PRIME PROPERTY GROUP
Renaissance Project

Μεθοδολογία Αποστράγγισης

Εκτελεστική Περίληψη του Αγγλικού Κειμένου με ημερομηνία Νοέμβριος 2020

Νοέμβριος 2020

Εκτελεστική Περίληψη

1. Σύμφωνα με τα ευρήματα της γεωτεχνικής έρευνας η οποία έγινε εντός του οικοπέδου της προτεινόμενης ανάπτυξής (12 διερευνητικές γεωτρήσεις σε μέγιστο βάθος 12 μέτρων), αναμένεται ότι ο υδροφόρος ορίζοντας ευρίσκεται σε βάθος που κυμαίνεται από 3.5 μέχρι 5 μέτρα.

2. 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.

3. Σύμφωνα με την Γεωτεχνική Μελέτη (Geoinvest - 2019), η περισσότερη εισροή νερού εντός των εκσκαφών αναμένεται από τον γεωλογικό ορίζοντα C, ο οποίος αποτελείται κυρίως από Αλλουβιακές προσχώσεις, και ο οποίος επεκτείνεται σε βάθος 11.80 μέτρων, και παρουσιάζει υδροπερατότητα της τάξης των 1.5 x10^{-2} cm/s ή 13 m/day.

4. Το ολικό εμβαδό σε κάτοψη (building footprint), για το οποίο θα γίνει εκσκαφή στο βάθος θεμελίωσης, θα είναι 10,458 m^2 . Θα κατασκευαστεί ένα υπόγειο με υπολογιζόμενο βάθος θεμελίωσης στα 4.5 με 5 μέτρα, κάτω από το υφιστάμενο υψόμετρο εδάφους.

5. Αναμένεται ότι, για τον έλεγχο της οριζόντιας ροής νερού προς την εκσκαφή θα κατασκευαστεί μόνιμος περιμετρικός υδατοστεγής τοίχος (διάφραγμα/πασσαλότοιχος), σε βάθος τουλάχιστο 2 μέτρα κάτω από το χαμηλότερο σημείο εκσκαφής. Ο τοίχος θα είναι πακτωμένος εντός του ορίζοντα C.

6. Λαμβάνοντας υπόψη την γεωλογική στρατηγραφία της περιοχής του έργου, τα βάθη του υπόγειου υδροφόρου ορίζοντα, το βάθος της προτεινόμενης εκσκαφής, την κατασκευή περιμετρικού στεγανού διαφραγματικού τοίχου και τα αποτελέσματα των γεωτεχνικών ερευνών και εργαστηριακών δοκιμών, 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 αδιαπέραστων στοιχείων, επειδή το νερό δεν μπορεί να διεισδύσει διαμέσου των τοιχωμάτων.

7. Με την προϋπόθεση ότι ο στεγανός περιμετρικός τοίχος θα κατασκευαστεί σε βάθος τουλάχιστον 7 μέτρα (2 μέτρα κάτω από το χαμηλότερο σημείο εκσκαφής), αναμένεται ότι η εισροή νερού εντός της εκσκαφής θα είναι της τάξης των 0.5 – 0.6 m^3 ανα τετραγωνικό μέτρο εμβαδού κάτοψης, ανα ημέρα. Αυτό μεταφράζεται σε περίπου 5,500 κυβικά μέτρα ανα ημέρα, για ολόκληρη την εκσκαφή ή, εναλλακτικά, σε 2,700 κυβικά μέτρα ανα ημέρα, αν η εκσκαφή εκτελεστεί σε δύο φάσεις, δηλαδή να διαιρεθεί το έργο σε δύο

περίπου ίσες περιοχές (θαλάμους), με την κατασκευή προσωρινού ενδιάμεσου διαφραγματικού ή άλλου στεγανού τοίχου, ο οποίος θα κατεδαφιστεί μετα την συμπλήρωση της πρώτης φάσης.

8. Υπολογίζεται ότι στην περίπτωση κατά την οποία η εκσκαφή θα γίνει σε δύο φάσεις, θα χρειαστούν 6 – 8 γεωτρήσεις άντλησης, σε συνδυασμό με φρεάτια αποστράγγισης (sump wells) μικρού βάθους (3–4 μέτρα), που θα εγκατασταθούν στον χώρο της εκσκαφής.

9. Για το συγκεκριμένο έργο προκρίνεται η λύση της διάθεσης του αντλούμενου νερού σε φρεάτια με βάθος πέραν των 11 μέτρων έτσι ώστε να διοχετευθούν οι ποσότητες του νερού σε κατάλληλο γεωλογικό σχηματισμό (Ορίζοντα C – Αλλουβιακές προσχώσεις), με ικανοποιητική διαπερατότητα. Προκαταρκτικά φαίνεται ότι για να επιτευχθεί το πιο πάνω πρέπει να ανορυχθούν έξι φρεάτια εσωτερικής διαμέτρου 300mm. Το σχήμα πιο κάτω υποδεικνύει τυπική τομή του φρεατίου.

