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ΔΙΕΥΘΥΝΤΗ ΤΜΗΜΑΤΟΣ ΠΕΡΙΒΑΛΛΟΝΤΟΣ 8 Ιανουαρίου 2021
Φ/δι Μαρία Φεσά
Λευκωσία

ΘΕΜΑ: Αίτηση για τη Χορήγηση Πολεοδομικής Άδειας (Αρ. Φακ ΠΑ 420/17) για την Ανέγερση Μικτής Ανάπτυξης με Κέντρα Αναψυχής, Γραφεία και Οικιστικά Διαμερίσματα στην Ενορία της Αγίας Νάπας στη Λεμεσό με το Όνομα The Gallery

Σε απάντηση της επιστολής σας με ημερομηνία 11 Νοεμβρίου 2020 και αριθμό φακέλου 02.10.011.014.003.041 σας υποβάλλουμε επιπρόσθετα στοιχεία σχετικά με τη μελέτη μείωσης της στάθμης του υπόγειου νερού κατά τη διάρκεια της ανέγερσης της πιο πάνω αναφερόμενης ανάπτυξης. Συγκεκριμένα σας υποβάλλουμε τα ακόλουθα:

1. Dewatering Method Statement
2. Μεθοδολογία Αποστράγγισης - Εκτελεστική Περίληψη
3. Pumping Test Software Output

Παρακαλώ για τις δικές σας ενέργειες για την προώθηση της ΜΕΕΠ και εξέταση του έργου από την Επιτροπή Εκτίμησης Περιβαλλοντικών Επιπτώσεων και την έκδοση της σχετικής περιβαλλοντικής γνωμάτευσης.

Με εκτίμηση,

Πανίκος Νικολαίδης
Πολιτικός Μηχανικός/Μηχανικός Περιβάλλοντος

Κοιν. Δημοτικό Μηχανικό Δήμου Λεμεσού

ASKANIS GROUP OF COMPANIES
Gallery Project

Dewatering Method Statement

Ch. Kridiotis

December 2020

Contents

1. Hydrogeological Regime

1.1 Introduction

The proposed Development consists of a 13-floor tower on the sea front, and 5 floors on St Andrews Street, with two underground floors for parking. The total depth of excavation will be approximately 8m (including 1.5 thick foundation raft/pile cap throughout). In addition, the lift wells will be founded at an estimated depth of 10.5 m below existing ground level, i.e. 2.5 m below the general excavation level.

The building plot covers an area of 2291 m^2 and is generally flat, gently dipping towards the sea (south). At present, there are no existing buildings within the site.

The total area in plan (footprint) to be excavated down to foundation level is 1630 m^2, plus an additional area of 70m^2 which will form the lift wells' foundation.

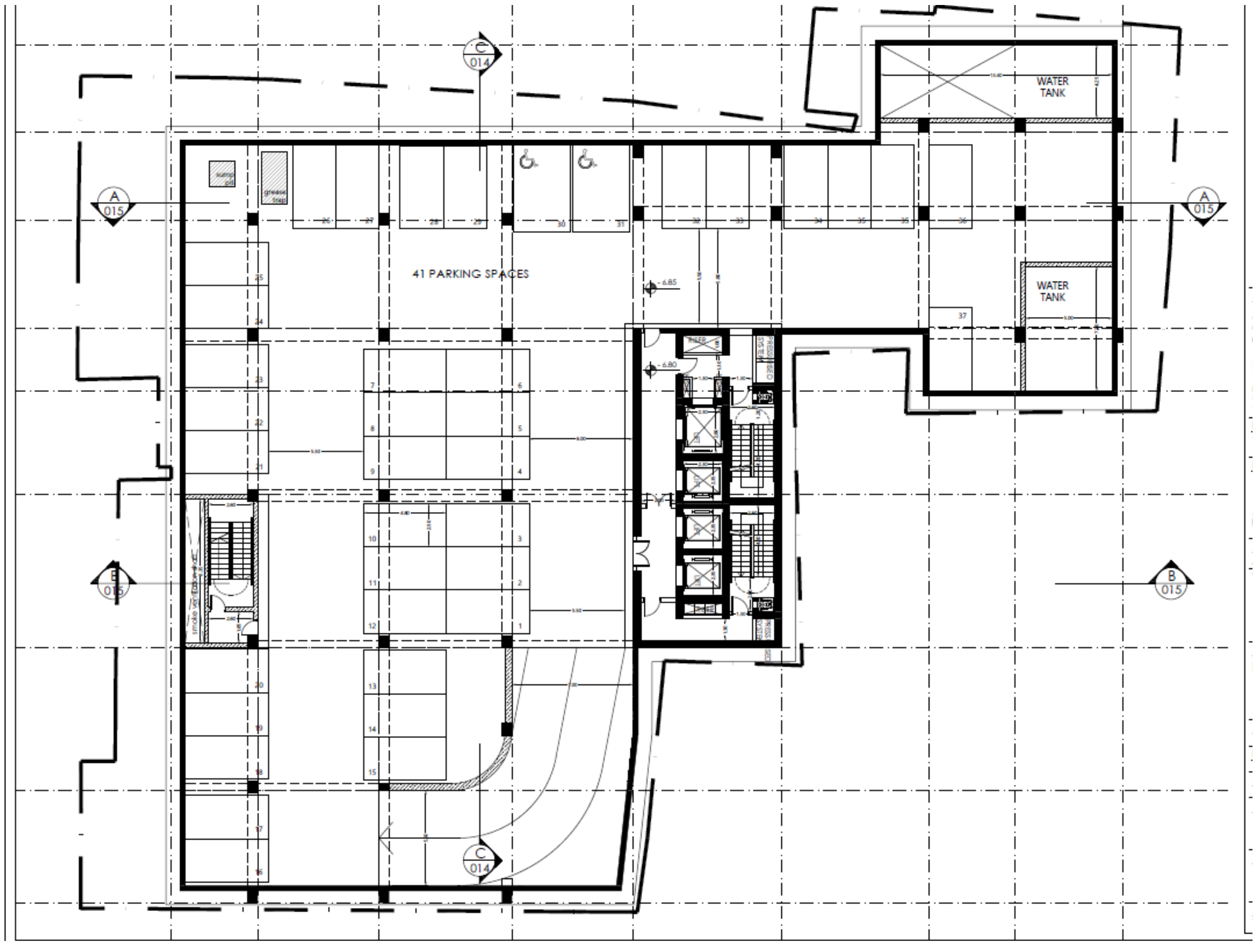
Prior to the excavation, a perimetric cut-off wall will be constructed which will form a positive impermeable cut-off, down to a less permeable geological horizon.

1.2 Hydrogeological Conditions

The hydrogeological conditions prevailing at the building site have been assessed based on the results of four (4) investigation boreholes drilled within the area of the building plot (Geoinvest, May 2017 and June 2019), plus an additional borehole drilled by others. Results so obtained, were also correlated to existing information obtained from desk study sources, i.e. available geological/geotechnical information from previous geotechnical and hydrological site investigation work carried out in the general area of the proposed Project, in particular the Microzonic study by the GSD.

The investigation boreholes were taken down to a maximum depth of 58m, with associated disturbed /undisturbed sampling and in situ testing.

In order to assess the mass permeability of the foundation strata, a full-scale Pumping test was carried out within the site during November 2019 and February 2020. Measurements recorded during the pumping test were analyzed with the use of a specialized computer programme and correlated to laboratory and in situ permeability tests.



Site Plan

Pre-construction dewatering assessment for the aquifers encountered, was also correlated to results obtained during full scale pumping tests performed for other sites in the general area, having similar geological conditions. Additionally, information from other construction sites in progress or previously executed in the broader Limassol coastal area, has been analysed for the purposes of this project.

The ground model considered provides information for:

- Determine requirements for ground water control and any practical or Environmental constraints.
- Develop preliminary conceptual model of ground water flow.
- Estimate total flow rate required to achieve drawdown at site down to excavation depth.

In accordance with information available from other sites in the area, and from the findings of the site investigation campaign and **pumping test performed within the site**, there are two main aquiferous horizons, both related to the coarse materials. The first, a phreatic aquiferous horizon, is related to Horizon B, found close to the surface and extending down to 13.5 meters. The second is related to the coarse-grained materials of Group C2, found at various depths as indicated in Table 1, below. Based on the general geology and morphology, it is estimated that the aquiferous layers of Group C2 are, partly, under pressure.

It should be noted that small amounts of ground water is present also within the sand rich sections of Group C1 (Fine Coastal Accumulations).

The static water level was recorded during the field work and during the pumping test and, no worth noting fluctuations were recorded. It was found to fluctuate between 1.60 and 2.20 meters below existing ground level.

It should be noted that a substantial amount of ground water is present also within the sand rich sections and gravelly lenses (Fine coastal accumulations), that have been encountered elsewhere. This fact, together with a large number of boreholes drilled all over the area, resulted to the interconnection and partly unification of these water bearing horizons. So, at the end, the whole area could behave as one unified aquifer, interrupted in places by impermeable soils.

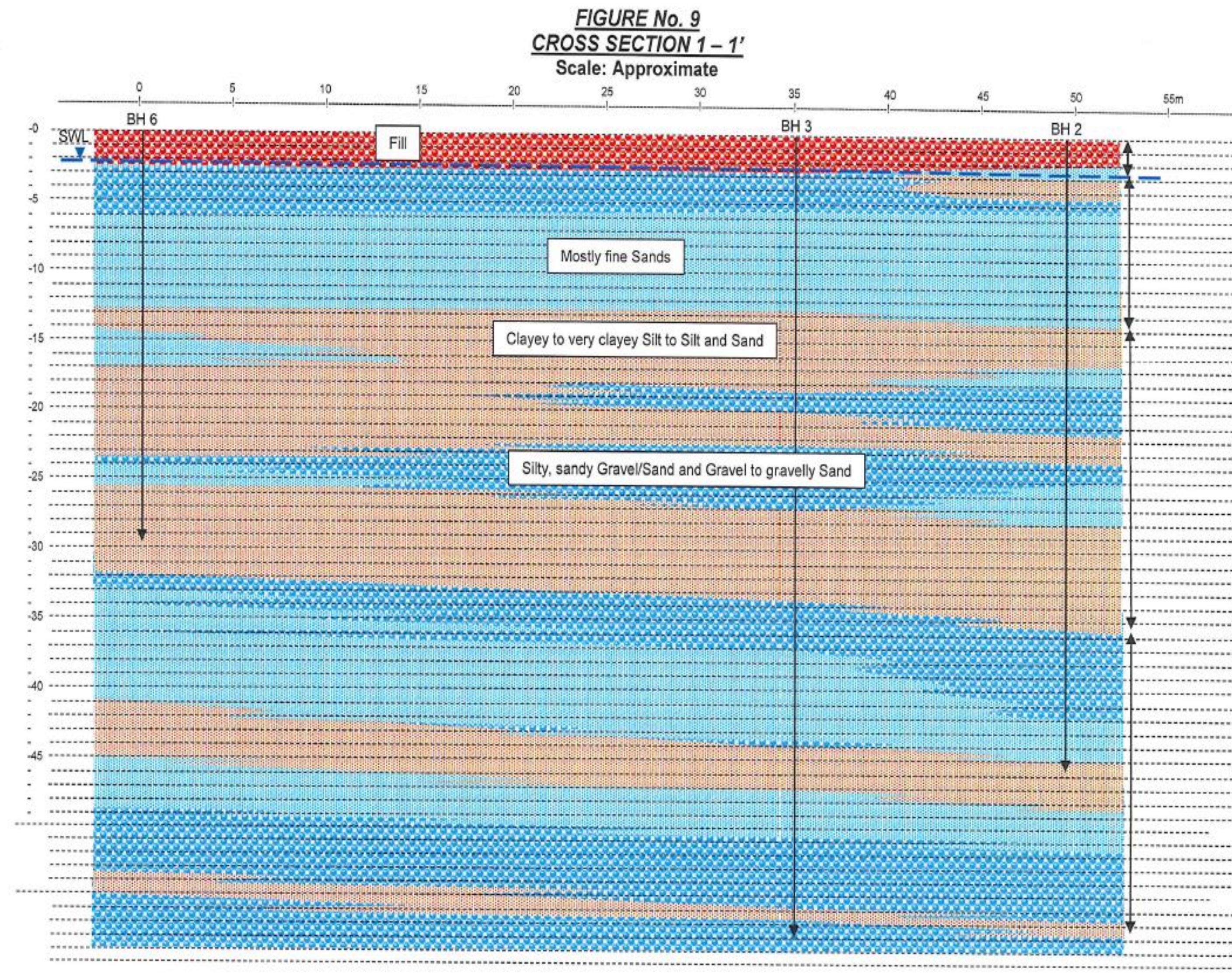


Fig.1 Geological Cross Section *(Geoinvest, 2019)*

1.3 Permeabilities - Hydrogeological Conditions

No in situ permeability tests were performed during drilling, since the conditions were not favorable for such in situ testing, due to shallow ground water table. Four laboratory permeability tests in accordance with BS5930 were performed on selected samples with the following results:

- Group B - Silty Sand: $8.8x10^{-4}$ cm/s or 0.72 m/day
- Group C1 – Sandy, very Clayey Silt: $4.7x10^{-5}$ cm/s or 0.04 m/day

- Group C1 – Clayey, very Sandy Silt: $1.8x10^{-4}$ cm/s or 0.15 m/day
- The permeability of the coarse material (Gravel, Cobbles) is estimated to be at least 15 m/day, depending on the content of the fine material.

Based on the above results, it is quite clear that the permeability of Groups B and C2 are high, whereas that of Group C1 is low. In the sand rich sections of Group C1 is estimated to be moderate.

Group	Soil Description	Depth Range(m)
A	**Fill**, consisting of clayey sandy silt soil, with igneous and sedimentary rock fragments.	0 - 2
B	**Beach Deposits** found below the fill and down to 13.5m. Consisting of: (i) Medium dense sand with varying amounts of silt, clay and gravel, (ii) A mixture of all fractions at various proportions, between 3 and 4.5 m. (iii) Intercalations of gravelly sand with sandy gravel, turning in places to sand and gravel.	2 – 13.5
C	**Coastal Accumulations (old alluvial deposits)** Consist mainly of: (i) Sandy Clayey Silt to (ii) Sandy Clay and Silt (iii) Clayey Sandy Silt to (iv) Clayey Sand and Silt (v) Silty Sand (vi) Sandy Gravel with some Cobbles (vii) Sand and Gravel Based on origin and particle size distribution soil group C may be sub-divided into groups C1 &C2 as follows:	13.5-45
C1	**Fine Grained Coastal Accumulations** Represented by sedimentary fine-grained accumulations of soil types (i) – (iv) of Group C. **Alternating with the coarse-grained materials, exhibiting a poor to moderately developed stratification.**	13.50-18 21.50-23.50 28.00-35.50 Below 45 meters
C2	**Coarse Grained Coastal Accumulations** Represented by mostly igneous (dark) coastal accumulations of soil types (v) – (vii) of Group C	18.00-21.50 23.50-28.00 35.50-45.00

Table 1. Soil description with Depth – Borehole 2

From available information derived from the site investigation work performed within the plot (Geoinvest) or in the general area of the building site (Limassol Microzonic study, GSD), the following hydrogeological data may be deduced for the different soil groups:

- Shallow water table – less than 2 m below ground level.
- Relatively low density and high compressibility and high liquefaction potential of horizon B between 9 and 13.5 meters.
- The large in-homogeneity of all beach and coastal deposits (abrupt facies changes in both horizontal and vertical sense).
- The high permeability and big amounts of ground water in soil groups B and C2, the latter being of considerable thickness and, probably, under pressure (artesian conditions?)
- The fine-grained coastal accumulations (Group C1) are alternating with the coarse-grained materials, starting at 13.50 meters and extending to more than 45 meters.

2. Inflow of water into the excavations during construction

For the control of ground water seepages into the excavation, a permanent cut off wall (secant pile, diaphragm, other) will be constructed to a depth of at least 5 m below the final excavation level. The cut-off will provide an impermeable barrier that will facilitate the lowering of the water table during construction, since lateral movement of ground water towards the excavation will be prevented and seepage paths towards the base of the excavations will be substantially increased.

In order to minimize the ground water inflow from the base of the excavation, the cut-off wall may be extended down to 13-14 meters or even deeper into geological group C1 (fine grained coastal accumulations – Geoinvest 2019), where the silt and clay dominates. In such a case it is expected that inflow into the excavation will be more manageable.

From available information (sources mentioned above), obtained during the site investigations and pumping test performed within the site, it is expected that strata at the anticipated excavation level will be of moderate to high permeability. Preliminary parameters considered for permeability, storativity and transmissivity for draw down calculations were taken from the pumping test results and, also, from tests performed in similar geological horizons in the general area.

