE-3 (Gravel): 83.3% gravel, 15.2% sand and 1.5% silts and clays.

The uniformity and distribution of the EGE-3 unit across the development site appears to be consistent. The gravel units are expected to be laterally extensive and highly permeable, forming a shallow unconfined aquifer.

3.0 Hydrogeological Conceptualisation

The site is underlain by unconsolidated alluvial sediments, comprising:

- A shallow layer of fine-grained silts and soils at surface;
- Underlying coarse gravel and sand deposits (EGE-3) , interpreted to be laterally extensive.

The EGE-3 gravel unit is inferred to form a shallow, unconfined aquifer with good hydraulic connectivity across the development area and extending in hydraulic continuum to the Chirchik River fluvial zone. Haskoning installed ten, relatively equally spaced monitoring

wells across the site (Figure 5). The wells were drilled to 35m bgl and penetrate the EGE-3 aquifer by approximately 32m, construction details of the wells (i.e screens, casing and response zones) are unknown.

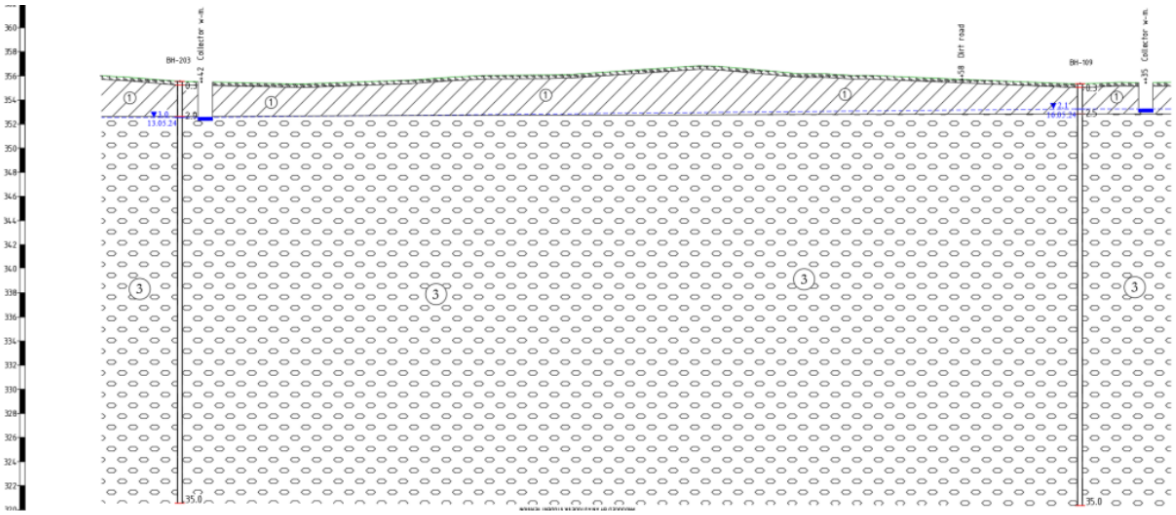


Figure 5: Monitoring well and EGE-3 profile (Haskoning, 2024)

Appendix C of the Haskoning report includes 20 large scale cross-sections, constructed along transects between monitoring wells which show a homogenous geological profilesimilar in features to the cross-section shown in Figure 5. This indicates the EGE-3 unit is continuous, consistent and present across the development area, the overlying silt units (EGE-1,2) are relatively consistent in thickness and represent the unsaturated zone (above the water table) and irrigation channels tend to be incised into the silt unit but their base intersects with the gravels and saturated zone.

The conceptualisation of the EGE-3 aquifer is as follows (some notes obtained from Haskoning, 2024):

- A shallow, unconfined aquifer recharged from infiltration of surface water, the irrigation network, field irrigation and meltwater from snow and ice. Rainfall plays a relatively insignificant role in recharge as the evapotranspiration rates are likely to be very high.
- According to long term monitoring data (Haskoning, 2024) the highest groundwater level is observed from July to August and the lowest from December to February, the annual fluctuation is approximately 1.5m. Long-term maximum values are likely to be approximately 1m higher than observed and this means they will be at a depth of 0.8m below ground level. The reason behind the reported higher water levels in summer is not clear and cannot be verified at this stage.
- Groundwater losses occur through evaporation, transpiration and abstraction.
- The aquifer is in hydraulic continuum with the drainage channels.

Hydraulic Properties

Hydraulic Conductivity (Permeability)

The coarse gravel and sand deposits are expected to be extremely permeable with high intergranular hydraulic conductivity (K) and porosity, consistent with unconsolidated Quaternary alluvial sediments. Hydraulic conductivity in such materials typically varies over range and is strongly influenced by the relative proportions of gravel, sand, silt and fines.

Where coarse gravels dominate and fine material is limited, upper-range K values are expected. Literature values (Freeze and Cherry, 1979) reference gravel K[2] between 1 and 10^{-3} m/s and porosity between 25-40%. Lower values can be expected where finer fractions are more prevalent.

Such conditions indicate high transmissivity i.e. yield in the aquifer and rapid groundwater movement with a strong hydraulic response to excavation and pumping.

An empirical / theoretical relationship between particle size and permeability has been analysed to see if a realistic permeability value can be estimated from 211 Particle Size Distribution (PSD) graphs included in the Haskoning report from soil samples collected from borehole drilling. The results indicate a high degree of consistency aside from BH101 and BH209 with uniformity coefficients (Cu) between 1-3 and D_{10} between 20-35mm.

However, gravels do not meet all the assumptions for the equations and the resultant K values are considered to be unrealistically high:10^{-1} to 10^{0} m/s, and for some even 10^{+1}.

Using the Hazen method, most of the Cu values fall within the acceptable range (Cu < 5), apart from BH101 and BH209, though D_{10} values (20-38mm) fall well outside the range the Hazen method was validated for (0.1-3mm). The K results are generally around 10^{-1} m/s.

The Kozeny-Carman method requires laminar/creeping flow, which for coarse gravels is unlikely to be the case. Using D_{10} = 10-30mm and an assumed porosity of 0.3, it gives K = 10^{-1} to 10^{0} m/s.

USBR method (Vukovic & Soro, 1992), using D_{20} value = 25mm gave an unrealistic K = 21 m/s.

All results point to the K values for these gravels being very high.

Topsoil: The K values for the top 0-2m are either the same as the gravel or jump down to 10^{-8} - 10^{-9} using both Hazen and Kozeny-Carman.

Outliers - BH101 and BH209: These were the only ones that stood out, though they seem similar to each other and give consistent Cu between 20-60 and D_{10} values between 0.3-2.4mm, giving K values of 10^{-3} - 10^{-4} m/s. this was through the whole 30m of BH01 and 10m of BH209.

[2] Hydraulic conductivity in unconsolidated, inter-granular porous material such as gravel is the same as permeability for water flow . The terms are used interchangeably,