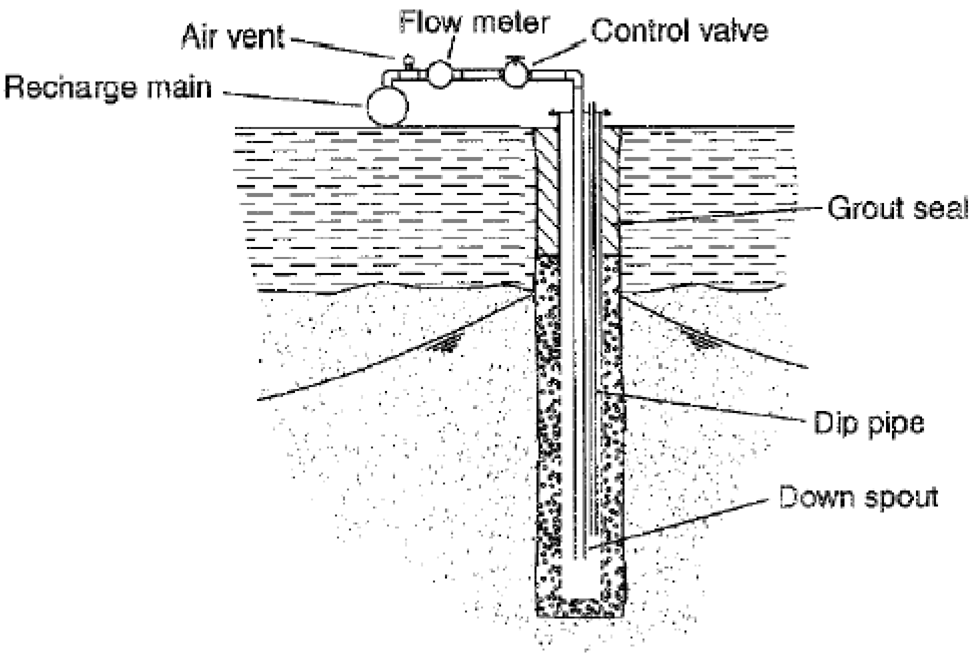

Fig.3 Recharge Well

Φρεάτιο Επαναφόρτισης

10. Υπολογίζεται ότι στην περίπτωση κατά την οποία η εκσκαφή θα γίνει σε δύο φάσεις, θα χρειαστούν 6 – 8 γεωτρήσεις άντλησης, σε συνδυασμό με

φρεάτια αποστράγγισης (sump wells) μικρού βάθους (3–4 μέτρα), που θα εγκατασταθούν στον χώρο της εκσκαφής.

11. Το σύστημα αποστράγγισης θα συνδεθεί με σύστημα φρεατίων επαναφόρτισης (recharge wells) τα οποία θα εγκατασταθούν στον χώρο πρασίνου και στην περιφέρεια της εκσκαφής, για την απόρριψη του νερού άντλησης.

12. Για τον καλύτερο υπολογισμό της ποσότητας νερού που δύναται να επαναφορτιστεί πίσω στον υδροφορέα κάτω από πραγματικές συνθήκες λειτουργίας, έχει διεξαχθεί ολοκληρωμένη δοκιμή επαναεισαγωγής (infiltration test) εντός του οικοπέδου, με την χρήση δύο φρεατίων επαναεισαγωγής.

13. Από την ανάλυση των αποτελεσμάτων της δοκιμής επαναφόρτισης των δύο φρεατίων 1 και 2, εξάγεται το συμπέρασμα ότι ένας μέγιστος ρυθμός απορρόφησης των 20 κυβικών μέτρων ανα ώρα έγινε κατορθωτός, κάτω από ατμοσφαιρικές συνθήκες.

14. Για σκοπούς υπολογισμού θεωρείται ότι για την αποστράγγιση ποσότητας νερού 230 m^3/hour (περίπτωση ολικής εκσκαφής), θα ανορυχθούν περίπου 12 γεωτρήσεις επαναφόρτισης (recharge wells), ή εάν η εκσκαφή γίνει σε δύο φάσεις 7 γεωτρήσεις. 'Stand by' γεωτρήσεις θα πρέπει επίσης να είναι διαθέσιμες, για τις περιπτώσεις που κάποια φρεάτια θα χρειαστούν μερική συντήρηση.

15. **Μείωση της συγκέντρωσης των στερεών στο αντλούμενο νερό.**

Για να μειωθεί η συγκέντρωση των αιωρούμενων στερεών στο αντλούμενο νερό και η πιθανή απόφραξη των φιλτροσωλήνων, προτείνεται η κατασκευή δεξαμενής καθίζησης με διαστάσεις 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.

Μελέτη Εκτίμησης Επιπτώσεων στο Περιβάλλον για την κατασκευή και λειτουργία του Έργου «Renaissance» στο Δήμο Λεμεσού

Dewatering Method Statement

Νοέμβριος 2020

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PRIME PROPERTY GROUP
Renaissance Project

Dewatering Method Statement

CK_N106_20

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Geotechnical & Water Engineers

Contents

Executive Summary

1. Based on the findings of the site investigations which were carried out within the building plot, it is expected that the ground water table will be encountered at depths varying between 3.5 to 5 meters.

2. Purpose of this report is to make an estimate of the dewatering requirements during excavation of the foundations and basement of the proposed building and, taking into consideration the various construction stages, to propose the preferred method of dewatering and method of disposal of pumped water.