It should be mentioned that the large number of boreholes drilled over the whole area, resulted to the interconnection and unification of water bearing horizons. So, at the end, the whole area could behave as one unified aquifer, interrupted in places by impermeable soils. In this context and for calculation purposes, it may be assumed that water pressures within soil horizons outside the wall are close to hydrostatic (Fig.2).

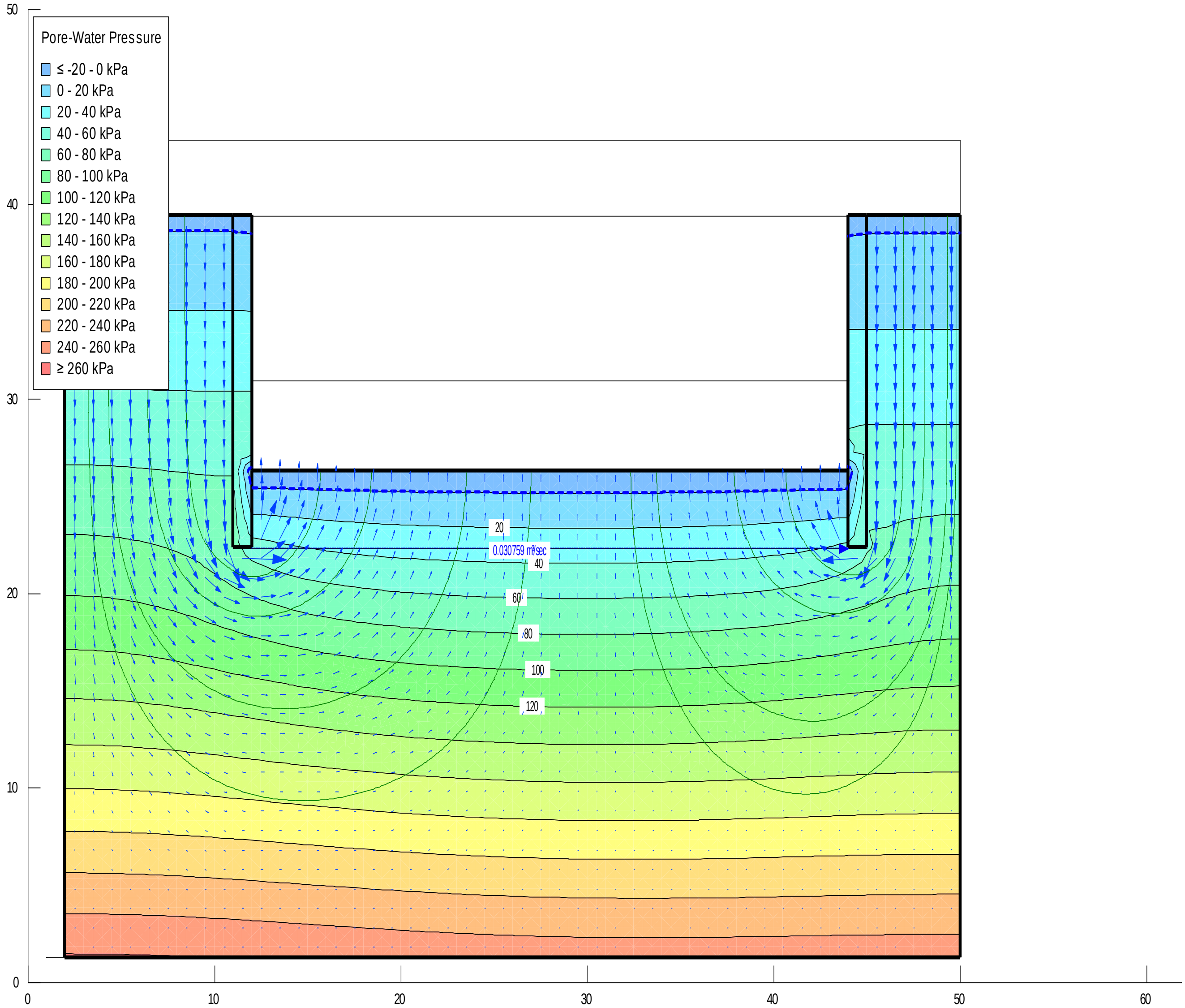


Fig.2 Schematic Hydraulic Model

As mentioned above of great importance of course, are the actual permeabilities of the stratum into which the diaphragm wall will be keyed, below the base of the excavation.

Considering the anticipated stratigraphy of the site (based on the information available), the depth of the foundation and the perimetric diaphragm wall and the pumping test results obtained, an estimate of the anticipated inflow of water into the excavation during construction has been made. It has been considered that water inflow will come partly from around the wall and partly directly from the stratum below the base of the excavation.

The amount of inflow into the excavation during construction was estimated based on a flow net analysis, considering the inflow of water between the vertical diaphragm walls, which constitute flow lines as they represent impermeable elements.

The flow, q per unit length of wall is given by:

$q=k\ h\ Nf/Nd$ where K is the coefficient of permeability

h is the total head difference

Nf is the number of flow channels

Provided that a watertight perimetric wall will be taken down to a depth of about 14m (into the Clayey to very Clayey Silt to Silt and Sand deposits), it is preliminarily estimated that the inflow into the excavation will be in the region of 1.1 to 1.3 m^3 per square meter of plan area, per day. This translates into a total amount of inflow of around 1,900 cubic meters per day, for the whole excavated area, or 950 cubic meters per day if the excavation is carried out in two stages, i.e. divide the site into two approximately equal areas (compartments), by constructing a **temporary** intermediate secant pile wall. In such a case, the temporary wall will be demolished as excavation progresses for the next compartment.

The above figures are an estimate based on all available information to date and, they will depend on the extent of the inflow from the underlying strata.

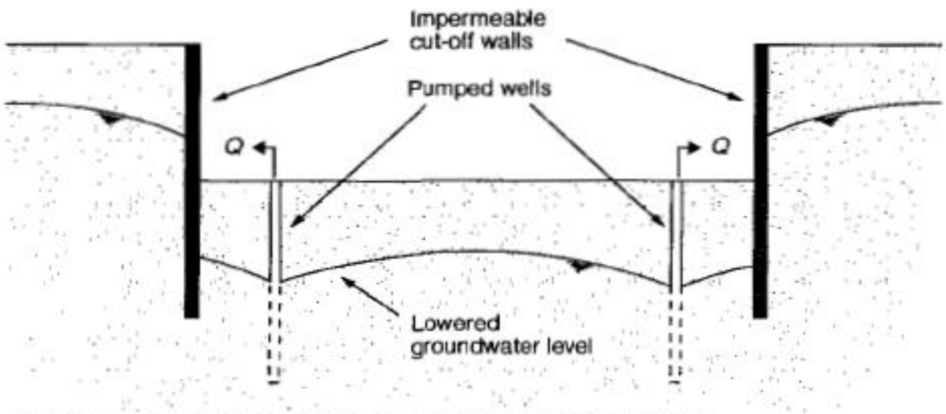
2.1 Ground Water Control

There are several techniques or methods available for controlling ground water flow into excavations at a construction project. The selection of a technique or techniques appropriate to a particular project will depend on many factors. However, the lithology and permeability of the soils will always be of paramount importance.

Considering the total footprint of the basement area, it is feasible to proceed with the excavation of the whole basement area in one stage. This will necessitate total pumping in the region of approximately 100 m^3 /hour.

Ground water control recommendations are mainly based on the publication by CIRIA 515 (Ground Water Control, Design and Practice). The proposed ground water control method has been selected, partly, from figure 1.10 of this publication.

Considering the necessity to lower the water table at least 1m below the excavation level and that most of the inflow will be from the strata below, the water table has to be lowered for the whole working area. It is necessary, therefore, to design for a pump-well system that will work in combination with the physical cut-off wall and the system of Recharge Wells which will be constructed for the disposal of dewatering water.



b) Excavation with a retaining wall and wells to prevent water ingress through the base

Fig.3 Pump-Well System

Provided that the excavation will be carried out in one stage, 12 no. pumped wells will be required (including stand by wells). It is recommended that deep pumped wells are supplemented with shallow sump wells (1 – 2 m depth), if necessary. Adequately designed and constructed wells will have to be installed at a predetermined grid along the perimeter of the excavation, approximately 2,5 to 3m parallel distance from the secant pile wall Additional pumped wells may be necessary in the excavation area and the Lift well (see diagram 1).

Special filters must be provided in order to protect against the migration of fines.

In order to provide dry working conditions, the water level should be lowered down to a minimum of 0.5 to 1.0 meter below the base of the final level of the excavation.

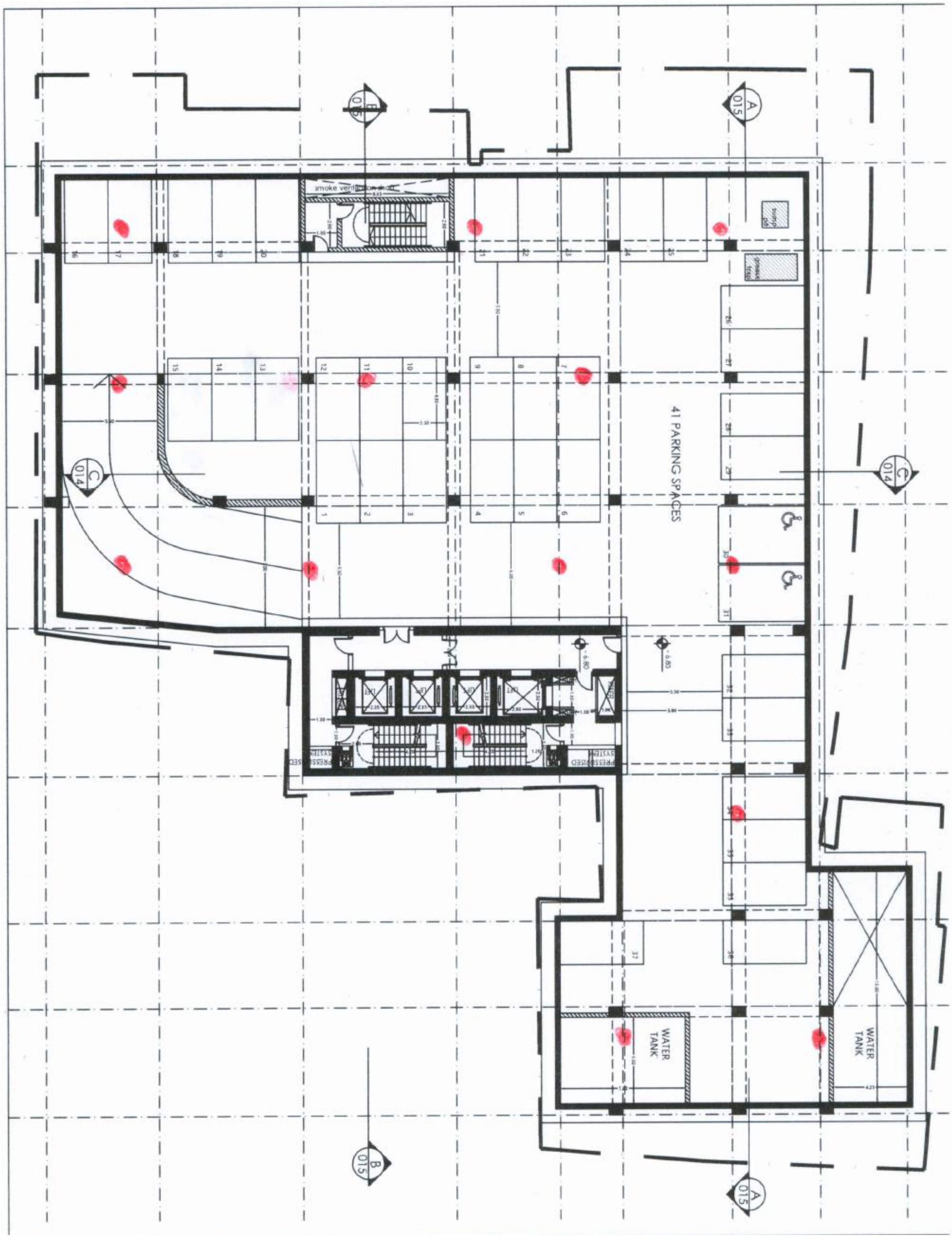


Diagram 1. Proposed arrangement of drainage holes in basement (shown in red)

Lifts' Well

For the excavation of the Lifts' well, which extends 2.5m below the general excavation level, a separate perimetric watertight cut-off may be constructed with the use of either sheet steel piles or secant piles. This will facilitate the excavation and dewatering to the required depth.

2.2 Mitigation of Environmental Effects

2.2.1 Dewatering Consenting

Environmental permits are required for extracting groundwater for dewatering purposes and for the disposal of the discharge into water bodies (sea, rivers), the storm water system or back to the aquifer.

All sites that implement dewatering must obtain the necessary permits from the competent Authorities. The Engineer and Contractor must fully understand the conditions and lead-in times for activities covered by the permits.

Compliance with permit conditions can be demonstrated though the ITP for the project and associated quality assurance records.

2.2.2 Discharge to the Environment

It is usual practice to discharge dewatering water into the environment. Specific factors that need to be addressed are the siting of the discharge, the effects on the discharge location and the ability of the discharge environment to accept the volume of discharge.

It is important to be aware, that not only is groundwater being extracted to lower the water table, but it is also likely that there will be sediment mobilized by groundwater flow and also possible that contaminants residing in the groundwater will be drawn into the system.

Therefore, it is of paramount importance to implement appropriate monitoring and treatment. Potential methods include:

- Implementation of mitigation measures, such as devices to treat the discharge, to reduce or avoid adverse discharge of suspended solids or contaminants.
- Appropriate design of the dewatering system to minimize the loss of fines from in-situ soils and avoid ground settlement.

- Proper containment of any wastewater system being worked on and removal of septic water prior to works as appropriate.

For high rise buildings under construction near the coast in the Limassol area, the common practice previously adopted for the disposal of ground water was to pump it (after treatment for the removal of fines and/or other contaminants) into the sea, at a predefined distance from the beach and, through specially designed diffusers. The point of discharge must satisfy certain criteria imposed by the competent authorities.

In the case of the Askanis Project and based on the experience acquired from the performance of full-scale infiltration tests at geologically similar sites, a recharge system is considered, where ground water abstracted by the dewatering system is returned back to the aquifer. Such a scheme requires caution and careful planning.

2.2.3 Dewatering Discharge Quality – Suspended Solids

Dewatering permits require that dewatering water pass through a sediment removal device such as a sediment tank, prior to discharge, with total suspended solids (TSS) in the discharge leaving the site not exceeding 30 g/m3. Deviations from the TSS limits are generally noted through a visual check of the water being released into the environment. Standard samples can be used for comparison to allow a rough instant field assessment of discharge quality. If required a sample is taken and tested in a laboratory (24 hr. to 48 hr. turn-around).

Because primary sedimentation tanks remove the solids that settle quickly, it is only particles with a long settling time that will be discharged from the primary treatment. Therefore, samples of discharge water that meet the approved limits should be prepared in a laboratory based on the typical particle size expected to be discharged from the primary tanks. These can be compared with samples taken on site to allow approximation of the TSS value of the discharge.

This would allow any compliance breach to be addressed early. The visual testing is low cost and able to be actioned and recorded quickly.

Adequate removing of fines from pumped water is **critical** for the efficient operation of recharge wells and avoidance of filter clogging. Therefore, if considered necessary, further treatment of pumped water (e.g. use of sand filters) should be carried out.

2.2.4 Settling Tank Design

Settling- Definition

A unit operation in which solids are drawn toward a source of attraction. The particular type of settling considered here, is gravitational settling. It should be noted that settling is different from sedimentation.

Advantages of Settling Tanks

- Simplest technologies
- Little energy input
- Relatively inexpensive to install and operate
- No specialized operational skills
- Easily incorporated into new or existing facilities

Factors affecting settling velocities (Vo)

- particle specific gravity
- particle size distribution

Design data required to ascertain mechanical construction are, specific gravity of solids, size distribution of solids, underflow construction, operating temperature, and geographical location.

Basic design principles

- Chamfered weir to enhance laminar flow(85% of water depth)
- full-width weir
- Determine effective settling zone and sludge zone
- Basin floor area of 41 liters per minute (Lpm) per m2 of flow.
- 250 to 410 Lpm per m width of weir for outflow.
- Submerge inlet weir 15% of basin water depth.
- Use 25 cm wide weirs and use rounded edges.
- Maximize length of settling chamber as much as possible.
- In plan, the length may vary from two to four times the width.
- The length may also vary from ten to 20 times the depth. The depth of the basin may vary from 2 to 6 m. The influent is introduced at one end and allowed to flow through the length of the clarifier toward the other end.