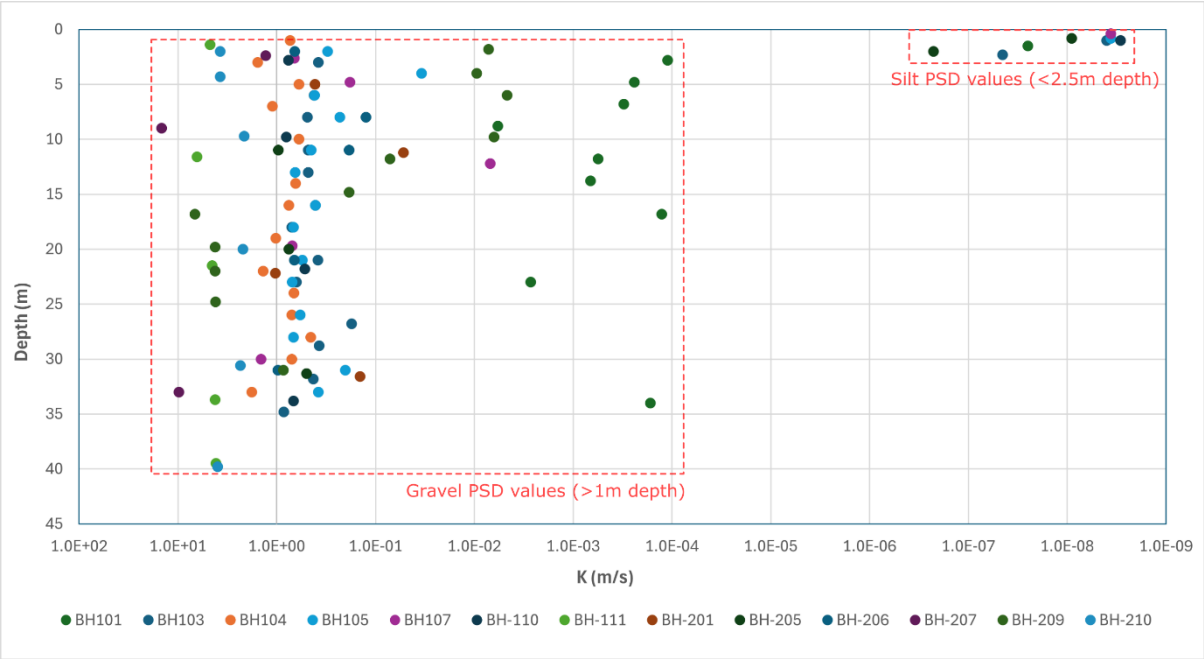


Figure 6: Hazen analysis

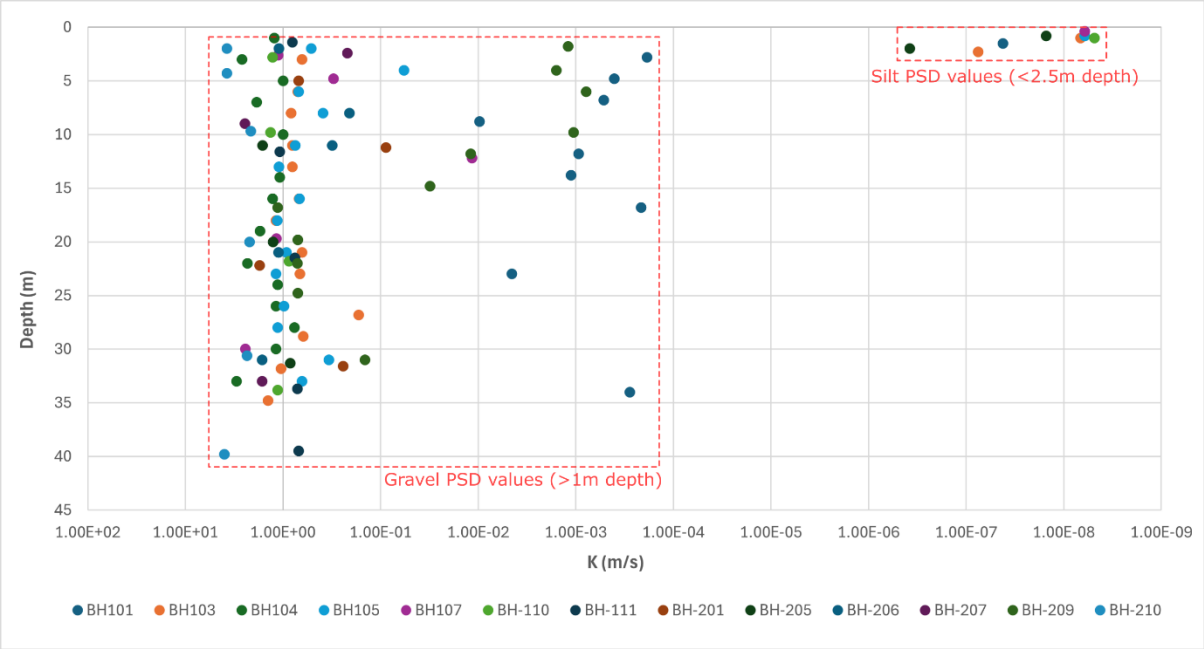


Figure 7: Kozeny-Carman analysis

Porosity and Storage (Specific Yield)

Haskoning reported (Table 23) water content of the units, the methodology for this is unclear, the results appear to reflect residual retention of water in laboratory samples, potentially strongly affected by drainage of the samples in ex-situ conditions. The results show the more cohesive sample (EGE-1) retains more water than the gravel sample (EGE-3) indicating (qualitatively) that the specific yield of the gravel (its drainage potential) is high.

1. EGE-1 (Silt Clay): 21.6%

2. EGE-2 (Sand): 19.1%

3. EGE-3 (Gravel): 10.9%.

The results are indicative only (by inference).

Elsewhere (Appendix B) Haskoning reported average porosity of the gravels with sandy aggregate up to 35% water-saturated with a limited number of samples reporting higher porosity (40-50%). The latter values are exceptionally high and if correct indicate completely clean gravels with no interstitial material in pore space.

Pore-water Pressure

Hydrostatic water pressure, calculated from liquefaction analysis on three samples (Haskoning, pp103) was determined by depth increment and results are plotted in **Figure 8**. These indicate a progressive and consistent increase in head with depth through the EGE-3 profile with no indication of sub-units, differential pressure steps or compartments / confining sub-layers.

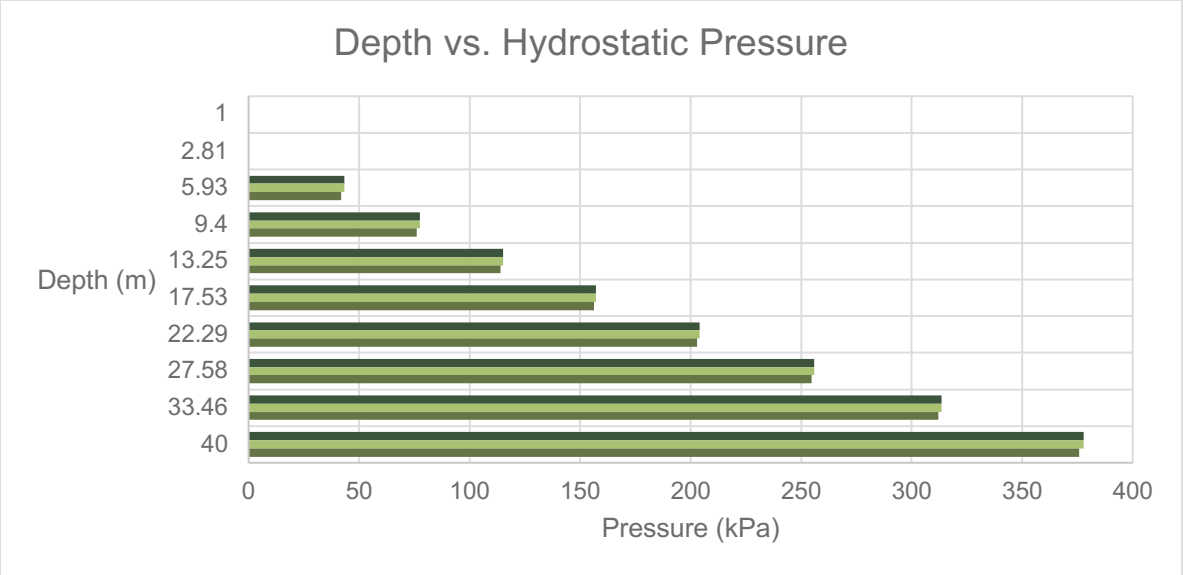


Figure 8: Depth Vs Hydrostatic Pressure (Haskoning, 2025)

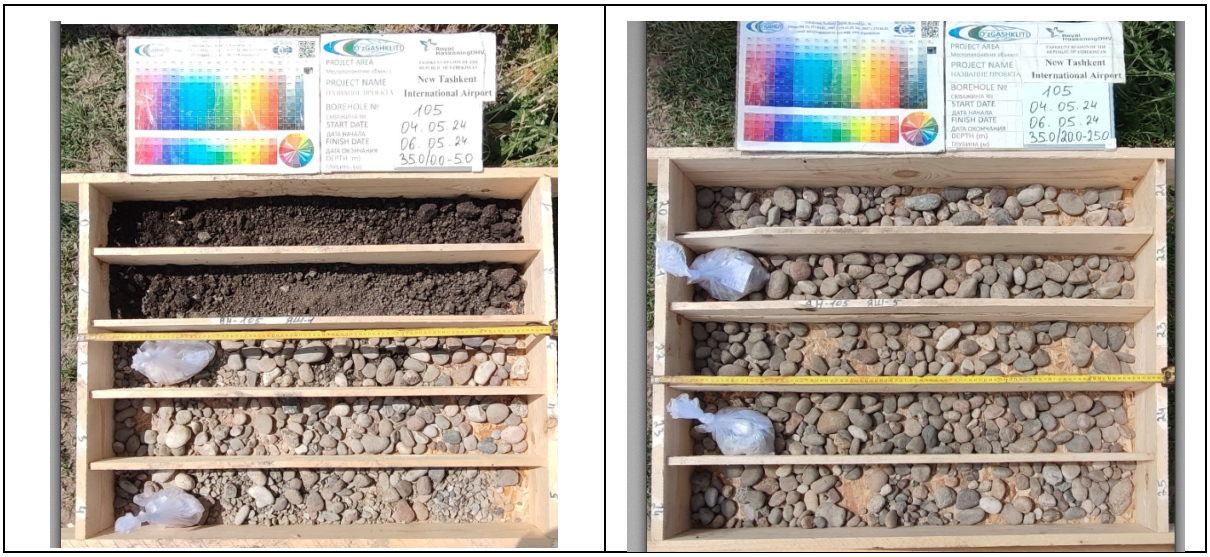


Figure 9: Soil samples, A) Upper 5m, Silt and Gravel, B) 25m depth Gravel only (Haskoning, 2025)

Infiltration

Double Ring Infiltration (DRI) tests were conducted to assess the infiltration rate of surface water at the project site. Based on the DRI test results, the infiltration ratio (k) values ranged from 0.00096 cm/sec to 0.00110 cm/sec. The results can be defined as “Moderate” in terms of infiltration characteristics.

Hydraulic Gradient and Piezometry

Given the flat topography, groundwater flow is interpreted to be controlled primarily by regional hydraulic gradients within the alluvial system; and localised recharge from irrigation infrastructure. Groundwater flow direction is inferred to broadly follow the regional gradient towards the Chirchik River system.

The top-of-casing location and vertical elevation of each of the wells was surveyed and depth to water table recorded in eight piezometers. Recordings were taken on four occasions between the end of May and middle of June 2024. The range of water level fluctuation recorded in this time is between 0.05 and 0.1m in all wells with no clear trend of whether water levels are declining or increasing. The resulting piezometric (water table) map is shown in Figure 10. Groundwater was encountered between 1.2 and 2.8m below ground level representing a water table elevation of between 353.1 to 364.4 mASL. Hydraulic gradient and piezometry is currently defined as:

- Groundwater was encountered in all boreholes. Based on the groundwater monitoring record, the groundwater depth ranged from 1.2 to 2.6 meters, (Average 1.8 meters) below ground surface level across the site.
- There is a relatively uniform and shallow hydraulic gradient of 0.0034[3] with flow towards the southwest and the groundwater regime is likely closely linked to the Chirchik River as a major discharge zone.