3. In accordance with the findings of the geotechnical report (Geoinvest - 2015), most of the inflow will originate from Horizon C (mainly alluvial deposits), which extends down to 11.80 m, and exhibit permeabilities of 1.5 $x10^{-2}$ cm/s or 13 m/day.

4. The total area in plan (building footprint), to be excavated down to foundation level is 10,458 m^2 . A one level basement will be constructed, with an anticipated foundation level at 4.5 to 5 m below existing ground level.

5. It is anticipated that, for the control of ground water seepages into the excavation, a permanent perimetric cut off wall (diaphragm/secant pile), will be constructed to a depth of at least 2 m below the final excavation level. The wall will be keyed into the alluvial deposits of Horizon C.

6. Considering the stratigraphy of the site, the ground water levels established and the depth of the proposed excavation, the perimetric diaphragm wall to be installed and, the test results obtained from the site investigations and laboratory tests, an estimate of the anticipated inflow of water into the excavation during construction has been made using a flow net analysis. Pre-construction dewatering assessment for the aquifers encountered, was also correlated to results obtained during the infiltration tests performed at the site and also, to results obtained from full scale pumping tests performed for other sites in the broader area of the project, within similar geological stratigraphy.

7. Provided that a watertight perimetric wall will be taken down to a depth of about 7m (i.e. 2m below excavation level), it is estimated that the inflow into the excavation will be of the order of 0.5 – 0.6 m^3 per square meter of plan area, per day. This translates into around 5,500 cubic meters per day, for the total excavated area, or alternatively, to 2,700 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 <u>temporary</u> intermediate secant pile or other, cut off walls.

8. It is estimated that, for the case when the excavation will be carried out in two stages, 6 no. wells will be required, in combination with a number of shallow sump wells to be installed within the excavation area.

9. A ground water recharge system shall be used as a means of disposing the ground water abstracted by the dewatering system. A full infiltration test was carried out at the site, involving two recharge wells.

10. From the analysis of the data obtained during the infiltration tests carried out in boreholes 1 and 2, it may be concluded that a maximum absorption rate of 20 m^3/hour was achieved in both boreholes, with no pressure applied.

11. For design purposes it is considered that for an estimated dewatering requirement of 230 m^3/hour (case of total excavation), approximately 12 recharge wells will be necessary, or for the case when the excavation is carried out in two stages 7 recharge wells. Stand by recharge wells shall also be provided (Chapter 4).

1. Hydrogeological Regime

1.1 Introduction

The proposed Development consists of a complex of buildings, with one underground basement for parking. The total depth of excavation will be approximately 5m - including 1.5 thick foundation rafts throughout. In addition, the lift wells will be founded below the general excavation level.

The building plot is situated between Androutsou, Karaiskaki, Poumpoulinas and Ayiou Andreou streets, about 600m west of Ayios Nikolaos round about and, covers an area of approximately 12,800 m^2 which is generally flat, gently dipping towards the sea (south). At present, there are no existing buildings within the site. An approximate area of 2,355 m^2 will be allocated as public green.

The total area in plan (building footprint, Figure 1), to be excavated down to foundation level is 10,458 m^2. The Development will consist of 8 and 5 storeys buildings. Prior to the excavation, it is anticipated that a peripheral diaphragm or secant pile wall will be constructed, which will form a positive impermeable cut-off barrier.

Figure 1- Basement Plan (Οικονομίδης -Ακκελίδου Architects)

1.2 Hydrogeological Conditions

The hydrogeological conditions prevailing at the building site have been preliminarily assessed, based on the results of twelve (12) investigation boreholes, drilled within the area of the building plot (Geoinvest, December 2014 - Jannuary 2015). Results so obtained, were also correlated to existing information obtained from desk study sources, i.e. available geological/geotechnical information from previous construction, geotechnical and hydrogeological work carried out in the broader area of the proposed Project.

The investigation boreholes were taken down to a maximum depth of 15m, with associated disturbed /undisturbed sampling and, in situ and laboratory testing.

In view of the anticipated relatively shallow excavation levels (single basement), it is strongly recommended that prior to the commencement of the construction works, trial pits are excavated down to depths of 5m, in order to further assess the in situ permeability and infiltration/recharge capacity of the underlying strata. The trial pits will facilitate a more accurate estimate of the hydrogeological conditions prevailing at excavation level.

Pre-construction dewatering assessment for the aquifers encountered, was also correlated to results obtained during the full infiltration tests performed at the site, as well as from full scale pumping tests performed for other sites in the broader area of the project, having similar geological stratigraphy.

The ground hydrogeological 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.

From available information derived from the site investigation work performed within the building site (Geoinvest), and the Infiltration Tests performed by Themeliotechniki during October 2020, the following hydrogeological data may be deduced for the different soil groups:

- Ground water was encountered in all boreholes at depths varying between 3.5 and 5.5m below existing ground level. This was further substantiated during the performance of the infiltration test – Chapter 4.
- Static water level recorded during the investigation campaign, was of the order of 2.5m at the southern part and 3m at the northern part of the plot.

Nevertheless, these recorded water levels, are expected to be the result of the recharge from storm drains and, seasonal variation of these water levels should be expected (Geoinvest, Dec.2014 - Jan. 2015).