Rectangular Settling Tank Design

Provided that a watertight perimetric wall will be taken down to a depth of about 14m, it is preliminarily estimated that the inflow into the excavation will be in the region of 1.1 m^3 per square meter of plan area per day. This translates into a total of around 1,700 cubic meters per day, for the whole excavated area.

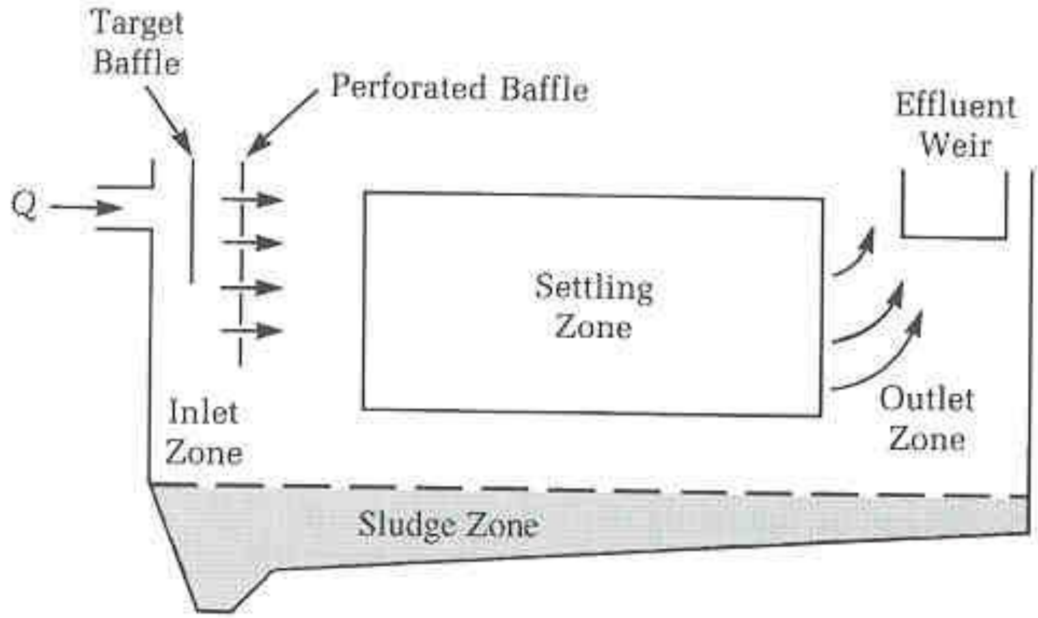


Fig.4-Basin Model

Considering a maximum volume of 2000 m^3 of ground water that has to be pumped per 24 hours, then a flow rate of 0.024 m3/sec has to be treated. Assume a settling tank having an effective settling volume that is 10 m long, 3 m tall and 3 m wide. Assume particles (Silt to fine Sand) that have a settling velocity of 0.0008 m/sec, then,

V_0 = Q/A = 0.024 m3/sec / (10 m x 3 m) = 0.0008 m/sec (settling velocity)

Therefore, since V_0 is equal or greater than the settling velocity of the particles of interest, they will be completely removed.

Similarly, settling time required for particles for 2.8 m effective depth is,

Retention time = 2.8 /0.002 = 1400 seconds = 0.4 hours

3. Ground Water Recharge System

In the case of the Askanis Project the disposal of the pumped water to the sea may impose, apart from anticipated environmental restrictions and, other logistical difficulties that will have to be resolved in order to implement it. Therefore, considering the hydrogeological conditions prevailing at the site, the findings of the site investigation work and in particular the results of the Pumping Test, it is considered prudent that the disposal of ground water by means of a ground water recharge system should be considered. It is noted that this method has been used successfully at other sites in the Limassol coastal area (e.g. NEO project), having similar geotechnical conditions.

In this connection, reference is made to two full scale in situ infiltration tests, recently performed by us for,

i. The Renaissance Project of Prime Development, and
ii. Dasoudi Residence Project (Limassol seafront)

In the case of the Renaissance Project two no. recharge test Wells were tested in order to assess the in-situ recharge potential of the ground by injecting water at a specific level in the sequence of the stratification, where higher permeabilities are encountered.

From the analysis of the results carried out it was deduced that a maximum absorption rate of 20 m^3/hour was achieved in both boreholes, with zero bar pressure applied.

From previous experience and, also considering results obtained from similar tests performed in the general area in similar geological horizons (e.g., NEO project), it is expected that in the case that the recharge wells are pressurized up to one bar, higher injection rates may be achieved.

For design purposes it is considered that for an estimated dewatering requirement of 100 m^3/hour (case of total excavation), approximately 8, 300mm internal diameter, recharge wells will be necessary. Allowing for possible clogging of some of the boreholes, and therefore requiring rehabilitation, another 2 stand-by boreholes should be made available (Diagram 2).

Notwithstanding the above, it is recommended that, at construction stage, an infiltration test is performed first at the specific site in one of the recharge wells, in order to more accurately determine the number of recharge wells required.

This alternative offers the following advantages:

1. An important environmental benefit of the recharge method is that, by returning the pumped water to the aquifer, the possibility of increasing the salinity of the coastal aquifer due to excessive pumping to the sea (sea water intrusion), is avoided.
2. Prevents excessive aquifer depletion and, therefore does not put other ground water sources at risk.
3. Sensitive adjacent structures are not affected.

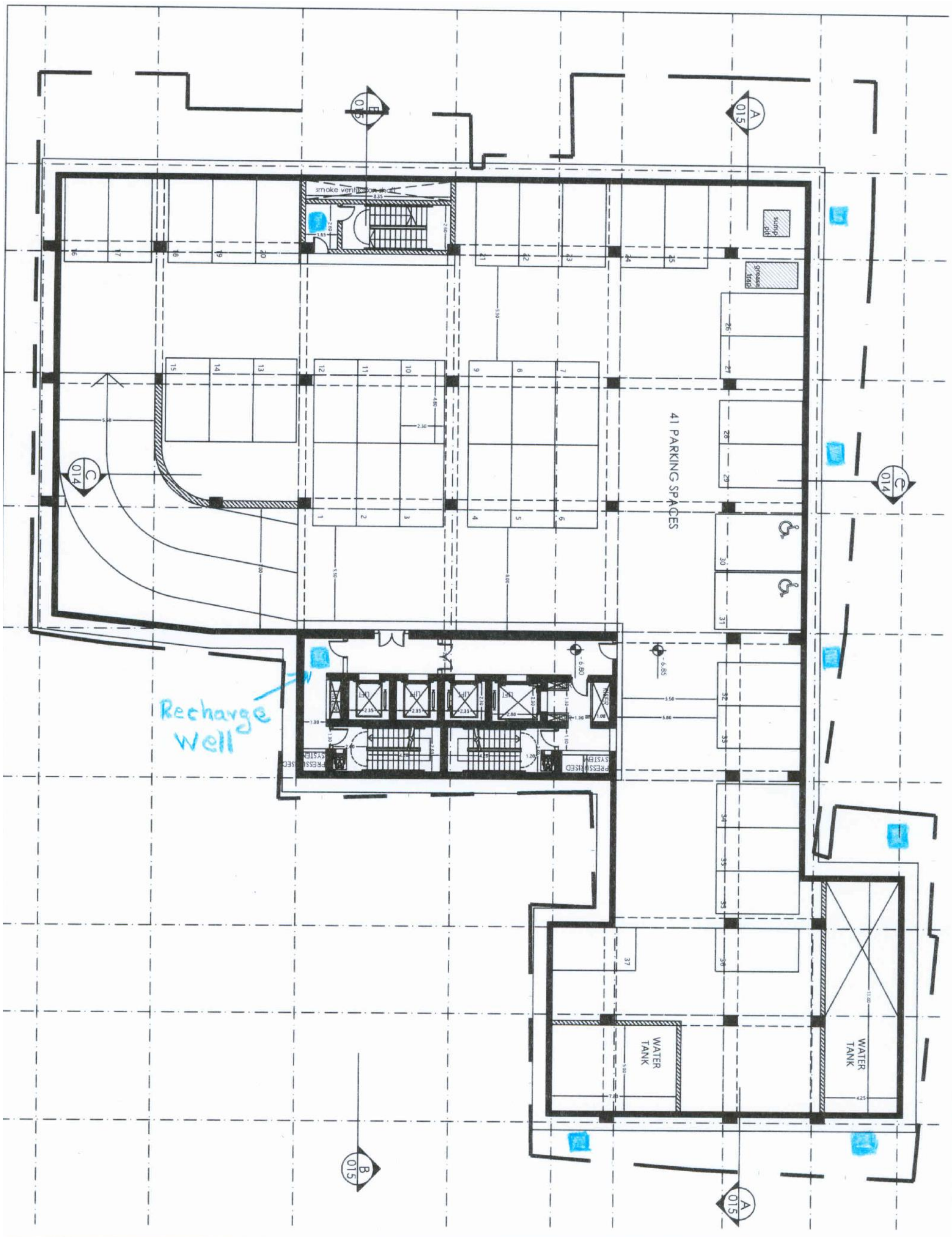


Diagram 2. Recharge wells (shown in blue)

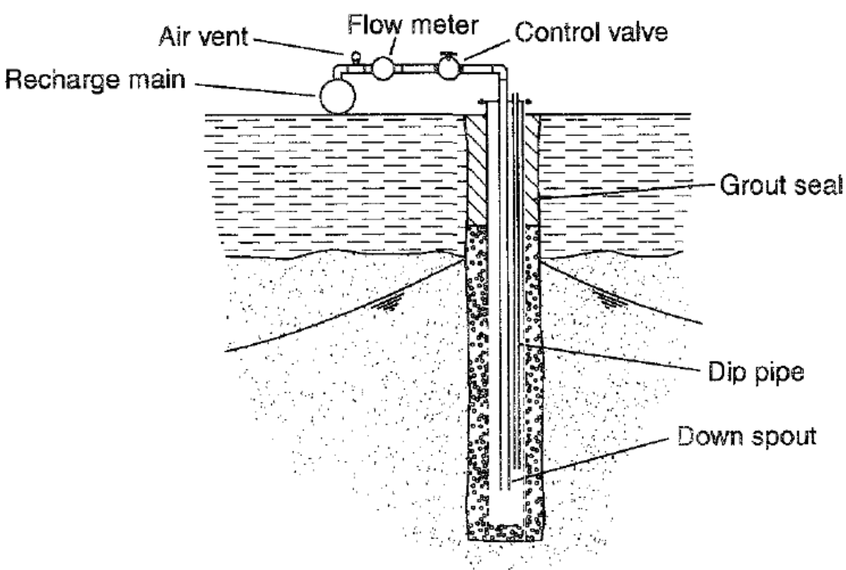
Unlike recharge trenches, recharge wells can be designed to inject water at a specific level in the sequence of the stratification, where higher Permeabilities are encountered. For the specific site under consideration, the wells can be taken down to depths greater than 20m into geological horizon C2 (Coarse Grained Coastal Accumulations), into zones where the predominant fraction is Gravel, with estimated maximum Permeabilities of up to 15 m/day.

The hydraulic requirements of recharge wells are essentially the same as those of extraction wells. As a result, recharge wells are designed drilled and developed the same way as extraction wells. On the other hand, recharge wells are prone to clogging, and therefore recharge water should be as clean as possible, with the use of suitable settling tanks plus other means, as previously discussed.

3.1 Installation of Recharge Wells

The wells will be drilled down to an exact depth which will be established based on geological conditions encountered at the specific site and have an external diameter of Φ900mm.

The boreholes will be installed with UPVC lining or other durable material agreed with the Supervising Engineer. The lining will have an internal diameter of Φ315mm (PN12.5). It must be capable of inserting in the borehole without risk of breakage, or damage to the joints.



Recharge Well

Perforated, slotted, or screened pipe shall only be used in that section where injection is intended. The hole aperture in such a pipe shall be smaller than any aggregate placed between the liner and the borehole wall.

After installation of the lining tube, rounded 5mm to 10mm pea gravel shall be placed in the annulus between the borehole wall and the lining tube to a level 1m above the perforated pipe as the casing is withdrawn. Care must be taken to ensure that the level of the aggregate is maintained just above the bottom of the casing to prevent collapse of the borehole. In addition, excessive heights of aggregate above the bottom of the casing could jam the casing during withdrawal and risk lifting the liner to the detriment of the excavation.

A concrete seal shall be placed on top of the aggregate to extend nominally 1.0 meters above and below the interface with the overlying unsuitable material (Fill). Care should be taken to prevent contamination of the gravel.

APPENDIX I (i)

Pumping Test

Askanis Gallery Project – Pumping Test

Purpose

The Pumping test was performed in order to establish Transmissivity, Specific Capacity and mass Permeability in accordance with **BSI BS 14686:2003** and **BS EN ISO 22282 Part 4:2012.** Methods and procedures followed were, additionally, in accordance with **CIRIA C-515**, Ground Water Control, Design and Practice.

The pumping test provides necessary parameters and data required for the design of the hydraulic model at pre-construction stage, which amongst other, will provide information related to:

- Estimation of the radius of influence due to dewatering
- Estimation of the flow rate required to achieve the drawdown
- Estimation of any potential settlement due to dewatering

Pumping Test Procedure

The Subcontractor (Geotechniki) provided a test pump and all the accessory equipment, including power source and a submersible pump to execute the aquifer test. The equipment was reliable for all the period of the pumping test operations, at the design rate.

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Prior to the pumping tests being carried out in accordance with BSI BS 14686:2003, monitoring data (water level fluctuations) over a duration twice the length of the pumping test was undertaken.

The discharge of the test pump was measured by a discharge meter. A control valve was installed so that the discharge rate did not vary more than five percent from the average rate.

Ground water samples were taken during the pumping test and a suitable “tap” was installed to enable the sampling.

The mode of operation of the test, the depth level of the pump and test yields were as instructed by the Supervising Engineer.

During the test pumping the water discharge was piped to a point of surface drainage sufficiently far from the pumping well.

A single step pumping test was executed and, the drawdown and recovery of the aquifer was measured. For both, the drawdown and the recovery, water-level measurements and the pumping yield were recorded.

The measure of the water level was done with the use of suitable dip-meters, in which case the physical presence of personnel was required at all measurements.

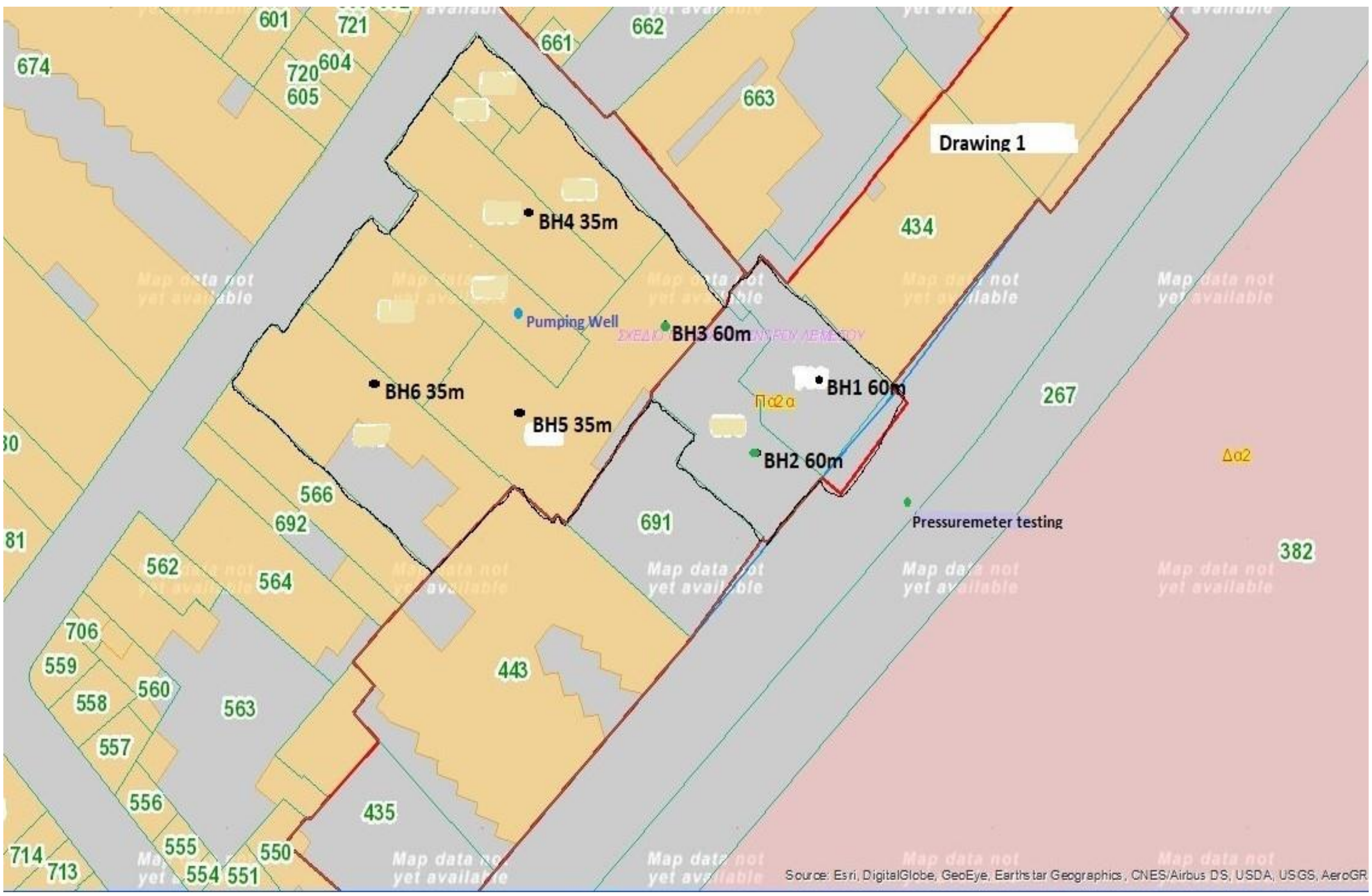
Measurements were taken in the pumping well and two standpipe installations. During the test, the pumping yield remained constant until completion of the test (equilibrium condition).