[3] As determined from groundwater levels in BH105 (367mASL) and BH108 (351mASL) separated by a lateral distance of 4730m.

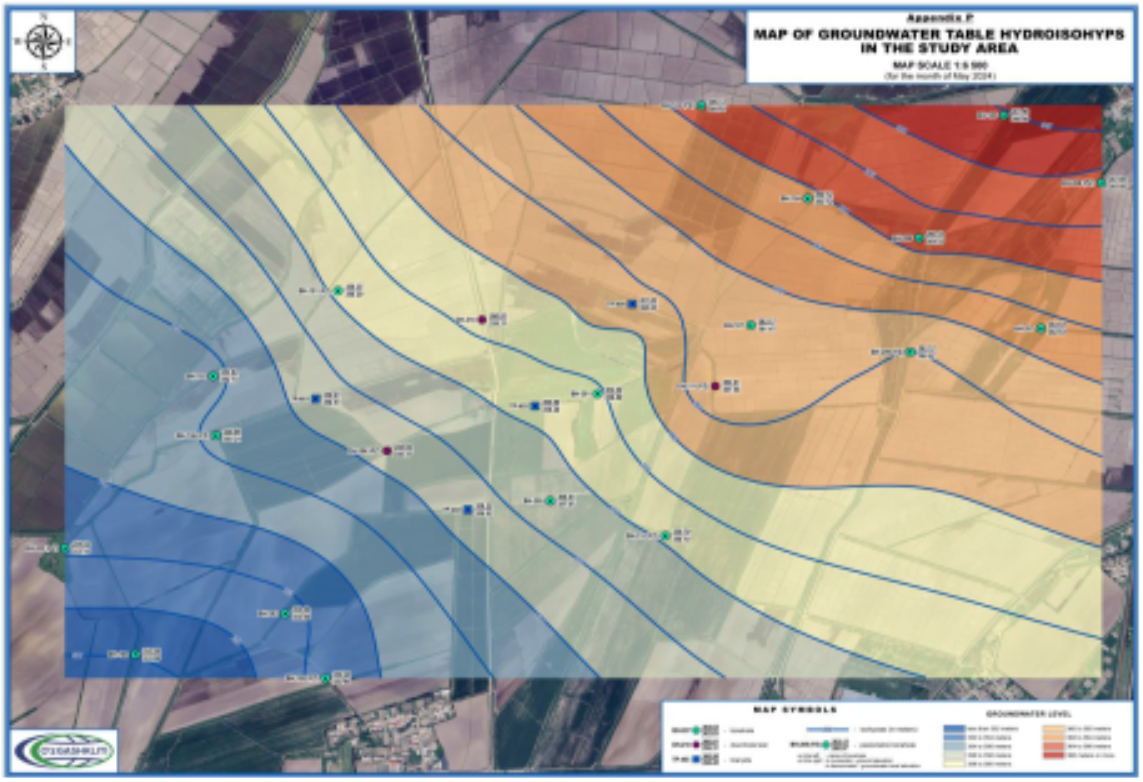


Figure 10: Water Table

Site-specific values remain subject to confirmation through field testing.

Recharge

Recharge to the shallow groundwater system is interpreted to occur primarily via seepage from irrigation canals and infiltration from irrigated agricultural fields, including seasonally flooded rice paddies. Direct recharge from rainfall is considered secondary relative to irrigation inputs.

The irrigation and drainage network is closely coupled with the shallow aquifer. Irrigation canals act predominantly as a source of groundwater recharge, rather than as effective drains, particularly where channels are unlined or poorly sealed. Stormwater drains are generally shallow and intercept only near-surface water, with limited influence on the underlying gravel aquifer.

Seasonal irrigation practices are therefore expected to sustain persistently shallow groundwater conditions, with limited opportunity for natural drainage of the aquifer at the local scale.

4.0 Hydrological Effects from the Planned Airport Construction

The airport construction concept is based on preliminary design information, including the TIA Input to Vision Invest for ESIA (19.01.26) and the Preliminary Details of Diversion Canals presentation. The description below is therefore indicative and subject to refinement as design develops.

4.1 Dewatering and Groundwater Interaction

The main terminal building will mainly be above natural ground-level with the exception of a basement associated with incoming baggage handling. The terminal building is shown in the red polygon in Figure 11

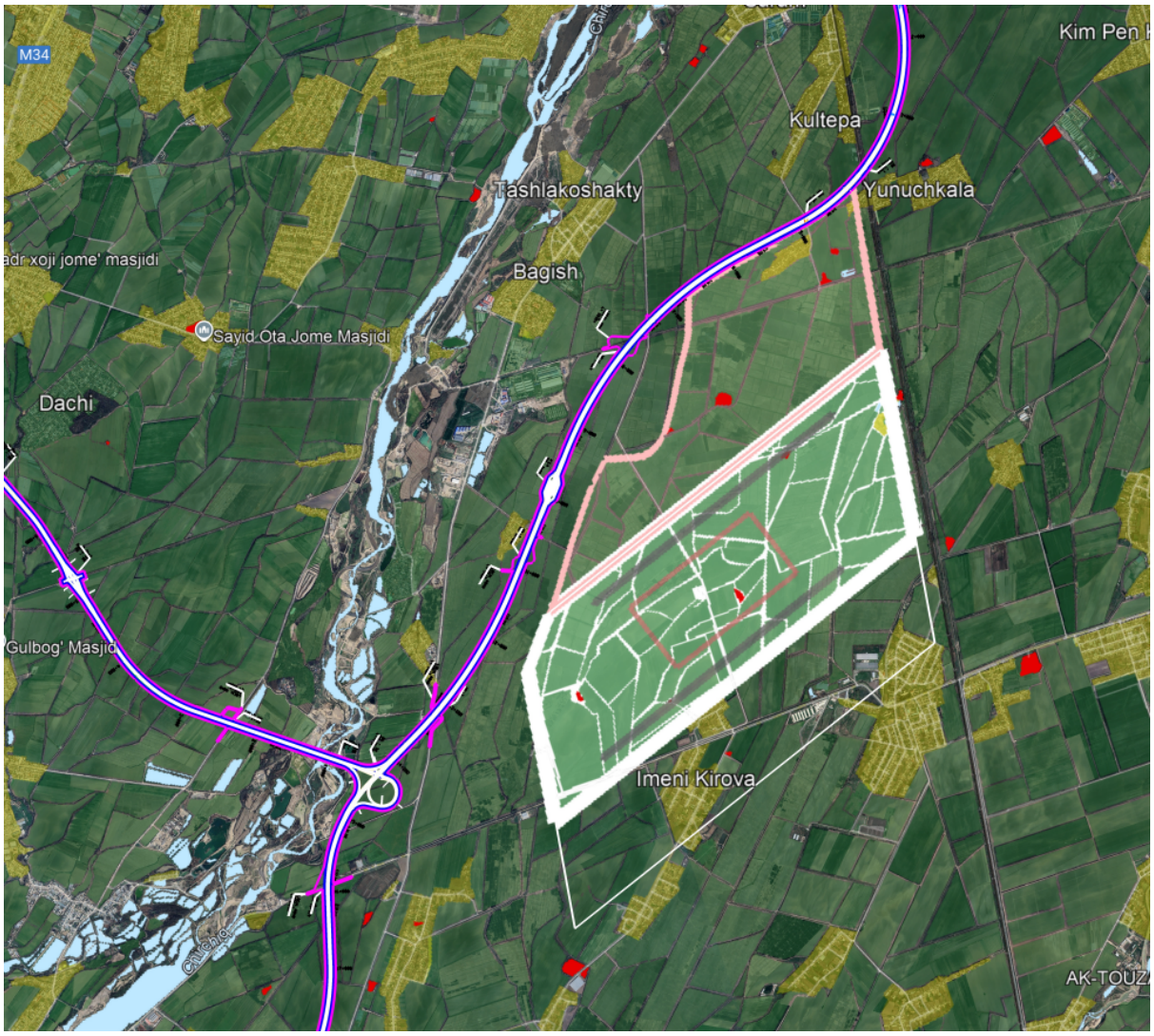


Figure 11: Site setting location of Airport terminal

High level plans have been provided (TIA Input to VI for ESIA – 19.01.26.ppt) indicating the location of main facilities which require basement / deep excavation (along with excavation levels). In total seven airport facilities are expected to require 'deep' excavation as shown in

Table 2 below and on Figure 12.

Table 2: Expected deep excavation.