- The permanent water table is found at deeper sections, most probably at depths of the order of 6 to 10.5m.
- More water is expected at deeper depths (›20-25m), below ground level.
- The aquifers at the site are both (i) phreatic (horizons C and D) and, (ii) under pressure, at depths greater than 20–25 meters in coarse materials found below horizon D.

1.3 Permeabilities

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

- Horizon B–Sandy Clayey Silt: $1.1x10^{-4}$ cm/s or 0.09 m/day
- Group C – Silty, Sand and fine Gravel: 1.5 $x10^{-2}$ cm/s or 13 m/day
- Group D – Sandy, very Clayey Silt: $9.5x10^{-6}$ cm/s or 0.008 m/day
- Group D – Clayey, very sandy Silt:$2.4x10^{-4}$ cm/s or 0.21 m/day
- The permeability of the coarse material (Gravel, Cobbles) of horizons C and D are estimated to be at least 25 m/day, depending on the content of the fine material.
- Based on the above results, it may be deduced that the permeabilities of the lower part of GroupC and the coarse part of Group D is high, whereas that of Groups B and D are low. In the sand rich sections of Group D is estimated to be moderate.

Group	Soil Description	Depth Range(m)
A	**Fill**, consisting of a variety of materials of, mostly, fine grained size, i.e. clay, silt and sand, with variable amount of gravel.	Variable thickness 1.6 - 3
B	**Recent Alluvial Deposits** found below the fill and down to 2.80 – 3.90 m. Consisting of: Modern Alluvial, mostly Sandy, Clayey Silts accumulations. Their thickness varies between 0.25 to 1.70 m.	Variable thickness 0.25 to 1.70

C	**Old Beach Deposits and old coarse Alluvial Deposits** Representing mainly alluvial deposits reworked by the sea, and shallow marine deposits. Mixed with organic material. They exhibit an immature stratification with abrupt facies changes in both vertical and horizontal sense. The predominant fraction is Sand and Gravel. The sand predominates at the uppermost part of the horizon and the gravel in the lower part. The thickness of this horizon varies between 6m and 9m.	Variable thickness 1.85/3.80 - 9.90/11.80
D	**Fine Old Sedimentary Deposits** They are represented by fine sedimentary deposits. Distinguished soil types are: ➢ Sandy clayey silt to sandy clay and silt ➢ Clayey, sandy silt to clayey sand and silt ➢ Silty sand to silt and sand The thickness of this horizon varies between 3.20m and 5m.	9.90 /11.80m extending to depths greater than 15m

Table 1 - Soil description with Depth – Boreholes 1-12

2. Inflow of water into the excavations during construction

It is proposed that for the control of ground water seepages into the excavation, a permanent cut off wall (diaphragm/secant pile) will be constructed to a depth of at least 2 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.

Static water levels recorded during the investigation campaign, were of the order of 2.5m at the southern part and 3m at the northern part of the plot. These recorded water levels are expected to be the result of the recharge from storm drains and, seasonal variation of these water levels should be expected (Geoinvest, Dec.2014-Jan. 2015). On the other hand, the permanent water table is found at deeper sections, most probably at depths of the order of 6 to 10.5m**,** i.e. below the anticipated foundation excavation level. Therefore, in the case of a **one level basement**, the problems of dewatering are expected to be manageable.