The recovery measurements were recorded until the dynamic water-level was equal to the static water level of the aquifer.

Pumping Test Results

The pumping test (repeated test) started on 01/02/2020 at 07:00 and lasted until the 13:00 of the same day, with a mean rate of pumping of $36m^3/h$ and with the drawdown being stabilized after 4 hours. Following this, the recovery of the water level was monitored for 5 hours by which time, a complete recovery of the aquifer was observed. Monitoring of both the drawdown and the recovery was carried out in two observation boreholes at 13m (Observation borehole 1) to the north, and 13m (Observation borehole 2) to the south, of the pumping well.

The position of the pumping well (14.5 m depth, 300 mm internal diameter perforated upvc pipe) is indicated in the site plan (drawing 1), below.



Drawing 1- Pumping Well Location

Relevant ground elevations for pumping well and monitoring holes were:

Pumping well: +0.12m
Monitoring station 1: 0.00m (North)
Monitoring station 2: 0.18m (South)

Measurements and Analysis of Results

Observation Borehole 2

Drawdown Data for Observation Borehole 2 (13m South of Pumping Well)					
Water Level Drawdown					
Date	Time	Time since the beginning of pumping (min)	Depth of Water (m)	Drawdown of water level (m) - s	Pumped Well Depth of Water (m)
01/02/2020	07:00	0	1.8	0	2.20
		1	1.8	0	3.40
		2	1.81	0.01	6.50
		3	1.83	0.03	8.30
		4	1.85	0.05	8.70
		5	1.87	0.07	9.50
		6	1.91	0.11	10.00
		7	1.96	0.16	10.30
		8	1.99	0.19	10.50
		9	2.02	0.22	10.50
		10	2.03	0.23	10.50
		15	2.06	0.26	10.50
		20	2.09	0.29	10.50
		25	2.12	0.32	10.50
		30	2.14	0.34	10.50
		60	2.22	0.42	10.50
		120	2.29	0.49	10.50
		180	2.32	0.52	10.50
		240	2.34	0.54	10.50
		300	2.34	0.54	10.50
	13:00	360	2.34	0.54	10.50

Table I - Drawdown Data for Observation Borehole 2 (13m South of Pumping Well)

Recovery Data for Borehole 2 (13m South of Pumping Well)					
Water Level Recovery					
Date	Time since beginning of pumping (min)	Time since the end of pumping (min)	Depth of Water (m)	Pumping Well Water Level (m)	Water Level Recovery (m) Residual Drawdown - s
01/02/2020	360	0	2.34		0.54
	361	1	2.32	3.65	0.52
	362	2	2.29	3.05	0.49
	363	3	2.27	2.72	0.47
	364	4	2.26	2.51	0.46
	365	5	2.24	2.41	0.44
	366	6	2.22	2.28	0.42
	367	7	2.2	2.25	0.40
	368	8	2.19	2.25	0.39
	369	9	2.18	2.25	0.38
	370	10	2.17	2.23	0.37
	375	15	2.13	2.23	0.33
	380	20	2.1	2.22	0.30
	385	25	2.06	2.20	0.26
	390	30	2.03	2.20	0.23
	420	60	1.91	2.20	0.11
	480	120	1.84	2.20	0.04
	660	300	1.81	2.20	0.01

Table II - Recovery Data for Borehole 2 (13m South of Pumping Well)

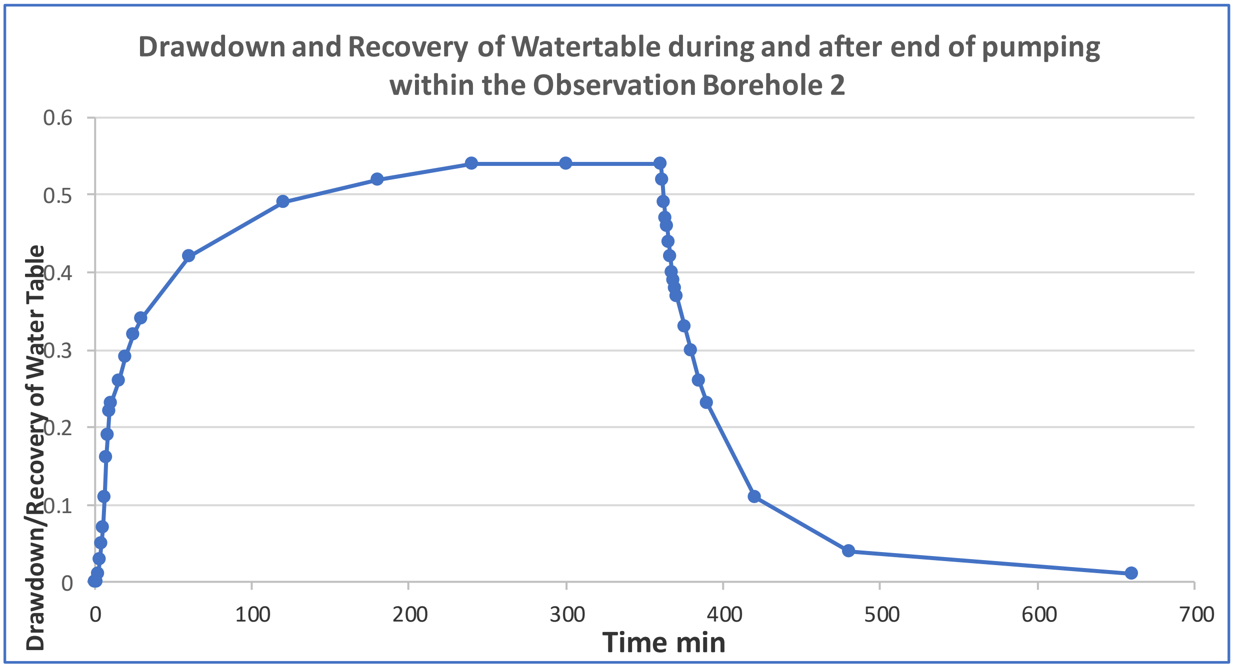
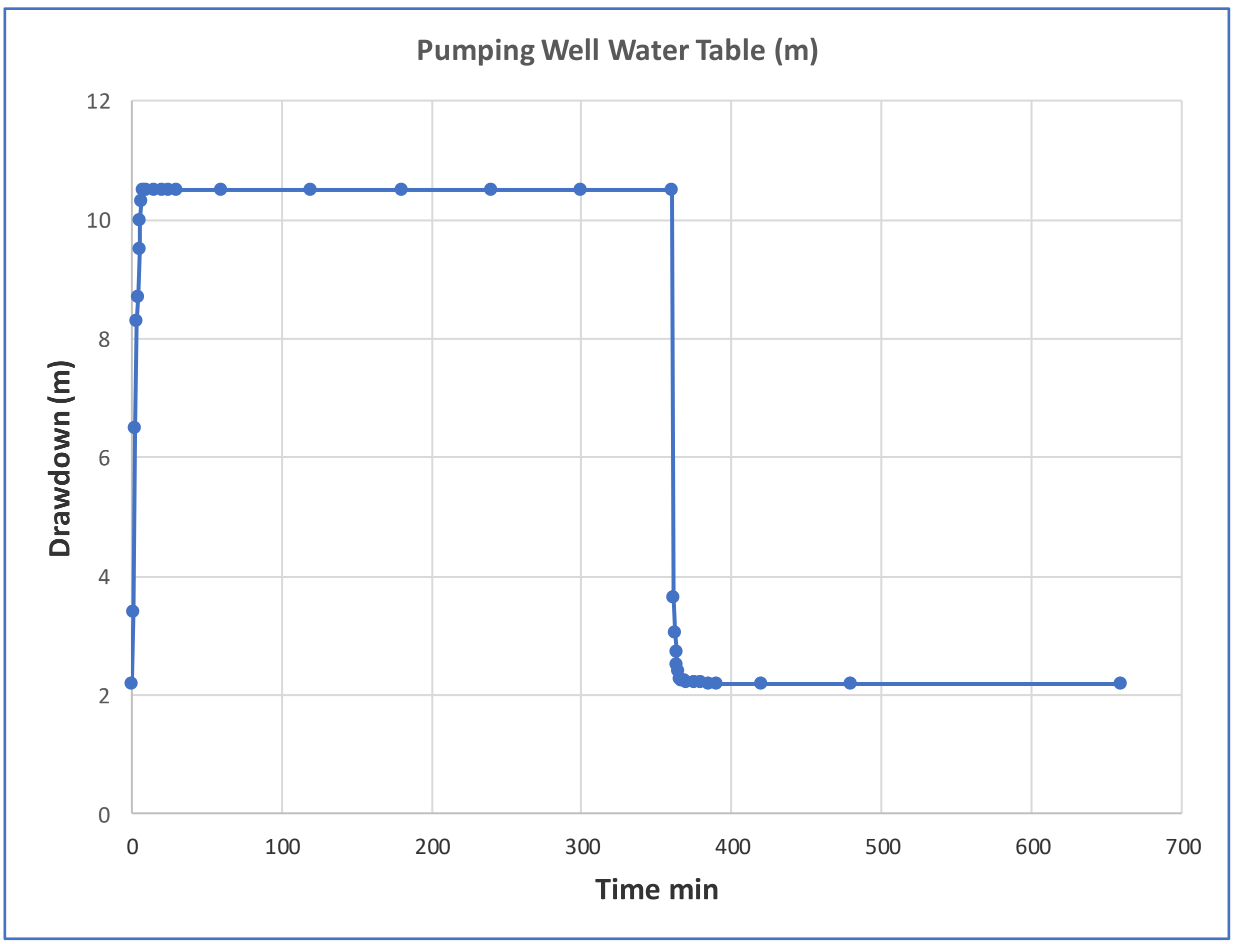


Figure 2 – Drawdown and Recovery – Borehole 2



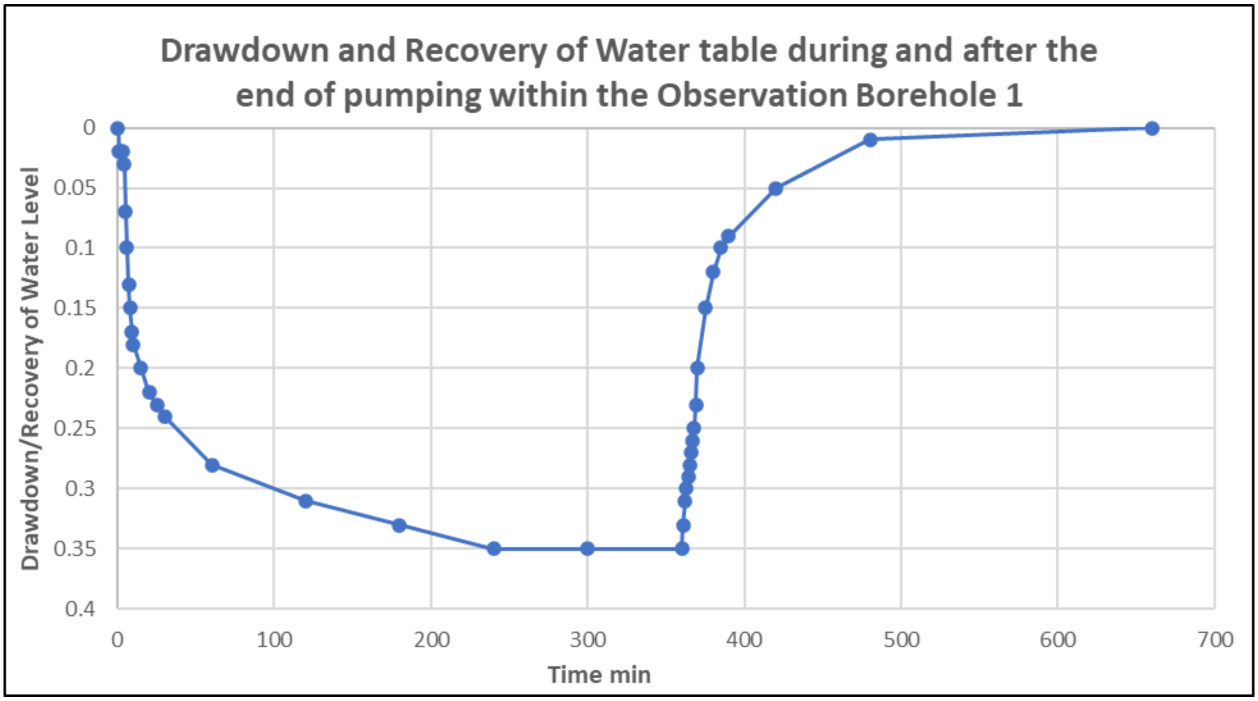
Observation Borehole 1

Drawdown Data for Observation Borehole 1 (13m North of Pumping Well)					
Water Level Drawdown					
Date	Time	Time since the beginning of pumping (min)	Depth of Water (m)	Drawdown of water level (m) - s	Pumped Well Depth of Water (m)
01/02/2020	07:00	0	1.45	0	2.20
		1	1.47	0.02	
		2	1.47	0.02	
		3	1.47	0.02	8.30
		4	1.48	0.03	
		5	1.52	0.07	9.50
		6	1.55	0.10	
		7	1.58	0.13	
		8	1.60	0.15	
		9	1.62	0.17	
		10	1.63	0.18	10.50
		15	1.65	0.20	
		20	1.67	0.22	
		25	1.68	0.23	
		30	1.69	0.24	
		60	1.73	0.28	
		120	1.76	0.31	
		180	1.78	0.33	
		240	1.80	0.35	
		300	1.80	0.35	
	13:00	360	1.80	0.35	10.50

Table III - Drawdown Data for Observation Borehole 1 (13m North of Pumping Well)

Recovery Data for Borehole 1 (13m North of Pumping Well)					
Water Level Recovery					
Date	Time since beginning of pumping (min)	Time since the end of pumping (min)	Depth of Water (m)	Pumping Well Water Level (m)	Water Level Recovery (m) Residual Drawdown - s
01/02/2020	360	0	1.80		0.35
	361	1	1.78	3.65	0.33
	362	2	1.76	3.05	0.31
	363	3	1.75	2.72	0.30
	364	4	1.74	2.51	0.29
	365	5	1.73	2.41	0.28
	366	6	1.72	2.28	0.27
	367	7	1.71	2.25	0.26
	368	8	1.70	2.25	0.25
	369	9	1.68		0.23
	370	10	1.65		0.20
	375	15	1.6		0.15
	380	20	1.57		0.12
	385	25	1.55		0.10
	390	30	1.54		0.09
	420	60	1.5		0.05
	480	120	1.46		0.01
	660	300	1.45		0.00

Table IV - Recovery Data for Borehole 1 (13m North of Pumping Well)



Based on the analysis of the results Appendix I (ii) the following parameters have been established