S.No	Asset Name	Asset No.	Existing NGL	Excavation Level	Founding Depth
1	Passenger Terminal Building	01	358.4	3.8	354.6
2	Sewage Treatment Plant	20	355	7.5	347.5
3	Aircraft Sewage and Potable water	21	358	5.1	352.9
4	Common Utility Plant	26	356	7.2	348.8
5	Potable water Treatment Facility	35	356	8.5	347.5
6	Power Distribution – Load Centre A	14	363.5	4.5	359
7	Power Distribution – Load Centre B	14	354.5	4.5	350

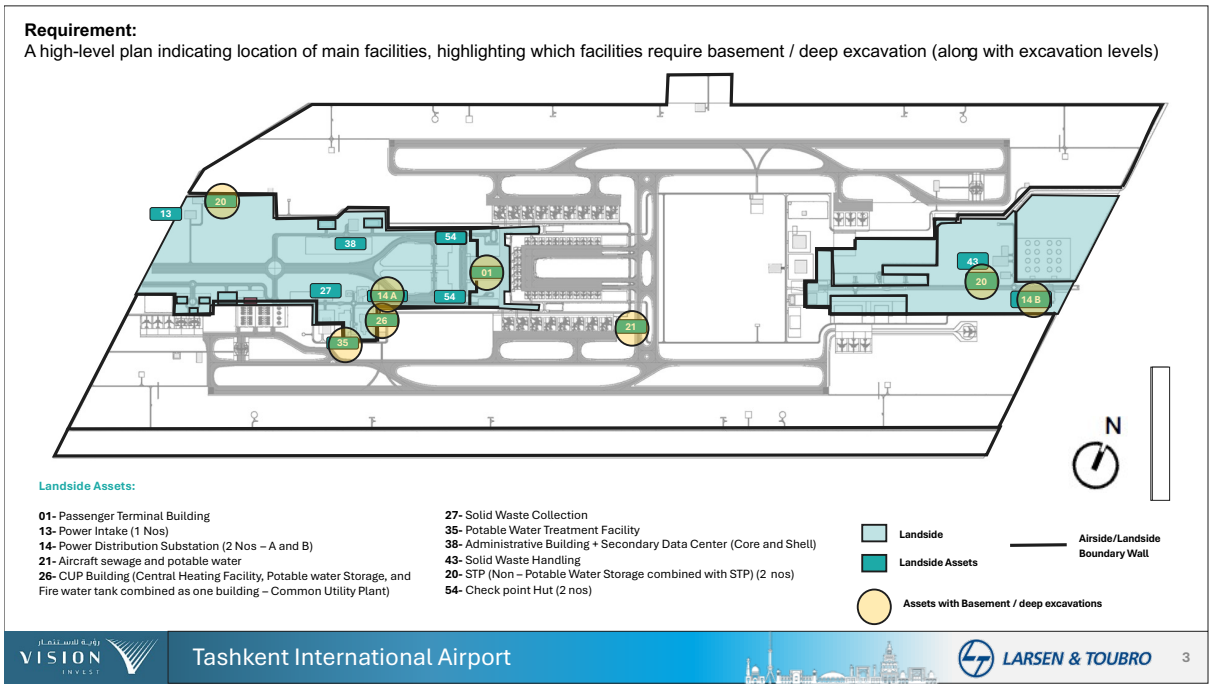


Figure 12: Airport Terminal main Facilities with sub-water table excavations

Assuming an average depth of water table of 1.8m below ground level, then the excavations will penetrate between 2 to 6.7m into the water table. The largest exaction is the basement of the baggage handling system (BHS) within the terminal building which has a footprint of 13,152m^2 and will penetrate 2 m below the water table. There are sub-elements of this facility that may extend lower such as the chambers required for vertical transportation and MES shaft. These sub-elements are relatively smaller each having a footprint of approximately 50m^2. The construction technique for building a large, sub-water table, basement facility also needs to be considered for dewatering as the chamber walls will need

to be constructed within a dry environment requiring a set-back distance and (unconfirmed) some form of hydraulic isolation such as sheet-piling, diaphragm walls and/or coffer dam that has a larger footprint than the actual finished build.

Complex groundwater management issues including control of lateral pore-water pressures, buoyancy effects and mitigation against basement up-welling and water-proofing as well as dewatering and groundwater management will require design. The design section through the PTB basement shows natural ground level at 358.40mASL, a construction base level of 354.6mASL upon which a liner and raft foundation would be placed. An angled excavation slope of 66° is shown illustratively to indicate the construction set-back distance and backfill needed. In reality given the saturated conditions of the unconsolidated gravels a realistic angle, taking into account hydrostatic pressure would be much lower.

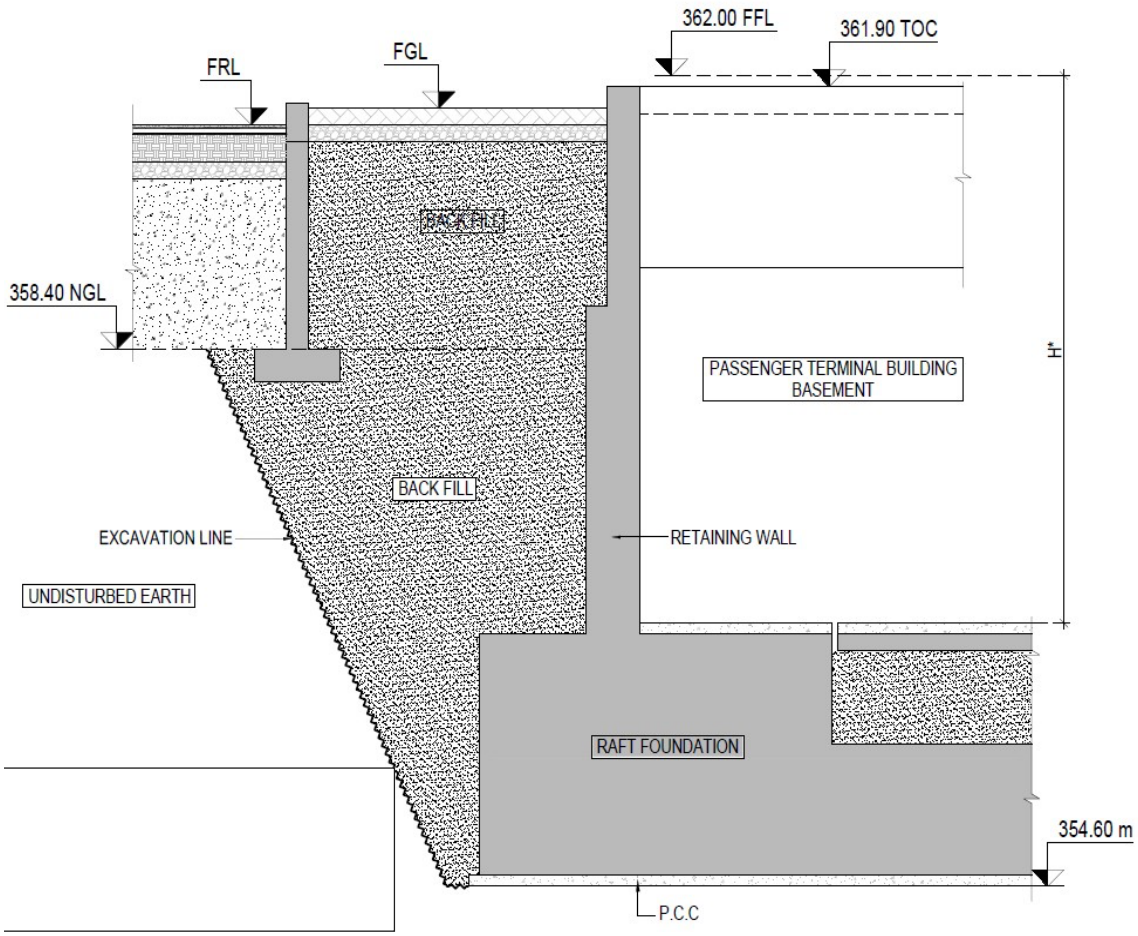


Figure 13 Cross section of excavation chamber for Passenger Terminal Basement

Preliminary construction diagrams indicate that foundation levels and excavations extend to approximately 3.4 m below existing ground level. Given likely shallow groundwater conditions, excavations of this depth are expected to intersect the groundwater table and require active dewatering during construction.

Also, given the water table is expected to fluctuate approximately 1.5m each year, under different seasonal conditions or inter-annual events, it is plausible that general earthworks, excavation and construction could intersect with a high water table and require groundwater control measures.

Dewatering Impacts

Shallow groundwater, high K values (order of 10^{-3} m/s) and relatively large volume construction chambers suggest that shallow and deep excavations are likely to require significant dewatering measures. Drawdown is expected to extend beyond the construction footprint, potentially over a wide area. Hydrogeological impacts include:

- High rates of groundwater inflow;
- Dewatering of even a few metres will likely produce large areas of drawdown;
- Challenging control measures in coarse gravels that associated with basal heave, uplift and buoyancy within excavations which could lead to further excessive and dewatering duration and rates;
- Management of significant pumped water volumes during construction.

Dewatering-induced drawdown has the potential to create adverse effects on irrigation, communities who use wells for domestic and livelihood support and agricultural receptors:

- Create underdrainage of soils over a large area and alter shallow groundwater conditions supporting irrigated agriculture;
- Increase seepage losses from nearby irrigation canals and collector drains;
- Change local groundwater gradients and flow paths.

Large scale dewatering also has the potential to:

- Result in adverse resource loss as excessive quantities of water need to be pumped to waste In the sense that the water is no longer deployable locally, even though it may be discharged to the Chirchiq River);
- Cause water quality changes within the aquifer by drawing in water from different units, elevations and potentially altering oxidation levels and other hydrochemical effects;
- Cause incompatibilities in water quality, erosion and exacerbating flood risk in the receiving water body;
- Potentially there are also opportunities (positive impacts) if dewatering can be selectively used to assist in water supply and stream support flows, however the principal approach for dewatering for new developments is that the dewatering measures should be temporary and not result in a permanently altered hydrological regime.

While these effects cannot be quantified at this stage, they represent a credible pathway for indirect environmental and socio-economic impacts and warrant consideration within the ESIA.

4.2 Drainage Diversion

As part of the surface-water management strategy, two lined diversion channels are proposed around the airport boundary. These channels are intended to:

- Intercept irrigation canals crossing the site;
- Convey catchment runoff and irrigation flows;
- Discharge diverted flows back to the Chirchik River downstream of the site.