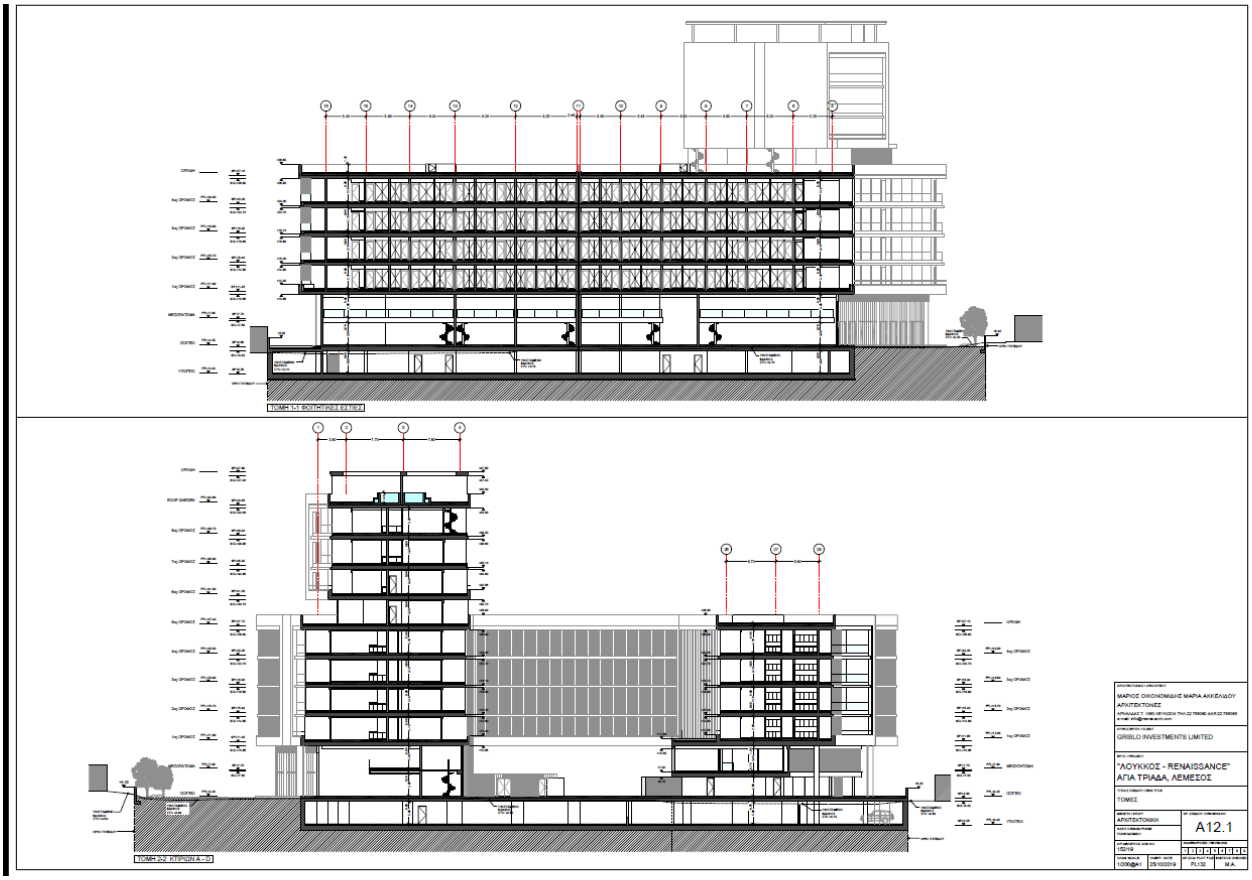


Figure 2 - Building Cross Sections (Οικονομίδης-Ακκελίδου Architects)

From the available geotechnical information (sources mentioned above), it is expected that the strata at the anticipated excavation level will be represented by the upper, finer part, of geological horizon C (sand predominates) which exhibits, moderate to high permeabilities. Actual, mass in situ permeabilities were also deduced from the results of the pumping/Infiltration Tests carried out within the site. Due to the anticipated relatively shallow excavations, valuable information may also be obtained from trial trenches, excavated down to 4.5 – 5m depths.

Design parameters, considered for permeability, storativity and transmissivity for draw down calculations, were also, taken from pumping tests performed in similar geological horizons and dewatering operations carried out during construction works for other sites in the broader area.

It should be also mentioned that, the large number of boreholes drilled over the whole area, may have resulted to limited interconnection and unification of water bearing geological horizons. Therefore, it is possible that, the whole area could behave in some instances, 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, in the case that ground water exists.

Of great importance of course, are the actual permeabilities of the stratum into which the diaphragm wall will be keyed-in, below the base of the excavation.

Considering the stratigraphy of the site, the ground water levels established, the depth of the proposed excavation, the perimetric diaphragm wall to be installed and, the test results obtained from the site investigations and laboratory tests, 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, which is horizon C.

The site is characterized by one phreatic water level found at depths 2.5 to 3m below existing ground surface. Nevertheless, as previously stated, these recorded water levels, are expected to be the result of the recharge from storm drains and, seasonal variation of water levels should be expected (Geoinvest, Dec.2014-Jan. 2015). The permanent water table is found at deeper sections, most probably at depths of the order of 6 to 10.5 m and, therefore, below the proposed excavation level. Pore water pressures in all horizons B, C and D are thus hydrostatic, and during excavation any pressure head difference will produce flows through the upper part of horizon C. This is caused by the total head difference between the two faces of the diaphragm wall and the water bearing part of horizon C at the bottom of the excavation.

The ground water level will have to be lowered at least 1m below excavation bottom. Considering the small excavation depth, and the static water level regime, the seepages from behind the diaphragm wall will thus be relatively small and most inflows will be due to upward vertical flows originating from horizon C.

For the calculation of total seepages it was assumed that seepages in to the excavation will come partly from behind the wall and partly directly from horizon C.

A flow net diagram has been constructed for this case assuming a horizon C of infinite depth. 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

and, *Nd is the number of head drops.*

Provided that a watertight perimetric wall will be taken down to a depth of about 7m (i.e. 2m below excavation level), it has been estimated that the inflow into the excavation will be of the order of 0.5 – 0.6 m^3 per square meter of plan area, per day. This translates into around 5,500 cubic meters per day, for the whole excavated area, or alternatively, to 2,700 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 temporary intermediate secant pile or other, cut off walls. The temporary walls will be demolished as excavation progresses for the next compartment.