Contact Info **Address** **Company Name** **City, State/Province**				**Pumping Test Analysis Report** Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES				
Location: LIMASSOL			Pumping Test: Pumping Test 1			Pumping Well: PW1		
Test Conducted by:						Test Date: 1/2/2020		
Aquifer Thickness: 20.00 m			Discharge Rate: 36 [m³/h]					
	Analysis Name	Analysis Performed by	Analysis Date	Method name	Well	T [m²/d]	K [m/d]	S
1	Boulton	H. Kridiotis	25/11/2020	Boulton	PW1	1.00×10^{2}	5.00×10^{0}	1.25×10^{-4}
2	Boulton	H. Kridiotis	25/11/2020	Boulton	OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-1}
3	Boulton	H. Kridiotis	25/11/2020	Boulton	OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-1}
4	Theis-Jacob Correction	H.Kridiotis	1/12/2020	Theis with Jacob Correct	PW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
5	Theis-Jacob Correction	H.Kridiotis	1/12/2020	Theis with Jacob Correct	OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
6	Theis-Jacob Correction	H.Kridiotis	1/12/2020	Theis with Jacob Correct	OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
7	Papodopoulos & Cooper	H. Kridiotis	1/12/2020	Papadopulos & Cooper	PW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
8	Papodopoulos & Cooper	H. Kridiotis	1/12/2020	Papadopulos & Cooper	OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
9	Papodopoulos & Cooper	H. Kridiotis	1/12/2020	Papadopulos & Cooper	OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
					Average	8.79×10^{1}	4.40×10^{0}	2.23×10^{-2}

APPENDIX I (ii)

Pumping Test – Software Output

Contact Info
Address
Company Name
City, State/Province

Pumping Test - Water Level Data

Project: "ASKANIS GALLERY" PROJECT

Number: 1

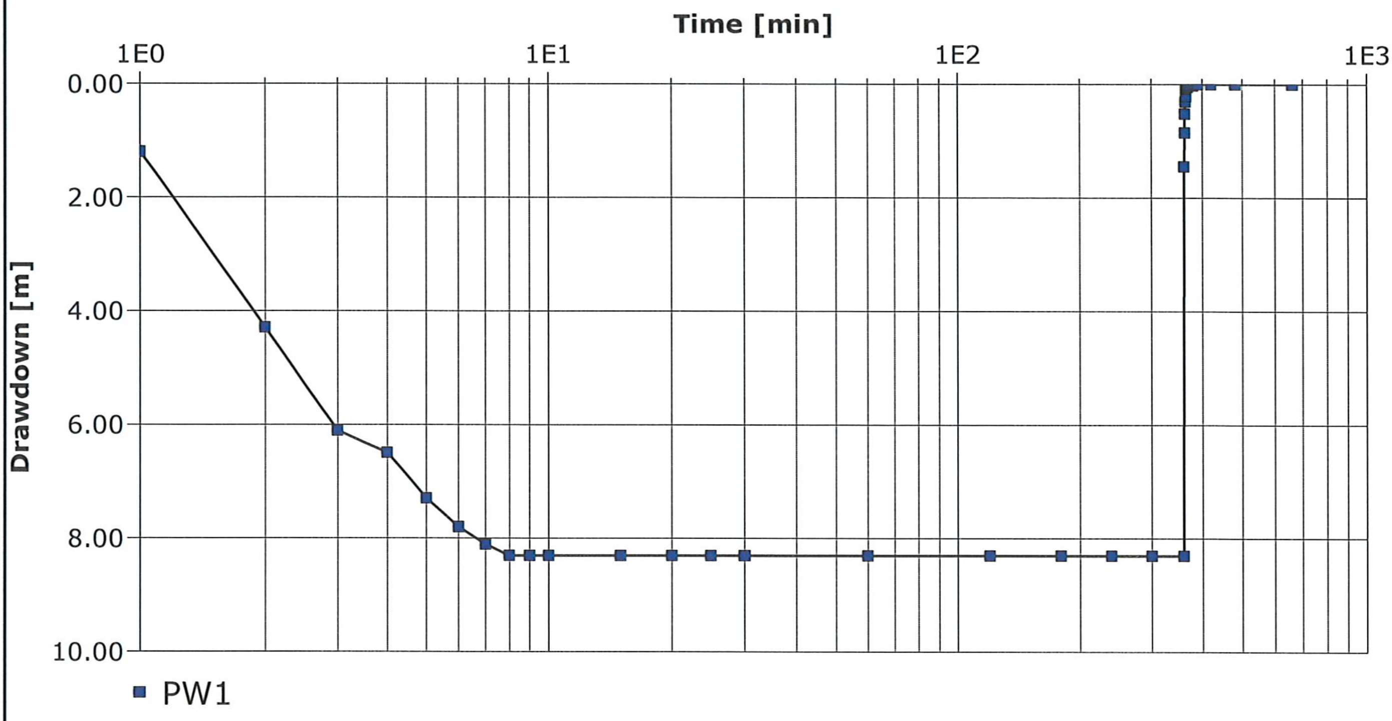
Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:	Test Date: 1/2/2020	Discharge Rate: 36 [m³/h]
Observation Well: PW1	Static Water Level [m]: 2.20	Radial Distance to PW [m]: -

	Time [min]	Water Level [m]	Drawdown [m]
1	0	2.20	0.00
2	1	3.40	1.20
3	2	6.50	4.30
4	3	8.30	6.10
5	4	8.70	6.50
6	5	9.50	7.30
7	6	10.00	7.80
8	7	10.30	8.10
9	8	10.50	8.30
10	9	10.50	8.30
11	10	10.50	8.30
12	15	10.50	8.30
13	20	10.50	8.30
14	25	10.50	8.30
15	30	10.50	8.30
16	60	10.50	8.30
17	120	10.50	8.30
18	180	10.50	8.30
19	240	10.50	8.30
20	300	10.50	8.30
21	360	10.50	8.30
22	361	3.65	1.45
23	362	3.05	0.85
24	363	2.72	0.52
25	364	2.51	0.31
26	365	2.41	0.21
27	366	2.28	0.08
28	367	2.25	0.05
29	368	2.25	0.05
30	369	2.25	0.05
31	370	2.23	0.03
32	375	2.23	0.03
33	380	2.22	0.02
34	385	2.20	0.00
35	390	2.20	0.00
36	420	2.20	0.00
37	480	2.20	0.00
38	660	2.20	0.00

Contact Info Address Company Name City, State/Province		**Aquifer Test Analysis**
		Project: "ASKANIS GALLERY" PROJECT
		Number: 1
		Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by:	Time-Drawdown	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

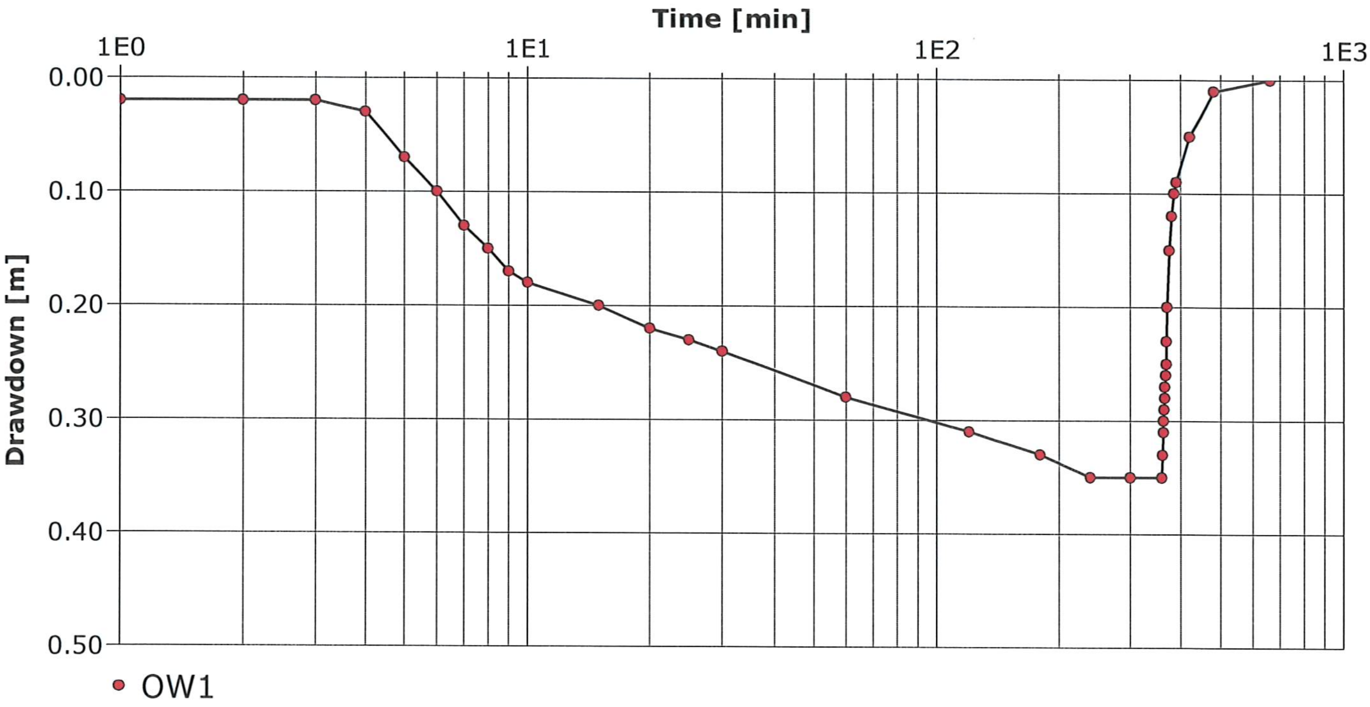


Contact Info Address Company Name City, State/Province	**Pumping Test - Water Level Data** Page 1 of 1
	Project: "ASKANIS GALLERY" PROJECT
	Number: 1
	Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:	Test Date: 1/2/2020	Discharge Rate: 36 [m³/h]
Observation Well: OW1	Static Water Level [m]: 1.45	Radial Distance to PW [m]: 13

	Time [min]	Water Level [m]	Drawdown [m]
1	0	1.45	0.00
2	1	1.47	0.02
3	2	1.47	0.02
4	3	1.47	0.02
5	4	1.48	0.03
6	5	1.52	0.07
7	6	1.55	0.10
8	7	1.58	0.13
9	8	1.60	0.15
10	9	1.62	0.17
11	10	1.63	0.18
12	15	1.65	0.20
13	20	1.67	0.22
14	25	1.68	0.23
15	30	1.69	0.24
16	60	1.73	0.28
17	120	1.76	0.31
18	180	1.78	0.33
19	240	1.80	0.35
20	300	1.80	0.35
21	360	1.80	0.35
22	361	1.78	0.33
23	362	1.76	0.31
24	363	1.75	0.30
25	364	1.74	0.29
26	365	1.73	0.28
27	366	1.72	0.27
28	367	1.71	0.26
29	368	1.70	0.25
30	369	1.68	0.23
31	370	1.65	0.20
32	375	1.60	0.15
33	380	1.57	0.12
34	385	1.55	0.10
35	390	1.54	0.09
36	420	1.50	0.05
37	480	1.46	0.01
38	660	1.45	0.00

Contact Info Address Company Name City, State/Province		**Aquifer Test Analysis**
		Project: "ASKANIS GALLERY" PROJECT
		Number: 1
		Client: ASKANIS GROUP OF COMPANIES
Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by:	Time-Drawdown	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	



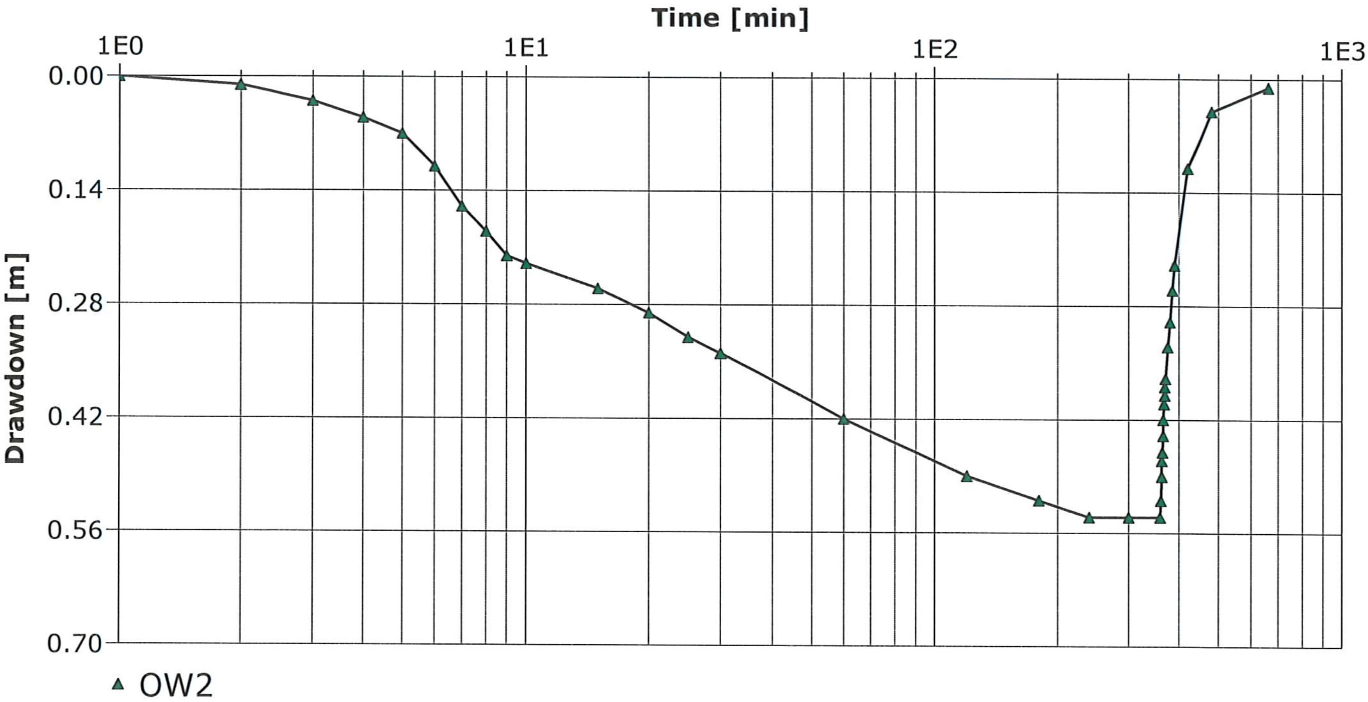
Contact Info Address Company Name City, State/Province	**Pumping Test - Water Level Data** Page 1 of 1 Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:	Test Date: 1/2/2020	Discharge Rate: 36 [m³/h]
Observation Well: OW2	Static Water Level [m]: 1.80	Radial Distance to PW [m]: 13

	Time [min]	Water Level [m]	Drawdown [m]
1	0	1.80	0.00
2	1	1.80	0.00
3	2	1.81	0.01
4	3	1.83	0.03
5	4	1.85	0.05
6	5	1.87	0.07
7	6	1.91	0.11
8	7	1.96	0.16
9	8	1.99	0.19
10	9	2.02	0.22
11	10	2.03	0.23
12	15	2.06	0.26
13	20	2.09	0.29
14	25	2.12	0.32
15	30	2.14	0.34
16	60	2.22	0.42
17	120	2.29	0.49
18	180	2.32	0.52
19	240	2.34	0.54
20	300	2.34	0.54
21	360	2.34	0.54
22	361	2.32	0.52
23	362	2.29	0.49
24	363	2.27	0.47
25	364	2.26	0.46
26	365	2.24	0.44
27	366	2.22	0.42
28	367	2.20	0.40
29	368	2.19	0.39
30	369	2.18	0.38
31	370	2.17	0.37
32	375	2.13	0.33
33	380	2.10	0.30
34	385	2.06	0.26
35	390	2.03	0.23
36	420	1.91	0.11
37	480	1.84	0.04
38	660	1.81	0.01