The diversion channels are intended to divert inflowing irrigation channel water away from the development site. The channels are designed to pick up multiple inflow points along the site boundary, an estimate of flow from the Irrigation Department suggests that around 8 m^3/sec irrigation water ingresses for the Chirchiq region[4]. Estimates for diversion channel sizing have been based on surface water catchment inflows (basin size, Mannings number etc). The sizing has been designed for maximum surface flow events. It is unclear if any assumptions for groundwater contribution have been included.

The channels are therefore assumed to be surface-water control structures, they should be regarded primarily as drainage and flood management measures. In highly permeable gravel deposits, lined channels are unlikely to act as effective groundwater cut-offs, and diversion of surface flows is not expected to prevent groundwater inflow to excavations where the aquifer is shallow and laterally extensive.

Accordingly, diversion channels alone should not be assumed to materially reduce construction dewatering requirements without further hydrogeological assessment and whilst cutting deeper channels would potentially drain out more of the aquifer, the net environmental impact effects would increase significantly. There would be diminishing benefit for incising deeper channels as the inflow rates would escalate and the hydraulics of the diversion would be impractical.

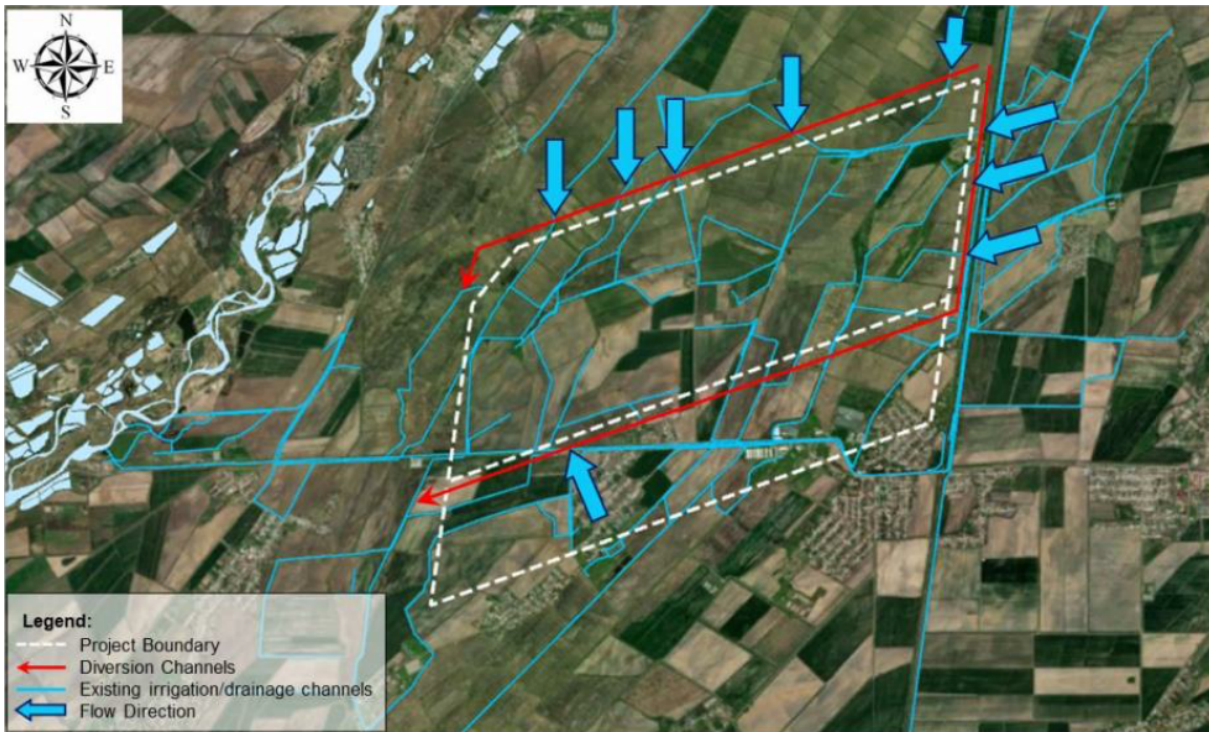
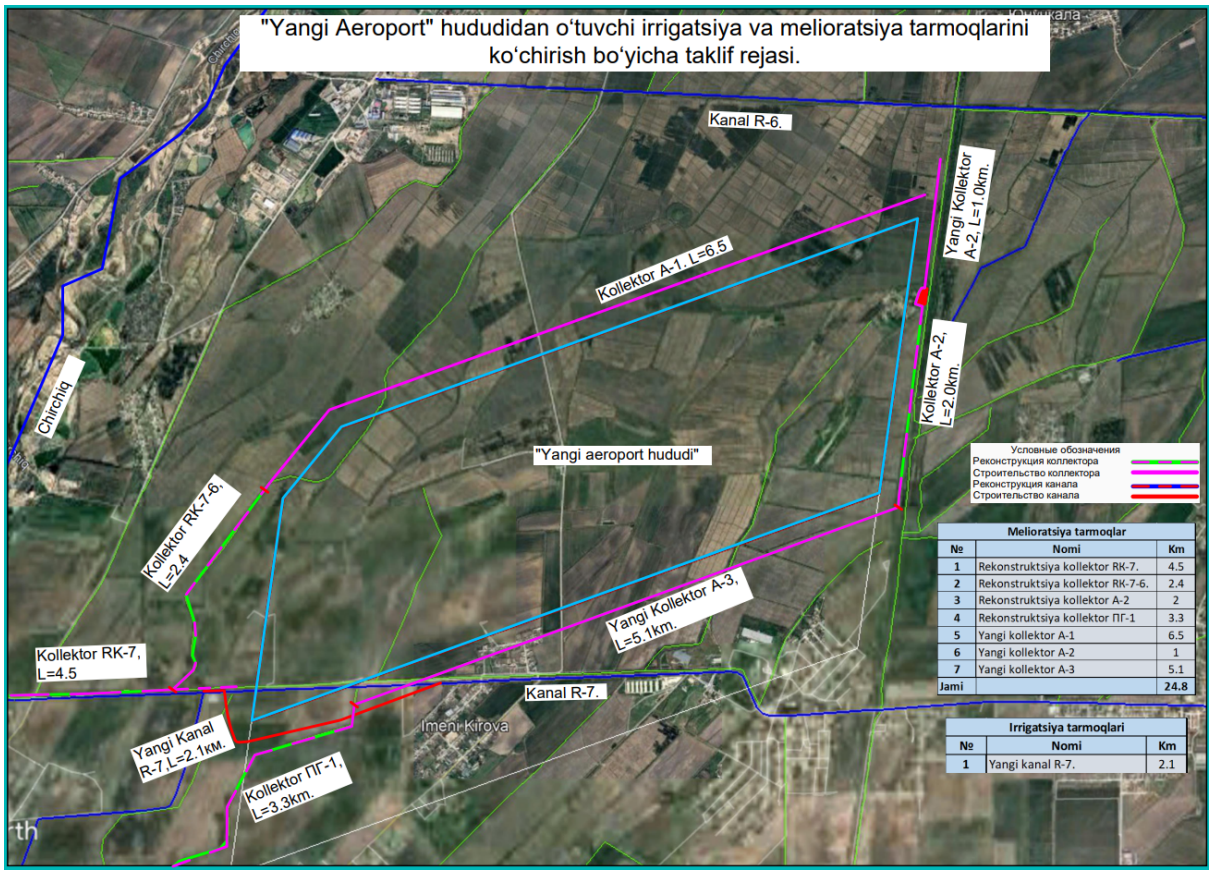


Figure 14: Drainage Diversion Concept

Note: The lower dotted line polygon shown above is not part of the project footprint.

The diversion concept has been developed into a system of individual collector drain segments connected to the two main drainage lines. As the collectors join they require progressively more drainage capacity downstream and their channel cross-sections have been sized to increase from nominal trapezoid channels initially at 2.5m depth to larger collector channels reaching 4m depth which will cut into the saturated gravels.