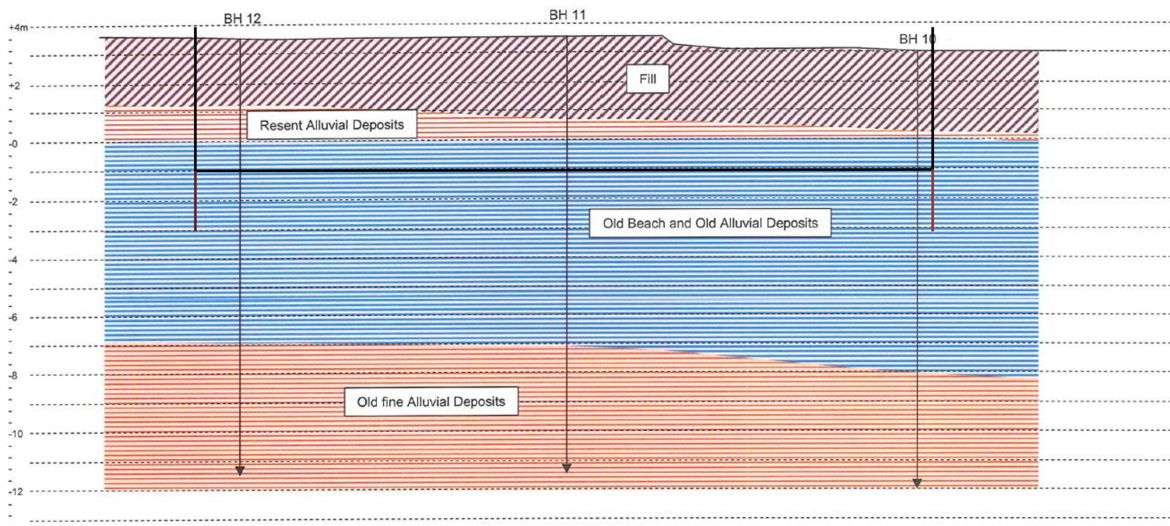


Figure 3 - Geological Cross Section 4-4 (Geoinvest Dec.2014-Jan.2015), with Excavation and cut off

2.1 Ground Water Control

There are several techniques or methods available for controlling ground water flow into excavations at a construction site. The selection of a technique or techniques appropriate to a particular project at a particular site 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 recommended to proceed with the excavation of the whole basement area <u>in two stages</u>, as previously mentioned. In this case pumping of about 115 m^3 /hour will be necessary and it is estimated that, for this case 6 no. wells will be required, in combination with a number of shallow sump wells.

Alternatively, if the whole building footprint area is excavated at once, then a total pumping of 230 m^3/hour will be necessary.

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, also considering the <u>relatively shallow</u> excavation depth.

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.

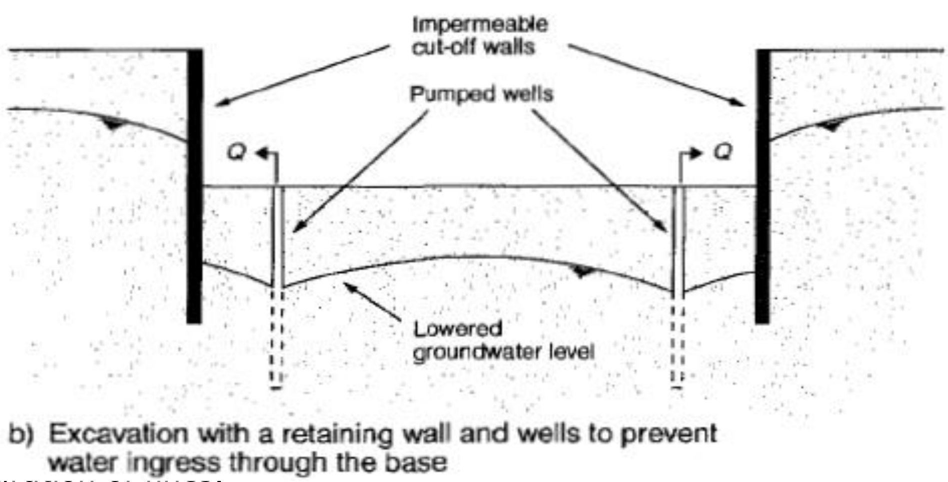


Figure 4 – Pump-well dewatering system

In the current case, most of the inflow will originate from Horizon C and not from behind the cut-off wall. It is, therefore, necessary to cover the excavated area with a number of drainage wells, which will have to be drilled down to relatively shallow depths, about 6m below excavation level. In combination with the deep wells, surface sump wells should also be installed, operating in shallow pits. As stated above, it is estimated that, for the case when the excavation will be carried out in two stages, 6 no. wells will be required, in combination with a number of shallow sump wells.

The boreholes need to be 800-900 mm in diameter with a 300 mm perforated steel pipe installed at the centre. The pipe must be wrapped in geotextile 200 grams/ m^2, to minimize the amount of silt and clay in the pumped water. The space between the pipe and the external wall of the borehole must be filled with a mixture (50-50) of single size 10 and 20 mm aggregates.

It is standard practice to adjust the design of the de-watering system to suit the equipment available and techniques preferable to the contractor, so its final nature can be adjusted accordingly, after re-evaluation of all factors considered.

Lifts' Wells

For the excavation of the Lifts' wells which extend 1 m 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), storm water systems or, return pumped water back to the aquifer through recharge wells.

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

In the case of the Renaissance Project, the disposal of the pumped water to the sea is considered impractical due to the logistics necessary to implement it and, in view of its characteristics, this project must be considered in isolation.

For this reason, 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. Due to the relatively shallow excavation depth and depth of permeable horizon C, a large diameter well recharge system has been examined.

Specific factors that have been examined 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 ground water 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.

The proposed use of a recharge system in order to return pumped water back to the aquifer 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.