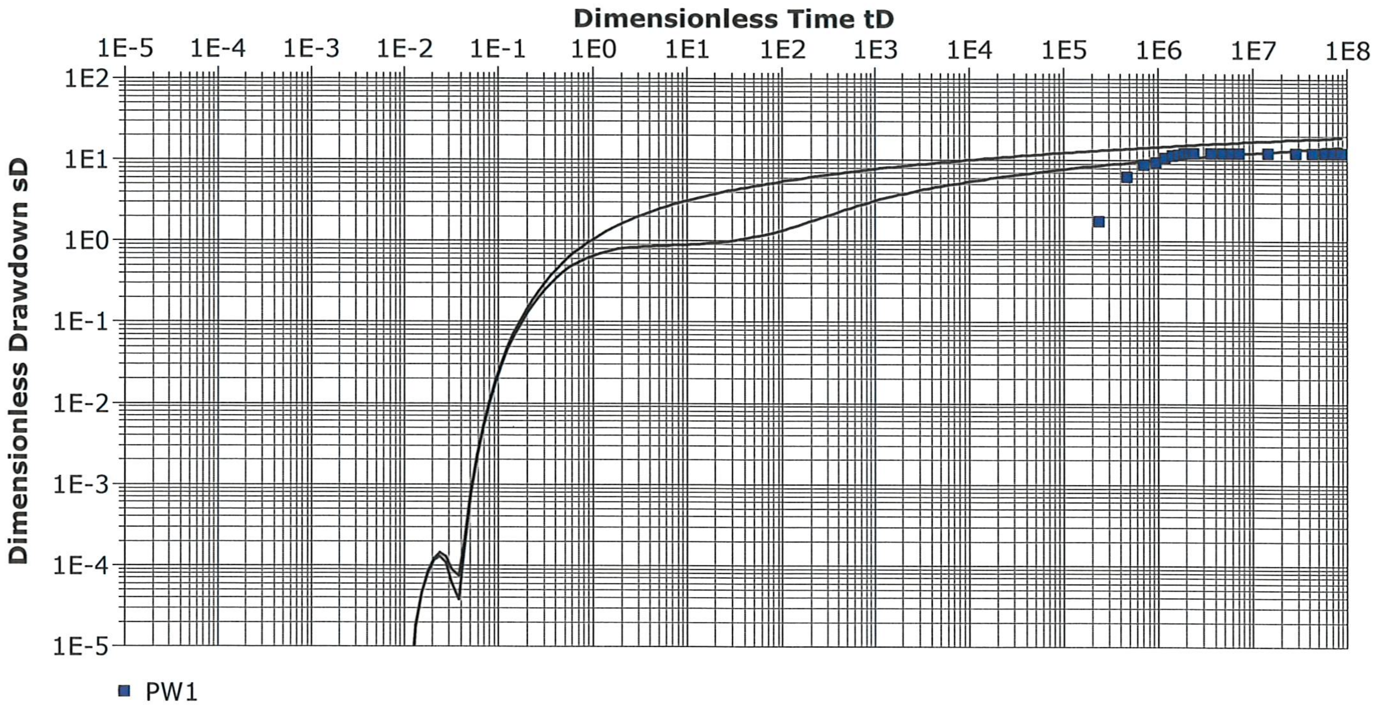
	Contact Info **Address** **Company Name** **City, State/Province**	**Aquifer Test Analysis** Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by:	Time-Drawdown	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	



	Contact Info **Address** **Company Name** **City, State/Province**	**Aquifer Test Analysis** Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H. Kridiotis	Boulton	Analysis Date: 25/11/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

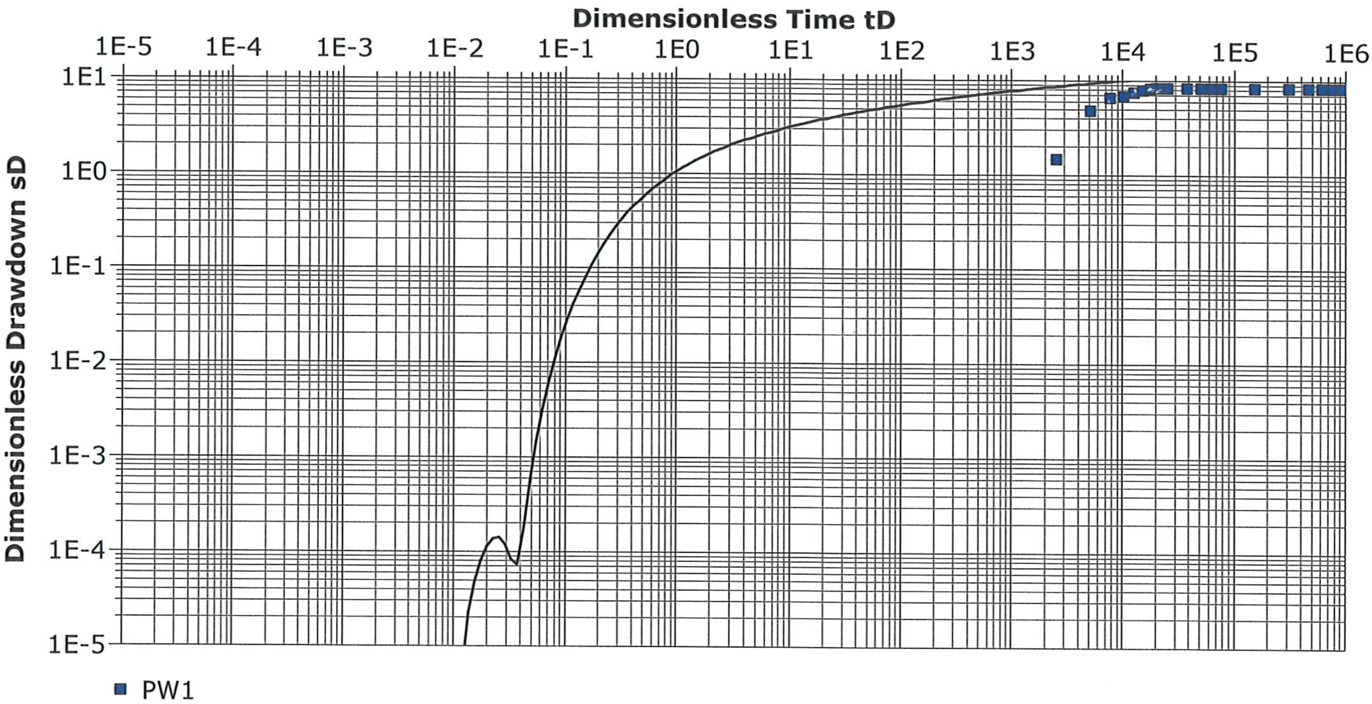


Calculation using Boulton

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Specific Yield	Drainage factor	Ratio Sy/S	Radial Distance to PW [m]
PW1	1.00×10^{2}	5.00×10^{0}	1.25×10^{-4}	1.00×10^{-2}	1.00×10^{2}	0.48

Contact Info Address Company Name City, State/Province	Aquifer Test Analysis
	Project: "ASKANIS GALLERY" PROJECT
	Number: 1
	Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H.Kridiotis	Theis-Jacob Correction	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

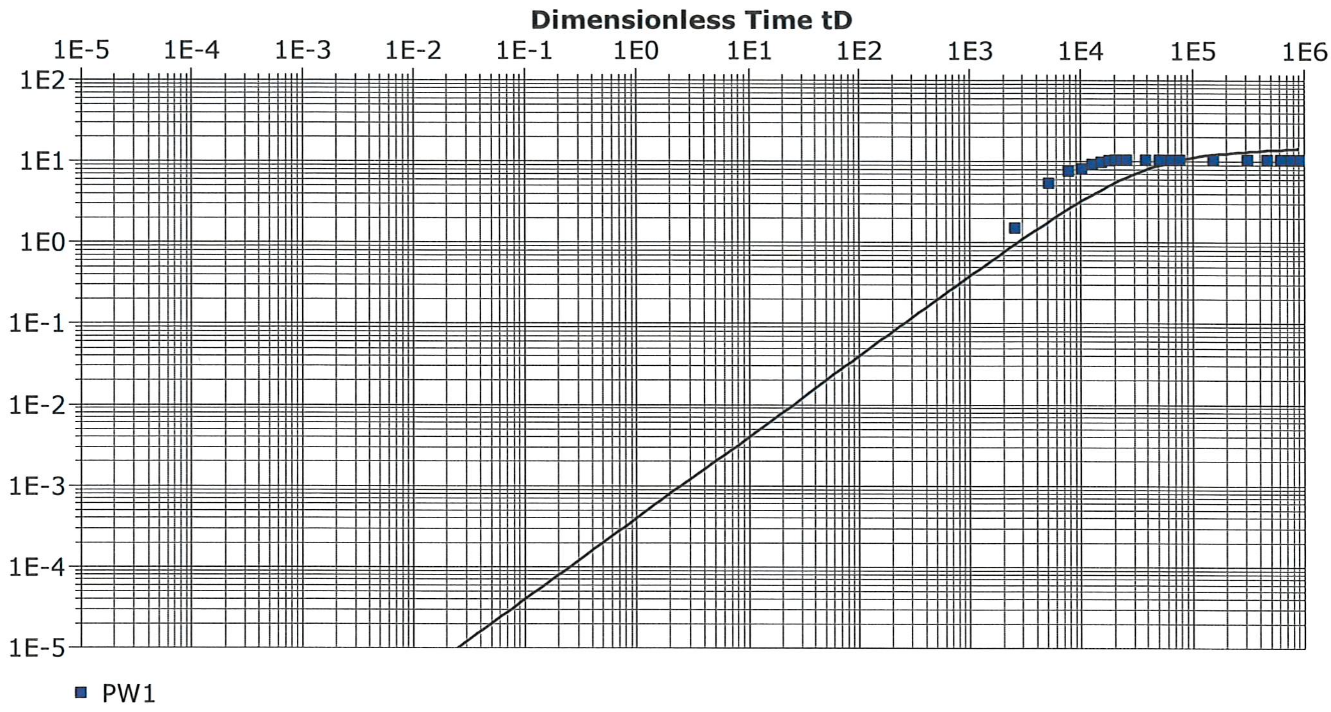


Calculation using Theis with Jacob Correction

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Storage coefficient	Radial Distance to PW [m]	
PW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}	0.48	

Contact Info **Address** **Company Name** **City, State/Province**	**Aquifer Test Analysis** Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H. Kridiotis	Papodopoulos & Cooper	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

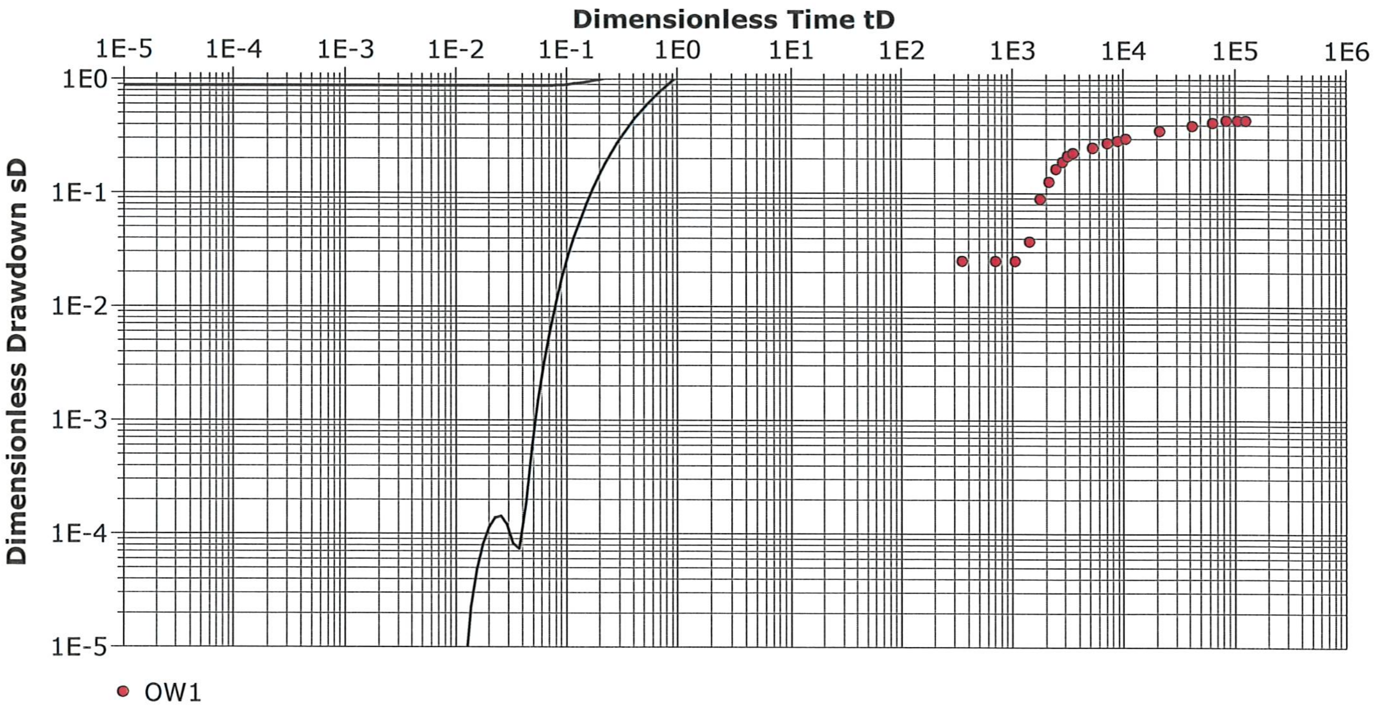


Calculation using Papadopulos & Cooper

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Well-bore storage coefficient	Radial Distance to PW [m]	
PW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}	0.48	

Contact Info Address Company Name City, State/Province		**Aquifer Test Analysis**
		Project: "ASKANIS GALLERY" PROJECT
		Number: 1
		Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H. Kridiotis	Boulton	Analysis Date: 25/11/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

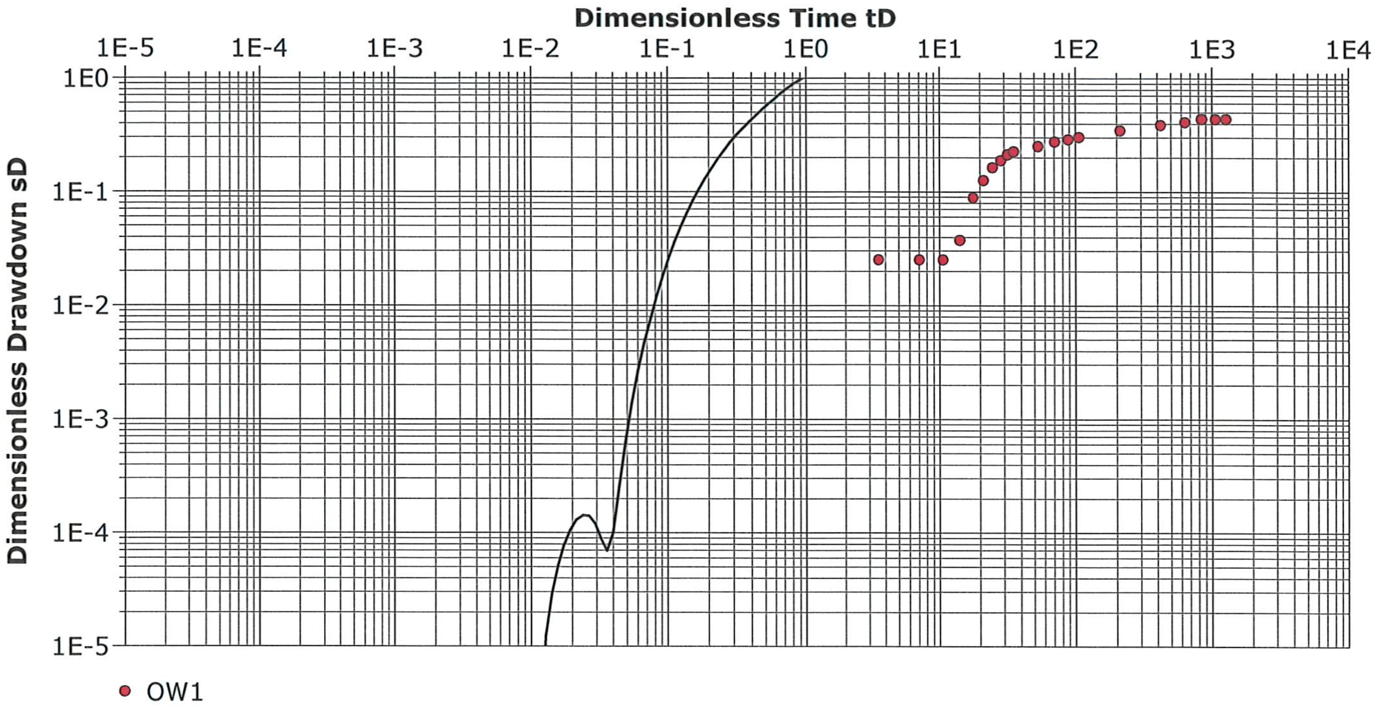


Calculation using Boulton

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Specific Yield	Drainage factor	Ratio Sy/S	Radial Distance to PW [m]
OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}	1.00×10^{-2}	1.00×10^{2}	13.0

Contact Info Address Company Name City, State/Province	**Aquifer Test Analysis** Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H.Kridiotis	Theis-Jacob Correction	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