[4] TNGHIA Presentation Hydrology



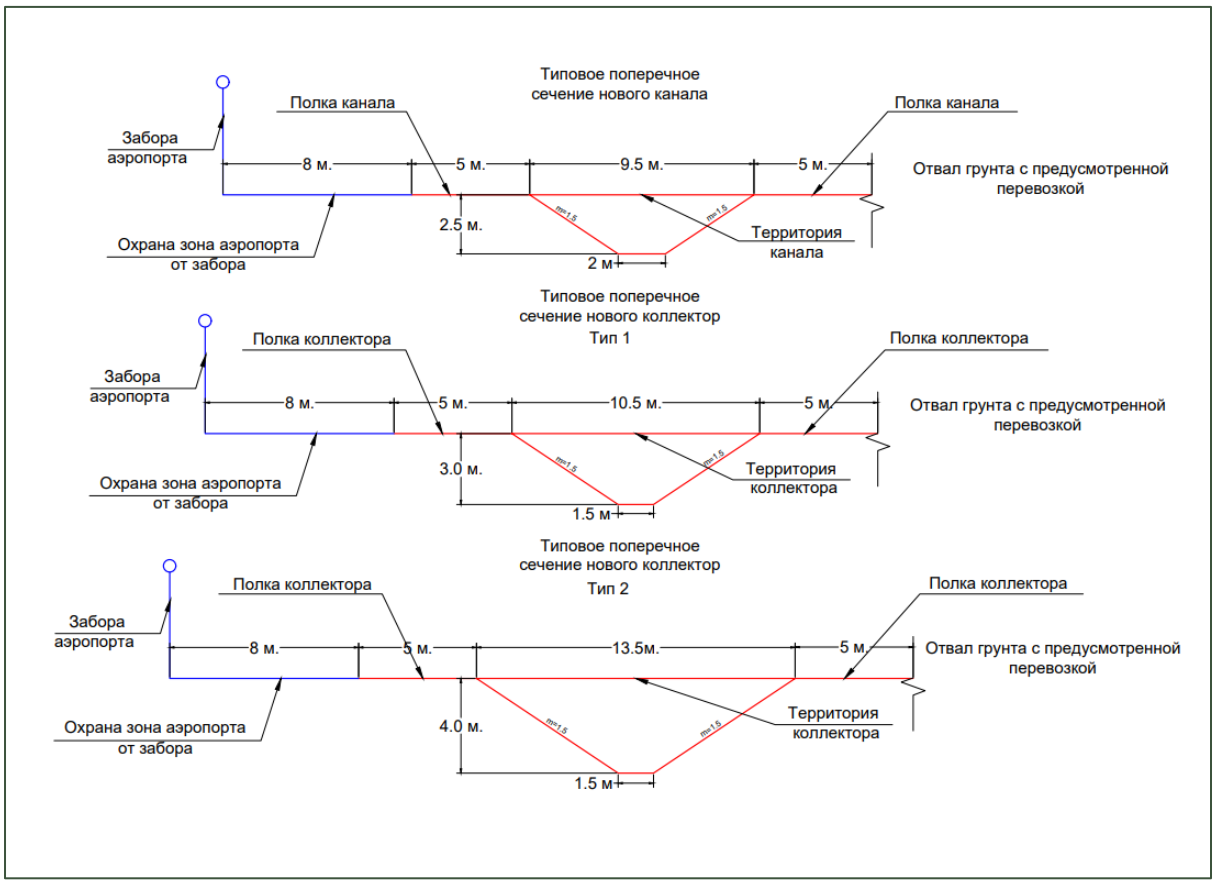


Figure 15: Collector system A) segments, B) cross-sections

Drainage Diversion Impacts

The diversion channels will alter the irrigation network and replace an interconnected, criss-crossing system with two large channels that will effectively drain faster than the previous system. This could potentially lead to an overall reduction in water level across a larger area over time as the system adjusts to a new water balance and modified hydraulics.

4.3 Other Aspects

Foundations and earth works will be required for the airport structure. Foundation details are not confirmed and may comprise combinations of pad, raft and piling. Preliminary calculations (Haskoning) indicated 4000 kN bearing capacity piling techniques would require friction or floating piles to be installed to lengths between 16 to 30.5m depth with pile diameters up to 1m. No realistic presence of bedrock precludes end-bearing piling. If piles, raft or pad foundations, substantial quantities of cement will be cast. This creates several hydrogeological risks including:

- Increased suspended sediment in the aquifer from cementing near or beneath the water table.
- Foundation risks including dragging contaminated material to depth and creating preferential contaminant transport pathways (piling).

Other potential aquifer disturbance activities include structural fill emplacement, soil improvement amendments (to address liquefaction issues in soils) emplacement of utility lines (water, sewage, oil pipes and cable conduits, etc.) at or below the water table with corrosion protection coatings (bitumen / epoxy paint).

Drainage, soak-aways and localised flooding can all introduce physical and chemical water impacts to the surrounding area.

5.0 Dewatering, Groundwater Management and Modelling

The primary dewatering and groundwater control activity that is evaluated in this initial assessment is a central basement located in the terminal building. The basement is intended as the baggage handling system (BHS) within the terminal building which has a footprint of 13,152m2 and will penetrate approximately 2 m below the water table (to be confirmed). There are sub-elements of this facility that may extend lower such as the chambers required for vertical transportation and MES shaft and in total eight facilities are identified as extending below the water table and requiring groundwater control for construction.

Groundwater control and management will require a combination of measures to be effective given the permeability and water table is so high, including:

- Active dewatering systems (e.g. wellpoints or deep wells to create advance dewatering zones);
- Sump dewatering and mitigations to reduce inflow.
- Effective modelling and planning to manage the inflows and have sufficient infrastructure at hand for pumping and piping. This will require additional site investigation work and pumping test pilot trials;
- Careful phasing of excavation works;

- Evaluation of the additional effect of the drainage diversion system;
- Management of pumped water, potentially integrating reuse within construction or irrigation systems where water quality permits.

Given the anticipated large radius of influence of dewatering due to the high permeability of the gravels, groundwater modelling is necessary to evaluate and optimise any system to assist with construction design, water management infrastructure and avoid excessive water resource withdrawal impact. Groundwater modelling is a systematic process in which data is used to construct an initial conceptualisation, then convert the conceptual model to a numerical or analytical representation, the model is then checked to be behaving realistically by calibration and then simulations undertaken to predict dewatering effects.

- Simple analytical methods are useful in the initial stages for calculating order of magnitude effect and setting the approximate area of influence but are not spatially accurate for design purposes or abstraction permitting and licensing; Analytical approaches can individually simulate different processes such as box dewatering, drainage to canals etc but cannot provide analysis of these processes in combination or in aggregate with each other.
- Progression to numerical modelling requires more data, model construction, calibration and verification but enables a much more accurate 'distributed' model to be set up that simulates different parameter values for layers, zones, and boundary conditions such as canals and drainage points.
- Numerical modelling can then be used to identify drawdown and effect at specific receptor points.

The general approach is a tiered level of effort that matches risks and decisions that need to be made. Simple analytical equations (Tier 1) provide a basis for more sophisticated analytical models (Tier 2) to fully time-variant numerical groundwater modelling (Tier 3).

5.1 Outline of Tier 1 Analytical Methods

There are in the order of over 20 different analytical solutions for determining groundwater flow to a well, well-points, a pit or a trench, and each solution has variants that reflect the nature of the aquifer that is being assessed. Using published Tier 1 Analytical Tools (Ver. 1.6)[5], the following methods are outlined:

1. Flow to a pit, unconfined solution (Marinelli and Niccoli, 1998)
2. Trench with flow from two sides, unconfined, partial penetration

5.1.1 Flow to a Pit, (Marinelli and Niccoli, 1998)

The Marinelli and Niccoli Eqn represents both side-entry flow (zone 1) and basal flow (zone 2). The parameters inputted and analytical output are shown in Figure 16 below. The recharge value is estimated as 10% of average annual rainfall. The method indicates a dewatering flow rate of 14.13m^3/sec for an un-restricted basement excavation, which is equivalent to 1.22Mm^3/d. The simulation shows zero basal entry flow, all inflow comes from side wall (zone 1). A 3m drawdown is assumed to construct a chamber 2m below the water

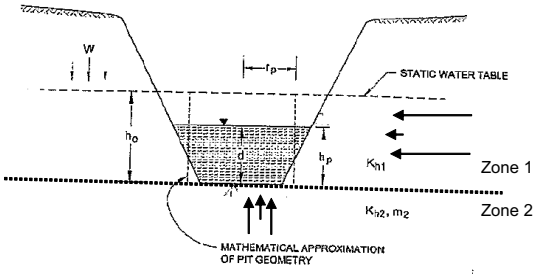
[5] UK Environment Agency Science Project Sc040020, Water Management Consultants Ltd,

table to enable 1m of dry conditions for placement of concrete flooring etc. No estimate of pressure, buoyancy or post-installation leakage is made.

ESTIMATION OF MINE INFLOWS (MARINELLI AND NICCOLI, 2000)

Client:
Project:

Parameters:
Kh1 = horizontal hydraulic conductivity zone 1
rp = effective pit radius
hp = saturated thickness at pit wall
ho = initial (pre-mining) saturated thickness above base of zone 1
d = depth of pit lake (usually equal to hp)
W = recharge rate
m2 = anisotrophy parameter (dimensionless)
Kh2 = horizontal hydraulic conductivity within zone 2
Kv2 = vertical hydraulic conductivity within zone 2
ro = radius of influence
User Input
Calculated



Calculation for zone 1 (flow to pit walls above base of pit)

Kh1 (m/d) =	86.4	**0.0010000**	**(m/s)**
rp (m) =	57.3	based on 13152m^2 basement	
ho (m) =	3		
hp (m) =	3		
W (mm/yr) =	30	**9.50663E-10**	**(m/s)**
d (m) =	3		
ro (m) =	**2175.669759**		
Q1 (kL/s) =	**0.014129193**		
Q1 (kL/d) =	**1220.762264**		
Q1 (L/s) =	**14.12919287**		

Calculation for zone 2 (flow to bottom of pit from base of zone 1)

Kh2 (m/d) =	0.01	**0.0000001**	**m/s**
Kv2 (m/d) =	0.001	**0.0000000**	**m/s**
m2 =	**3.1622777**		
Q2 (kL/s)=	**0**		
Q2 (kL/d) =	**0**		
Q2 (L/s) =	**0**		

Total Inflows (Q1 + Q2)

Qt (kL/s) =	**0.0141292**
Qt (kL/d) =	**1220.7623**
Qt (L/s) =	**14.129193**

Figure 16: Inflow estimation working 1 (Marinelli and Niccoli Eqn)

5.1.2 Trench Flow

The theoretical radius of influence of an assumed 0.1m drawdown in a trench such as 'Kollektor A-2' in gravel material with a permeability of 1m/s is 300m[6]. The 0.1m drawdown is assumed to be sufficient to create the necessary gradient fall to have effective diversion drainage. The amount of flow that will collect in a trench with length 1km is calculated to be 2,101.85m^3/day from this analogy.