2.2.3 Dewatering Discharge Quality – Suspended Solids

Dewatering permits require that dewatering water pass through a sediment removal device such as a settling tank, prior to discharge, with total suspended solids (TSS) in the discharge leaving the site not exceeding 80 g/m3 (for recharge purposes). 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).

For recharging water back to the aquifer, the main consideration is the possible clogging of infiltration trenches and boreholes.

Based on particle size and density of silt particles, it is estimated that a settling tank having dimensions 10m length by 3m wide by 3m height will be required for the efficient removal of fines. Settling times are expected to be in the region of 1 hour.

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.

3. Ground Water Recharge System

The recharge system shall be used as a means of disposing the ground water abstracted by the dewatering system. Such a scheme requires caution and careful planning. Due to the relatively shallow excavation depth and depth of permeable horizon C, in this case it is advantageous to use a large diameter well recharge system.

As recharge wells are prone to clocking by even small quantities of suspended solids, it is important that settling tanks are used. As a rule of thumb, for each abstraction well two recharge wells may be required when abstracting and recharging into the same aquifer. This is to allow for sufficient capacity and for a number of the wells to be out of commission being rehabilitated.

The operation of any form of a recharge system will require a recharge consent from the environmental authorities. This provision applies even if the ground water is being abstracted and returned to the same aquifer in the same site.

3.1 Recharge Trenches

Recharge trenches may also be considered if excavated to penetrate through the superficial low permeability deposits, which in this case is the fill, horizon A. Therefore, will have to be excavated down to the Recent Alluvial Deposits (Horizon C), about 5 to 6 m deep, where deposits of higher permeability have been encountered and also, so that the return of recharge water back to the main excavation is avoided, as much as possible.

3.2 Recharge Wells

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 and the danger of the return of recharge water back to the main excavation is avoided. For the specific site under consideration, the wells can be taken down to a depth of 10 meters, in horizon C, where the predominant fraction is Gravel, with estimated permeabilities (based on laboratory tests) of the order of 1.5 x10^{-2} cm/s or 13 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 as previously discussed.

If the standing ground water level is found at a higher elevation the recharge wells may require a grout seal in order to prevent water from travelling up the filter pack to ground level. In such a case, the well head can be sealed, and the recharge pipework pressurized slightly, so that the feed head is 2 to 3 m above ground level. In order to avoid over pressurizing the well a standpipe may be installed which can overflow if the pressure is exceeded.

<u>In order to further asses the infiltration potential of recharge wells a full field infiltration test was carried during period 27/10/2020 – 02/11/2020. The testing procedure adopted and the results obtained are detailed in paragraph 4, below</u>.

4. Recharge Wells for the in situ determination of Field Infiltration Rate

4.1 General

Two no. (2) Recharge Test Wells were drilled at an approximate distance of 20 m between them, 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.

For the specific site under consideration, the wells were taken down to geological horizon C (Old Beach Deposits and Old Coarse Alluvial Deposits - Site Investigation report by Geoinvest, January 2015), where the predominant fraction is Gravel, with estimated permeabilities (based on laboratory tests) of the order of 1.5 $x10^{-2}$ cm/s or 13 m/day.

Perforated, slotted or screened pipe was used in that section where injection was intended. The hole aperture in the pipe is smaller than any aggregate placed between the liner and the borehole wall. The perforated pipe commenced below the overlying unsuitable strata (Fill).

Plate 1 - Location of Recharge Boreholes BH1 and BH2

4.2 Installation of Recharge Wells

The wells were installed as shown schematically below and have the following technical characteristics:

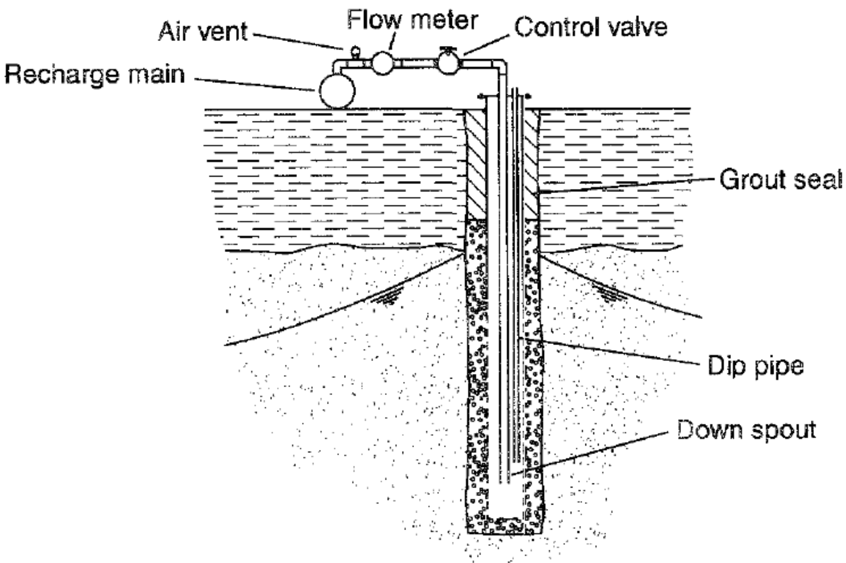


Figure 5 – Schematic presentation of Recharge Well

1. Borehole 1: Total drilled depth 11.30 m. The top 5.30 m of the 900 mm outside diameter UPVC casing pipe is "blind" and, the bottom 6 m perforated. The pipe extends about 0.70 m above existing ground level. Initial depth of ground water (measured from the top of the pipe) 3.70 m. The top 5 m of the pipe was externally grouted with cement grout in order to create an effective seal. After installation of the lining tube, rounded 5mm to 10mm pea gravel was placed in the annulus between the borehole wall and the lining tube.