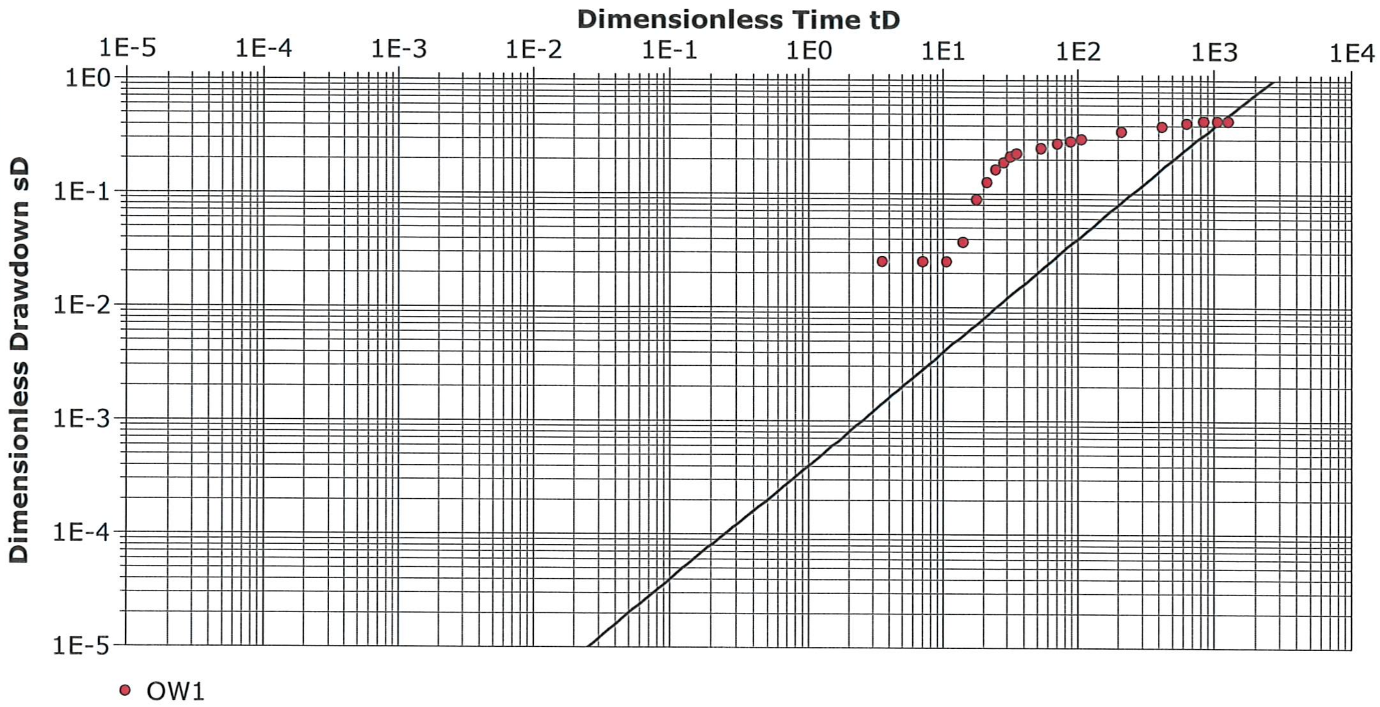


Calculation using Theis with Jacob Correction

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Storage coefficient	Radial Distance to PW [m]	
OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}	13.0	

	Contact Info Address Company Name City, State/Province	**Aquifer Test Analysis** Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H. Kridiotis	Papodopoulos & Cooper	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

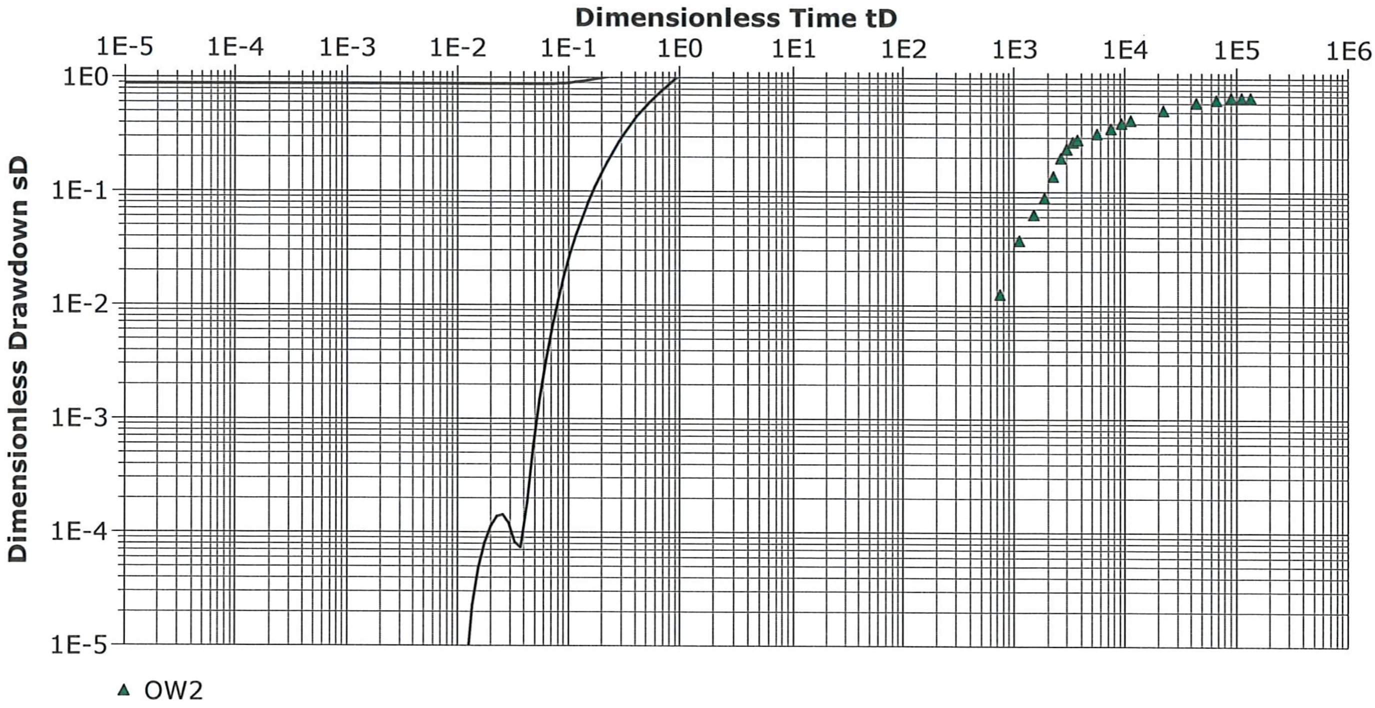


Calculation using Papadopulos & Cooper

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Well-bore storage coefficient	Radial Distance to PW [m]	
OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}	13.0	

	Contact Info Address Company Name City, State/Province	Aquifer Test Analysis
		Project: "ASKANIS GALLERY" PROJECT
		Number: 1
		Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H. Kridiotis	Boulton	Analysis Date: 25/11/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

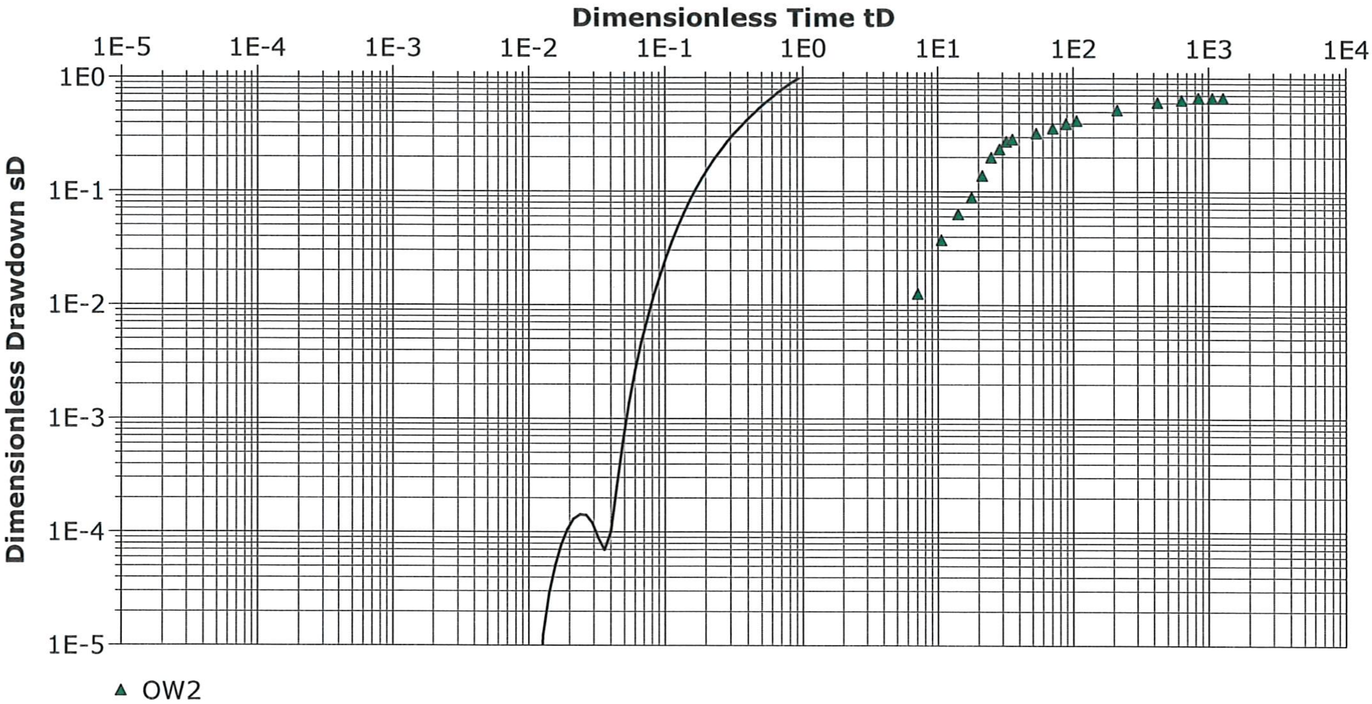


Calculation using Boulton

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Specific Yield	Drainage factor	Ratio Sy/S	Radial Distance to PW [m]
OW2	8.64×10^{1}	4.32×10^{0}	9.51×10^{-5}	1.00×10^{-2}	1.00×10^{2}	13.0

Contact Info **Address** **Company Name** **City, State/Province**	**Aquifer Test Analysis** Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H.Kridiotis	Theis-Jacob Correction	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

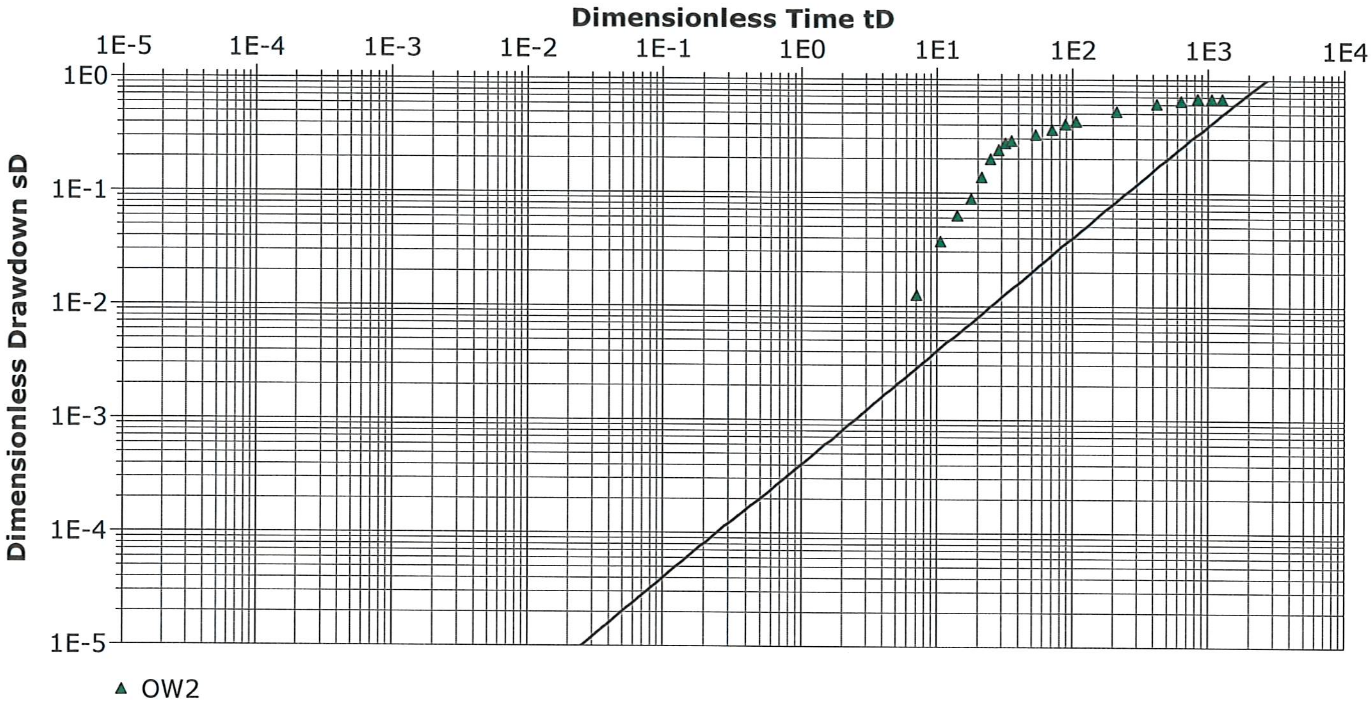


Calculation using Theis with Jacob Correction

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Storage coefficient	Radial Distance to PW [m]	
OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}	13.0	

Contact Info Address Company Name City, State/Province		**Aquifer Test Analysis**
		Project: "ASKANIS GALLERY" PROJECT
		Number: 1
		Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Analysis Performed by: H. Kridiotis	Papodopoulos & Cooper	Analysis Date: 1/12/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	



Calculation using Papadopulos & Cooper

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Well-bore storage coefficient	Radial Distance to PW [m]	
OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}	13.0	

Contact Info Address Company Name City, State/Province	Pumping Test Analysis Report Project: "ASKANIS GALLERY" PROJECT Number: 1 Client: ASKANIS GROUP OF COMPANIES

Location: LIMASSOL	Pumping Test: Pumping Test 1	Pumping Well: PW1
Test Conducted by:		Test Date: 1/2/2020
Aquifer Thickness: 20.00 m	Discharge Rate: 36 [m³/h]	

	Analysis Name	Analysis Performed by	Analysis Date	Method name	Well	T [m²/d]	K [m/d]	S
1	Boulton	H. Kridiotis	25/11/2020	Boulton	PW1	1.00×10^{2}	5.00×10^{0}	1.25×10^{-4}
2	Boulton	H. Kridiotis	25/11/2020	Boulton	OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-1}
3	Boulton	H. Kridiotis	25/11/2020	Boulton	OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-1}
4	Theis-Jacob Correction	H.Kridiotis	1/12/2020	Theis with Jacob Correctio	PW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
5	Theis-Jacob Correction	H.Kridiotis	1/12/2020	Theis with Jacob Correctio	OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
6	Theis-Jacob Correction	H.Kridiotis	1/12/2020	Theis with Jacob Correctio	OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
7	Papodopoulos & Cooper	H. Kridiotis	1/12/2020	Papadopulos & Cooper	PW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
8	Papodopoulos & Cooper	H. Kridiotis	1/12/2020	Papadopulos & Cooper	OW1	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
9	Papodopoulos & Cooper	H. Kridiotis	1/12/2020	Papadopulos & Cooper	OW2	8.64×10^{1}	4.32×10^{0}	1.00×10^{-4}
					Average	8.79×10^{1}	4.40×10^{0}	2.23×10^{-2}

ASKANIS GROUP OF COMPANIES
Gallery Project

Μεθοδολογία Αποστράγγισης

Εκτελεστική Περίληψη του Αγγλικού Κειμένου με ημερομηνία Δεκέμβριος 2020

Δεκέμβριος 2020

Εκτελεστική Περίληψη

Εισαγωγή

1. Η εταιρεία Askanis Group of Companies προγραμματίζει την ανέγερση πολυώροφου κτιρίου στην Οδό Αγίου Ανδρέου στη Λεμεσό. Η λεπτομερής περιγραφή του κτιρίου περιλαμβάνεται στην ΜΕΕΠ που έχει κατατεθεί στο Τμήμα Περιβάλλοντος.

2. Το οικόπεδο έχει εμβαδό περίπου 2,000 m^2 και, το ολικό εμβαδό σε κάτοψη (building footprint), για το οποίο θα γίνει εκσκαφή στο βάθος θεμελίωσης, θα είναι 1,630 m^2. Επιπλέον θα κατασκευαστούν δύο υπόγεια με υπολογιζόμενο βάθος θεμελίωσης τα 8 μέτρα, κάτω από το υφιστάμενο υψόμετρο εδάφους. Επιπλέον, τα φρεάτια των ανελκυστήρων θα εδράζονται σε βάθος 10.5 μέτρα, κάτω από το υφιστάμενο υψόμετρο εδάφους.