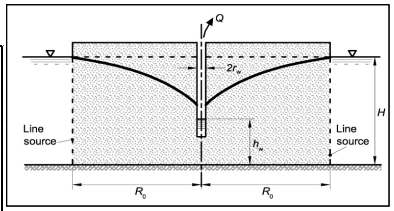
[6] Sichardt equation

10) Partial penetration by a single row of wellpoints of an unconfined aquifer midway between two equidistant and parallel line sources

$$Q = \left[\left(0.73 + 0.27\frac{(H - h_w)}{H}\right)\frac{Kx}{R_0}(H^2 - h_w^2)\right]$$

Essential input
Optional input
Calculated

		expected		min	max	
Head						
Height of water table at radius of influence	H	50	m			m
Height of water table at well	h_w	49.9	m			m
Conductivity						
Hydraulic conductivity of aquifer	K	86.4	m/d			m/d
Radius						
Length of trench	x	1000	m			m
Distance to line source, equal to radius of influence	R_0	300	m			m
Is R_0/H greater than or equal to 3 ?		Yes		No	#DIV/0!	
Total discharge from wellpoints	**Q**	2101.85	m^3/d	2101.85	2101.85	m^3/d



(Figure adapted from Mansur & Kaufman, 1962)

The following assumptions apply to this equation
- the slot is infinite in length
- R_0/H greater than or equal to 3
- the aquifer is unconfined
- the aquifer is homogeneous, isotropic and of uniform thickness
- the Dupuit Forcheimer assumption is valid
- the aquifer has reached steady state conditions
- the initial water table is horizontal

(Mansur & Kaufman, 1962)

Data sources (to complete an audit trail)		
Height of water table at radius of influence	H	assumed
Height of water table at well	h_w	assumed
Hydraulic conductivity of aquifer	K	median value from PSDs
Length of trench	x	1km from taklif rejasi.pdf
Radius of influence	R_0	calculated from Sichardt

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Figure 17: Inflow estimation working 2

Both analytical solutions show plausible, high inflow rates, exceeding 1000 and 2000 m^3/day respectively.

5.2 Outline of Tier 3 Numerical Method

Numerical methods require more effort in construction, calibration, parameter setting and model runs. However, the results are more scalable and informative and give appropriate definition for impact assessment and design. A numerical model would use site-specific topography and accurate survey data for river stage levels, drainage lines, pre-modelled recharge estimates, geological layering and accurate parameters, dimensions and configuration of the construction design. Finally, because dewatering is a time-dependent drainage process and the dewatering would be temporary, the numerical modelling would need to incorporate '*transient*' processes.

To assess the feasibility of numerical modelling, a simplified conceptual model was developed in Visual Modflow Flex Ver.8.0 using the following parameters and data sources:

- The site digital elevation model (projection UTM42N, WGS84)
- An assumed hydraulic conductivity value of 1 E^{-3} m/s applied across the whole model as a single layer.
- An assumed initial water table 3m below the ground level.
- Recharge at 30mm per annum and a General Head Boundary set in the southwest to drive the hydraulic gradient across the model domain.

Three model scenarios were run:

1. An initial baseline run that simulates the prevailing hydraulic conditions, flow direction, gradient and influence of Chirchik River which was represented approximately with ‘guesstimate’ river stage and location information.
2. A second run showing the drawdown effect from dewatering a basement with similar dimensions to the BHS basement. The basement dimensions are 13,152m^2 the model run used a single drainage cell of size 136m x 136m (18,496m^2) to reflect the extra working area needed to construct the basement. An approximate 3m drainage elevation below the water table was applied.
3. A third run, the same as No.2 above but with a 6m drainage level below the water table.

The results are shown in mass balance form and as graphical water table maps in the figures below. The modelling was established in a simplified, one layer, recti-linear, finite difference grid which included a general representation of the initial water table and, hydraulic gradient. Surrounding drainage lines could be represented as a drainage network but were omitted in this trial run.

Conceptual model set-up for numerical modelling is shown below.

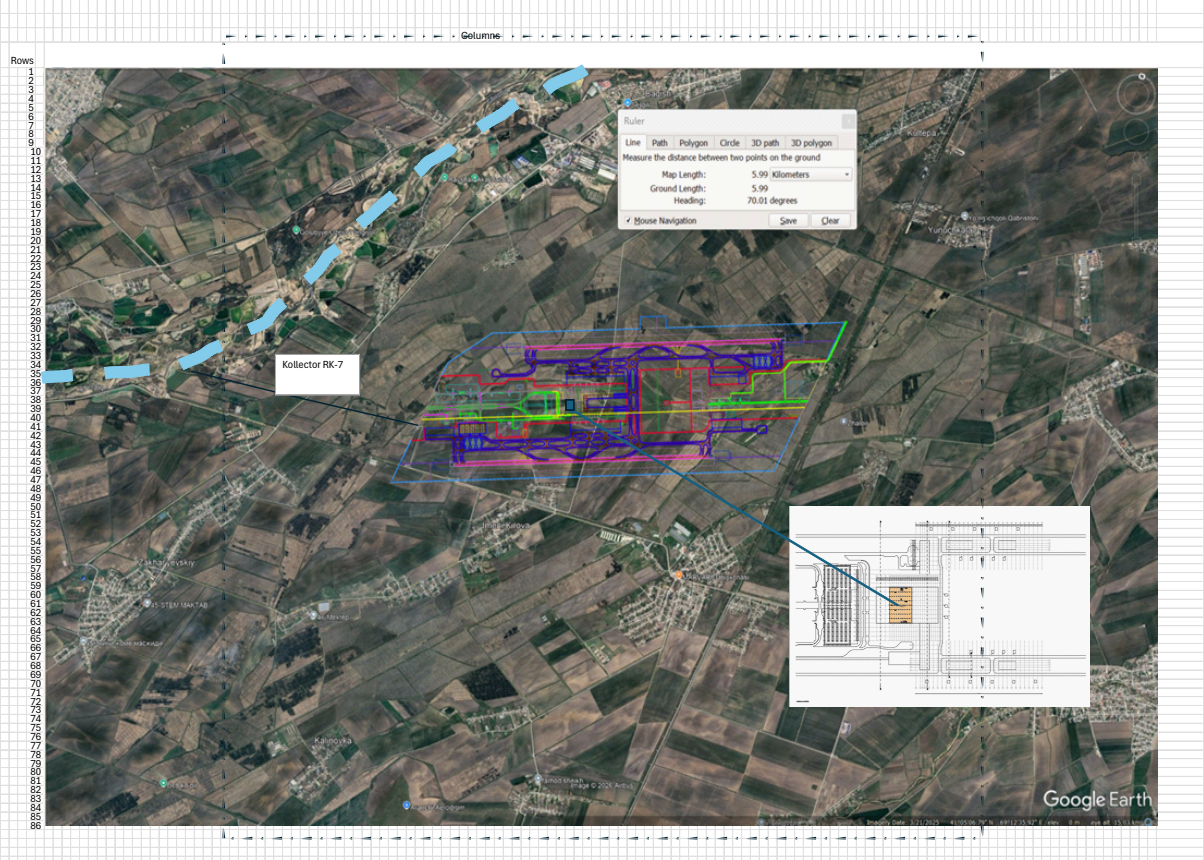
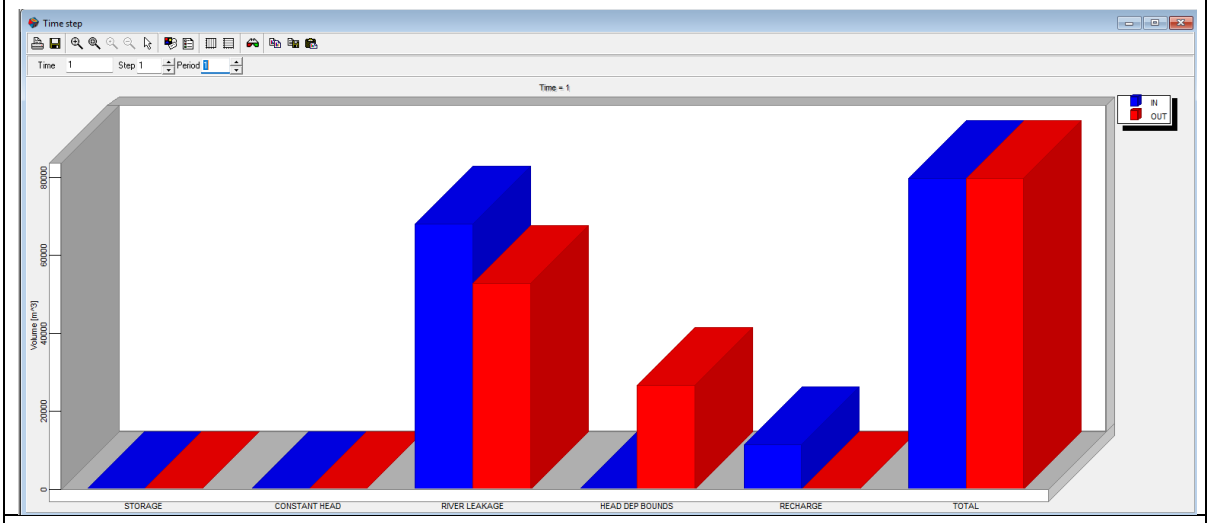
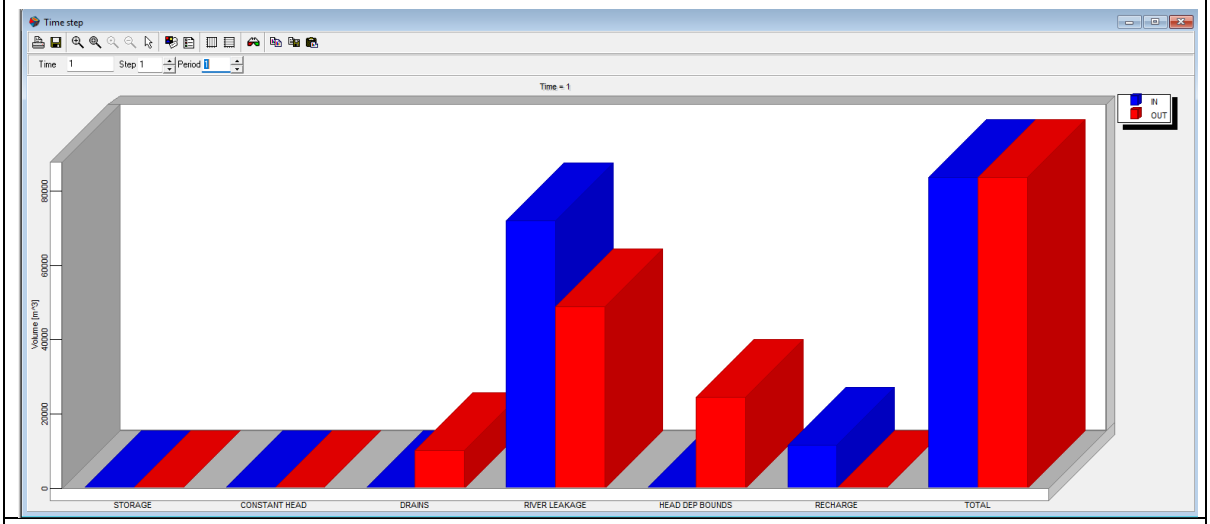


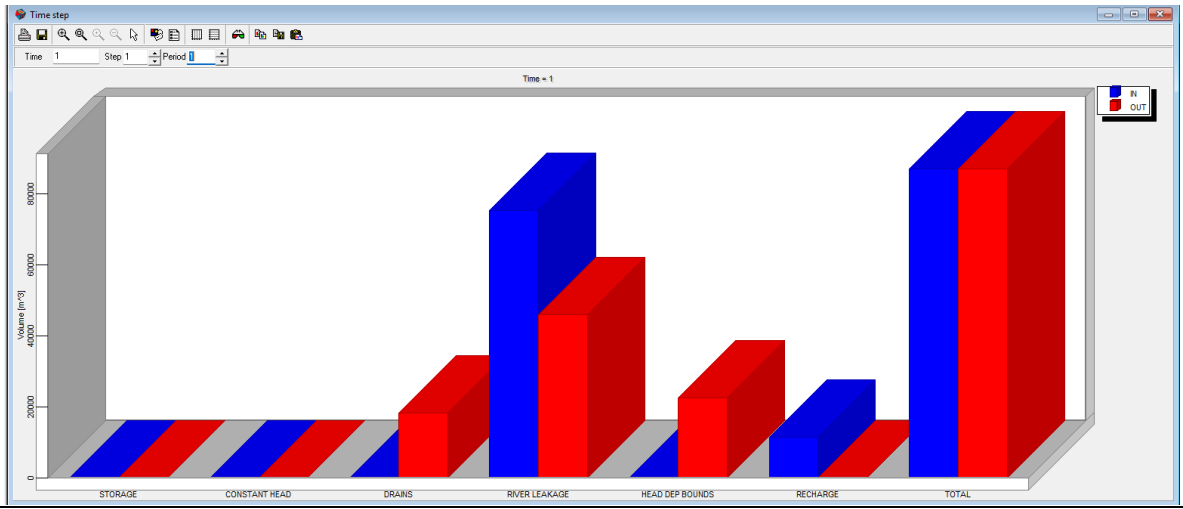
Figure 18: Mass Balance Results, Numerical Modelling



Run 1, Baseline

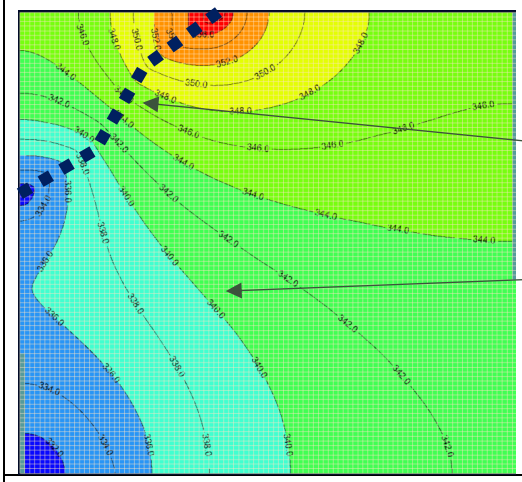


Run 2, drainage layer 3m below water table (dewatering basement)

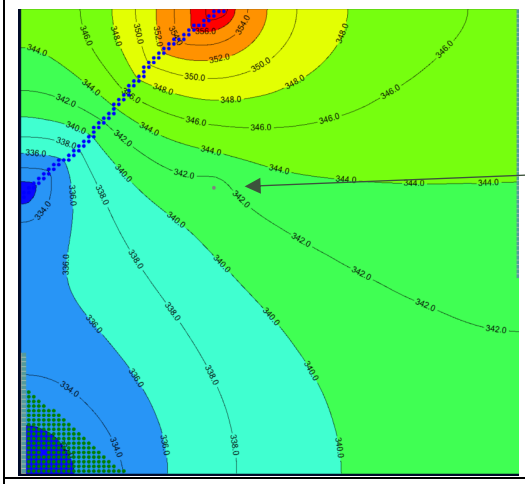


Run 3, drainage layer 6m below water table (dewatering basement)

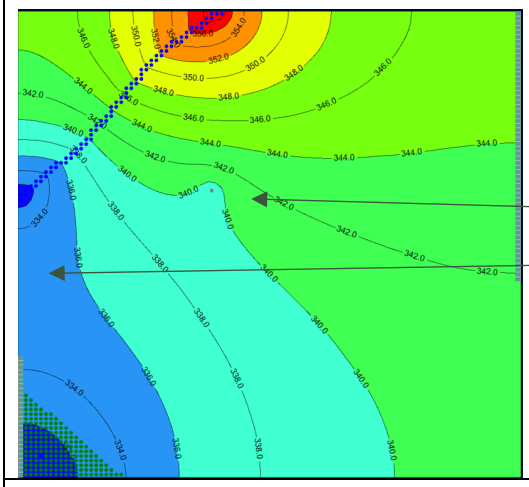
Figure 19: Mass Balance Results, Numerical Modelling



Run 1, Baseline



Run 2, drainage layer 3m below water table (dewatering basement)



Run 3, drainage layer 6m below water table (dewatering basement)

Figure 20: Example Water Table Results, Numerical Modelling

The trial numerical modelling mass balance results shows an acceptable match with the arithmetic analysis and the dewatering simulations show plausible results in terms of the likely order of magnitude effect on water table and potential flows in surface water systems.

The mass balance shows the drain outflow comes mainly at the expense of reduced river leakage (out) which is an important finding for the ESIA and should be examined in more detail. The trial run shows model runs will be effective in developing accurate impact analysis and design and mitigation information. However, this will require constructing a more accurate numerical model and testing process which is beyond the scope of this first conceptualisation.

6.0 Summary, Data Gaps and Recommendations

The site presents a high-risk hydrogeological setting for construction because of a high water table, high-permeability gravel deposits, extensive and deep gravel unit, hydraulic connection with nearby and regional surface water features and sustained recharge from irrigation infrastructure. Prior to construction of basements, chambers or shafts below the water table, the designers and impact assessors will need to thoroughly understand groundwater control measures.

Initial analysis suggests:

1. Diversion of the drainage network will not de-saturate the gravels in the working area and will have only limited, localised effect on dewatering.
2. Arithmetic analysis indicates dewatering to 3m depth below the water table will create inflows to basement excavations such as the BHS basement that exceed 1,200m^3/day. However, these estimates seem to underestimate the likely actual amount due to the arithmetic simplification.
3. Numerical (steady state) modelling suggests dewatering inflows to the BHS basement may exceed 10,000m^3/day. However, steady state modelling tends to over-estimate inflow as there are no time-varying changes in storage or boundary condition. The dewatering value will likely reduce if transient modelling is undertaken.
4. The radius of influence of the dewatering, by virtue of the highly permeable gravels, causes a relatively far-reaching effect on hydraulic gradient and interaction with surface water systems. More specific modelling and analysis will be required to determine an accurate estimate of drawdown extent, depth and effect on receptors.

The current main data gaps are:

1. Hydraulic conductivity (K) values, the PSD analysis is not reliable for determining values. A combination of constant head tests and pumping tests should be performed on monitoring wells with known screen and completion details. A pumping test with a nearby observation well should be performed to get accurate specific yield data.
1. Surveyed elevations for the drainage network and river stage levels.
2. More detail on the basement dewatering requirements and construction method, finished design, mitigations and duration of work.
3. Hydrogeological interpretation of water quality data and vulnerability.

Recommendations are:

- Obtain reliable K data and other groundwater data such as up-to-date, repeat groundwater level monitoring rounds for 2026.
- Construct and calibrate a numerical model to appraise inflow rates, drawdown effects, cumulative effects and impacts.
- Use the model to test different configurations to support a safe, manageable dewatering design.
- Run the model in transient mode.

- Model the drainage network, more accurate BHS facility design, mitigations such as sheet piling and basal liner / diaphragms and look at cumulative effects when other dewatering chambers are included such as the 50m^2 sub-element for the shaft requiring a deep excavation 6.7m below the water table.

Regards,

SLR Consulting Limited



Phil Burris
Technical Director

Mick Allen
Senior Hydrogeologist