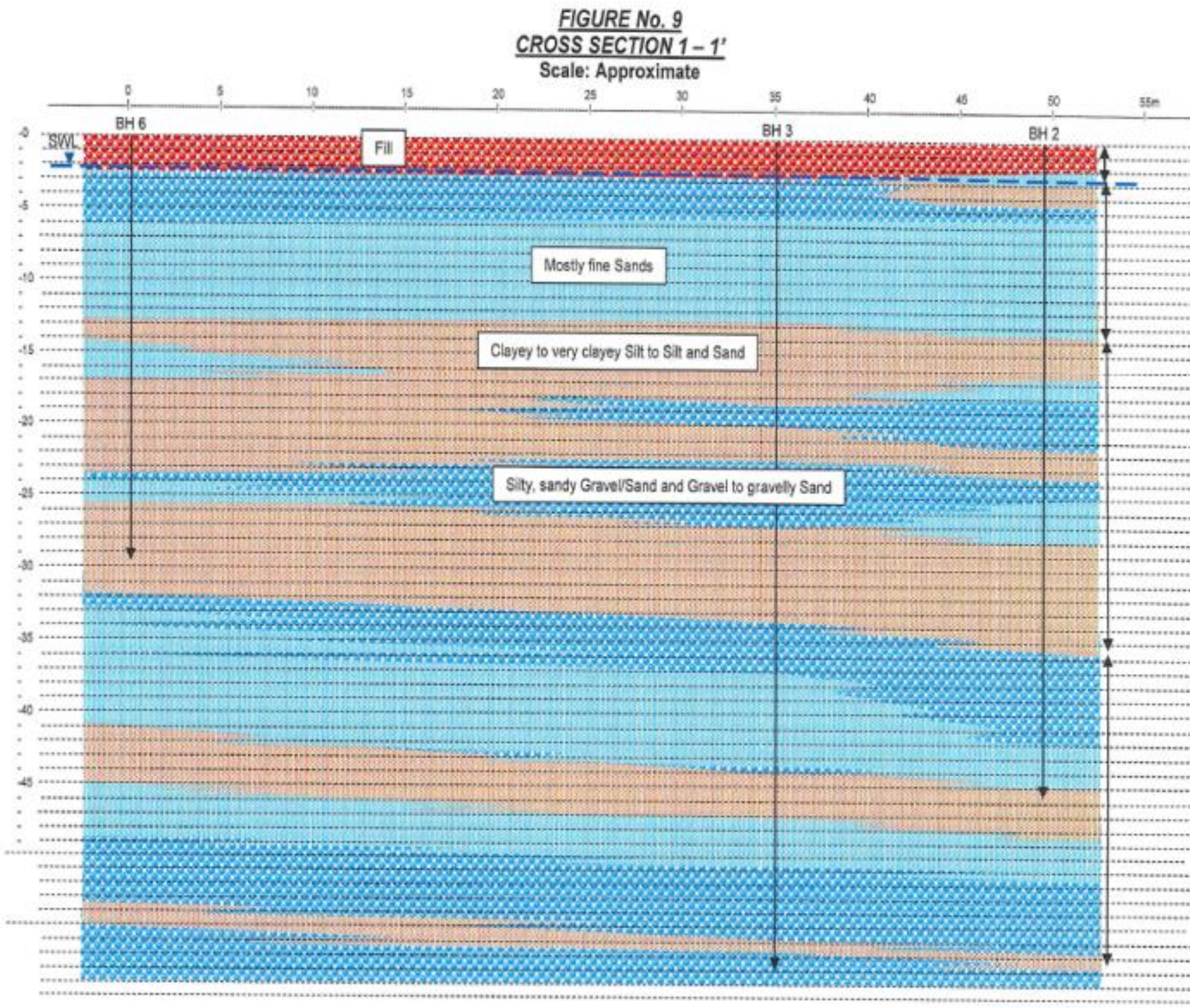
3. Αναμένεται ότι, για τον έλεγχο της οριζόντιας ροής νερού προς την εκσκαφή θα κατασκευαστεί μόνιμος περιμετρικός υδατοστεγής τοίχος (διάφραγμα/πασσαλότοιχος), σε βάθος τουλάχιστο 4 μέτρα κάτω από το χαμηλότερο σημείο γενικής εκσκαφής. Ο τοίχος θα είναι πακτωμένος εντός του γεωλογικού ορίζοντα C1, όπου σύμφωνα με τα ευρήματα της Γεωτεχνικής Μελέτης (Geoinvest - 2019), αναμένεται η παρουσία αυξημένης ποσότητας ιλύς/αργίλου (silt/clay) και επομένως <u>σχετικά</u> μειωμένη υδροπερατότητα της τάξης των 4.7×10^{-5} cm/s.

4. Σκοπός της μελέτης είναι ο υπολογισμός της εκτιμώμενης ποσότητας νερού άντλησης κατά την διάρκεια εκσκαφής του υπογείου και θεμελιώσεων του προτεινόμενου κτιρίου και, λαμβάνοντας υπόψη τις διάφορες φάσεις κατασκευής, να προταθεί η προτιμητέα μέθοδος αποστράγγισης καθώς και η μέθοδος απόρριψης του νερού άντλησης.

Υδρογεωλογικές συνθήκες

5. Από έρευνες που έγιναν στην περιοχή του τεμαχίου έχει διαφανεί ότι η στάθμη του υπόγειου υδροφορέα, βρίσκεται σε βάθος 1.60 με 2.20 μέτρα κάτω από την υφιστάμενη επιφάνεια του εδάφους. Το Νοέμβριο του 2019 και Φεβρουάριο του 2020, έγιναν πλήρεις δοκιμαστικές αντλήσεις νερού (pumping tests), για καλύτερο προσδιορισμό των υδρογεωλογικών χαρακτηριστικών του υδροφορέα/ων.

Στο πιο κάτω σχήμα παρουσιάζεται η γεωλογική τομή του υπεδάφους, σύμφωνα με τη γεωτεχνική Έρευνα.



Geological Cross Section *(Geoinvest, 2019)*

Έχουν εκπονηθεί εργαστηριακές αναλύσεις για τον καθορισμό της υδροπερατότητας των γεωλογικών στρώσεων που συναντιούνται στο υπέδαφος με τα ακόλουθα αποτελέσματα:

Group B – 8.8x10^{-4} cm/s (Βάθος από 2-13.5 μέτρα)
Group C- 4.7x10^{-5} cm/s (Βάθος από 13.5 – 45 μέτρα
Group C1 1.8x10^{-4} cm/s (βάθη από 13.50-18, 21.5-23.5, 28-35, πέραν των 45 μέτρων)

Ποσότητες άντλησης νερού

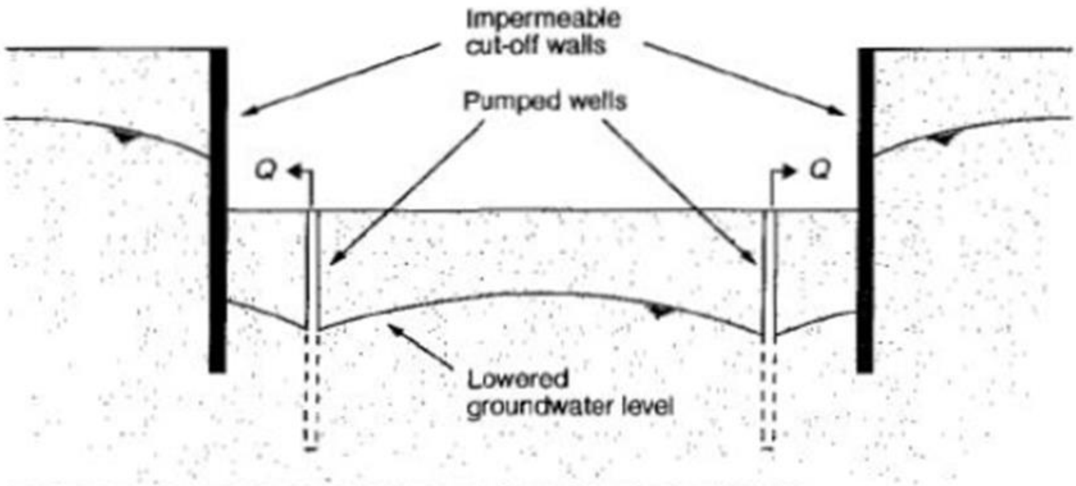
6. Λαμβάνοντας υπόψη την γεωλογική στρωματογραφία της περιοχής του Έργου, τα βάθη του υπόγειου υδροφόρου ορίζοντα, το βάθος της προτεινόμενης εκσκαφής, την κατασκευή περιμετρικού στεγανού διαφραγματικού τοίχου, τα αποτελέσματα των γεωτεχνικών ερευνών και εργαστηριακών δοκιμών και, τα αποτελέσματα της δοκιμαστικής άντλησης, έγινε υπολογισμός των ποσοτήτων νερού που αναμένεται να εισρέουν εντός της εκσκαφής κατά την διάρκεια της κατασκευής, με ανάλυση με την μέθοδο flow net θεωρώντας την εισροή νερού μεταξύ των κάθετων διαφραγματικών τοίχων, οι οποίοι αποτελούν flow lines-αδιαπέραστα στοιχεία, επειδή το νερό δεν μπορεί να διεισδύσει διαμέσου των.

Επίσης, λήφθηκαν σοβαρά υπόψη, οι εισροές υπογείου νερού που παρατηρήθηκαν σε παρόμοια κατασκευασθέντα ή υπό κατασκευή έργα, στην ευρύτερη παραλιακή ζώνη της Λεμεσού, σε παρόμοιους γεωλογικούς σχηματισμούς.

7. Υπολογίζεται ότι ο διαφραγματικός τοίχος θα έχει βάθος περίπου 14 μέτρα. Λαμβάνοντας υπόψη τα αποτελέσματα της γεωτεχνικής έρευνας και των δοκιμαστικών αντλήσεων (Pumping Test), καθώς και την κατασκευή αδιαπέραστου διαφραγματικού τοίχου, υπολογίζεται ότι η εισροή νερού στην εκσκαφή που θα γίνει, θα ανέρχεται περίπου στα 1.1 με 1.3 m^3 ανά τετραγωνικό μέτρο ανά ημέρα. Δηλαδή για ολόκληρη την εκσκαφή θα προκύψει ανάγκη άντλησης και απόρριψης , περίπου 1.900 κυβικών μέτρων νερού ανα ημέρα ή 950 κυβικών μέτρων νερού ανα ημέρα, αν η εκσκαφή γίνει σε δύο ίσου εμβαδού στάδια. Για σκοπούς άντλησης θα ανορυχθούν φρεάτια εντός της εκσκαφής του υπογείου, περιμετρικά και σε απόσταση περίπου 2.5 – 3 μέτρα από τον πασσαλότοιχο, καθώς και εντός της περιοχής της εκσκαφής (Διάγραμμα 1 της Έκθεσης). Επιπλέον, συστήνεται η ανόρυξη γεώτρησης άντλησης στην περιοχή εκσκαφής του φρεατίου του ανελκυστήρα.

Με βάση τα πιο πάνω, θα χρειάζεται ολική άντληση της τάξης των 100 m^3 /hour, για ταπείνωση του υδροφορέα τουλάχιστο 0.5 με 1 μέτρο κάτω από το βάση της τελικής εκσκαφής.

8. Υπολογίζεται ότι στην περίπτωση κατά την οποία η εκσκαφή θα γίνει σε μία φάση, θα χρειαστούν 10 - 12 γεωτρήσεις άντλησης, σε συνδυασμό με φρεάτια αποστράγγισης (sump wells) μικρού βάθους (1–2 μέτρα), εάν καταστεί αναγκαίο, που θα εγκατασταθούν στον χώρο της εκσκαφής.



b) Excavation with a retaining wall and wells to prevent water ingress through the base

Fig.3 Pump-Well System

Πιο πάνω παρουσιάζεται διαγραμματικά, παράδειγμα του προτεινόμενου συστήματος άντλησης.

Διάθεση Αντλούμενου νερού

9. Για το συγκεκριμένο έργο προτείνεται η λύση της διάθεσης του αντλούμενου νερού σε φρεάτια με βάθος πέραν των 20 μέτρων, έτσι ώστε να διοχετευθούν οι ποσότητες του νερού σε κατάλληλο γεωλογικό σχηματισμό (Ορίζοντα C2 – Αλλουβιακές προσχώσεις), σε ζώνες όπου η κύρια διαβάθμιση των υλικών είναι αμμοχάλικα, με ικανοποιητική διαπερατότητα. Προκαταρκτικά υπολογίζεται ότι, για να επιτευχθεί το πιο πάνω πρέπει να ανορυχθούν οκτώ φρεάτια (εξωτερική διάμετρος τρύπας 912μμ, εσωτερική διάμετρος θωράκισης 300mm). Για αντιμετώπιση της ανάγκης συντήρησης λόγω μπλοκαρίσματος (clogging) κάποιων από τα φρεάτια επανα-εισαγωγής, συστήνεται όπως υπάρχουν 2-3 εφεδρικά φρεάτια.

Όπως επεξηγείται στην Έκθεση Αποστράγγισης, σε πρόσφατες δοκιμές σε παρόμοιους γεωλογικούς σχηματισμούς κατέστη δυνατή η επαναφόρτιση γεώτρησης (recharge well), με 20 m^3/hour (Renaissance Project, NEO Project).

Στο κατασκευαστικό στάδιο θα γίνουν «full scale» επι τόπου δοκιμαστικές αντλήσεις νερού εντός φρεατίου(ων) (infiltration tests), για περαιτέρω επαλήθευση των πιο πάνω.

Το πιο κάτω σχήμα υποδεικνύει τυπική τομή φρεατίου επαναφόρτισης.

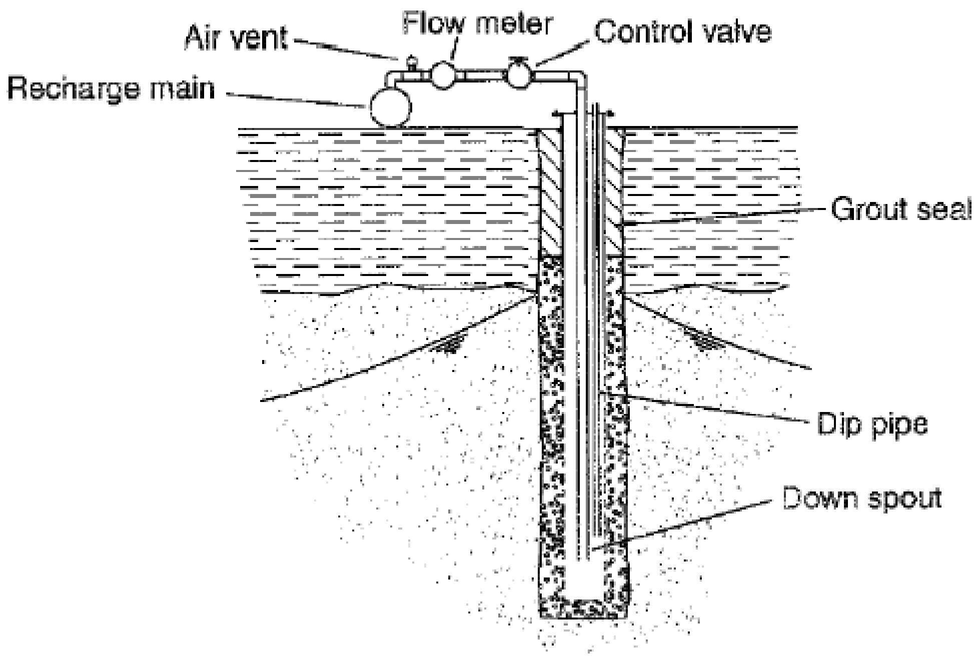


Fig.3 Recharge Well

Φρεάτιο Επαναφόρτισης

Μείωση της συγκέντρωσης των στερεών στο αντλούμενο νερό.

10. Για να μειωθεί η συγκέντρωση των αιωρούμενων στερεών στο αντλούμενο νερό είναι αναγκαία η κατασκευή δεξαμενής καθίζησης με διαστάσεις, τουλάχιστο, 10 μέτρα μήκος, 3 μέτρα βάθος και 3 μέτρα πλάτος έτσι ώστε να επιτευχθεί υδραυλικός χρόνος παραμονής 0.4 ώρες και μείωση των αιωρούμενων στερεών στα 30mg/l. Πιο κάτω παρουσιάζεται σχηματικό της δεξαμενής καθίζησης.

Τονίζεται ότι, για την επιτυχή λειτουργία του συστήματος επιστροφής του αντλούμενου νερού πίσω στον υδροφορέα, είναι απόλυτα αναγκαία η μέγιστη μείωση των στερεών στο αντλούμενο νερό για αποφυγή μπλοκαρίσματος των γεωτρήσεων επαναφόρτισης.