2. Borehole 2: Total drilled depth 11.00 m. The top 5.00 m of the 900 mm outside diameter UPVC casing pipe is "blind" and, the bottom 6 m perforated. The pipe extends about 1 m above existing ground level. Initial depth of ground water (measured from the top of the pipe) 3.85 m. The top 5 m of the pipe was externally grouted with cement grout in order to create an effective seal. After installation of the lining tube, rounded 5mm to 10mm pea gravel was placed in the annulus between the borehole wall and the lining tube.

4.3 Recharge Test Procedure

The recharge test procedure involved the following steps:

1. For testing and in order to prevent clogging, clean water from a 25 cubic meters capacity water bowser was injected into Borehole 1. The pipeline system was equipped with a flow meter and return pipe in order to better control the water flow. A manometer was installed at the top of the borehole, for measuring the water pressure in borehole 1. Simultaneously, the water table in borehole 2 was recorded. Water depth in borehole 2 measured from top of installed pipe (casing). Initial water table in borehole 1 before test

commenced = 3.70m. Water level in borehole 1 after completion of test = 3.40m.

2. Pumping water from Borehole 1 → injecting into Borehole 2. In order to economize the use of clean water and be able to assess the infiltration potential of the geological stratum under working conditions, borehole 2 was tested with water pumped from borehole 1. For this purpose a pump was installed in borehole 1 at 10 m depth and connected with a pipeline to borehole 2. A water flow meter was installed. Simultaneously, the water table in production borehole 1 was recorded.

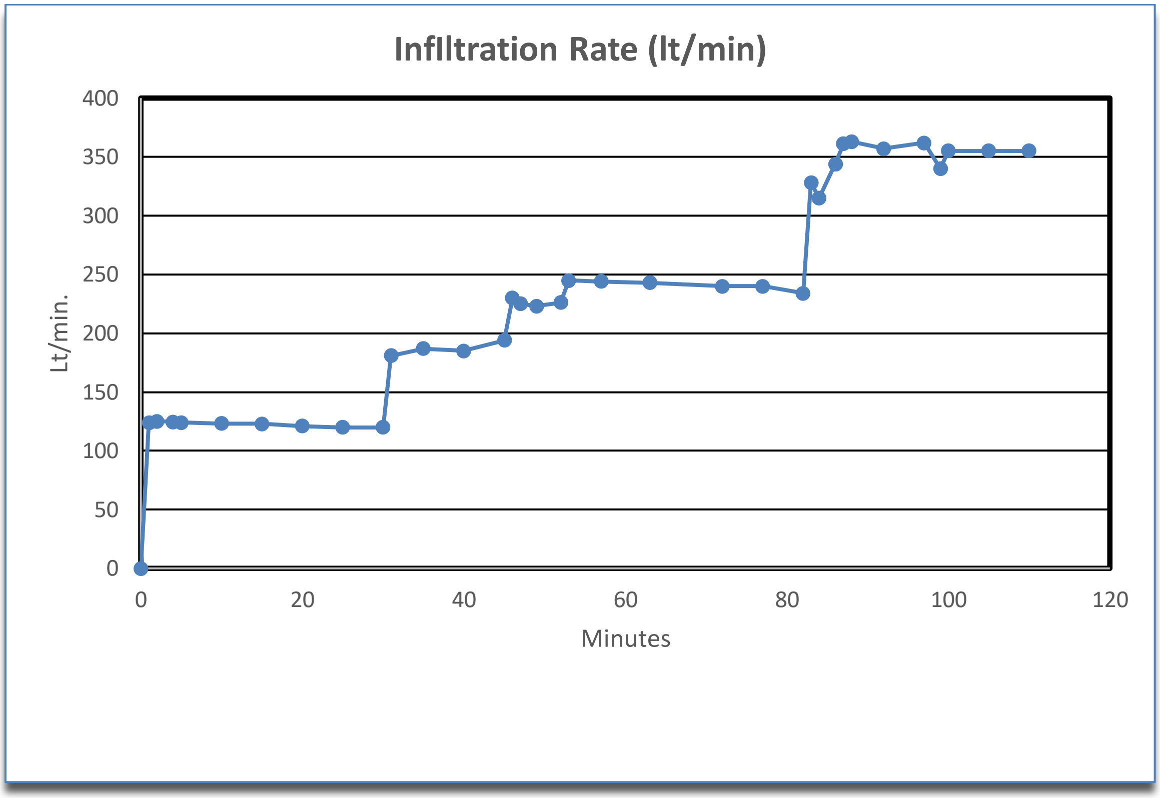
4.4 Field Measurements

Actual field measurements recorded during the in situ tests carried out at the site of the proposed Renaissance Project are given below, in tabular and graphical form.

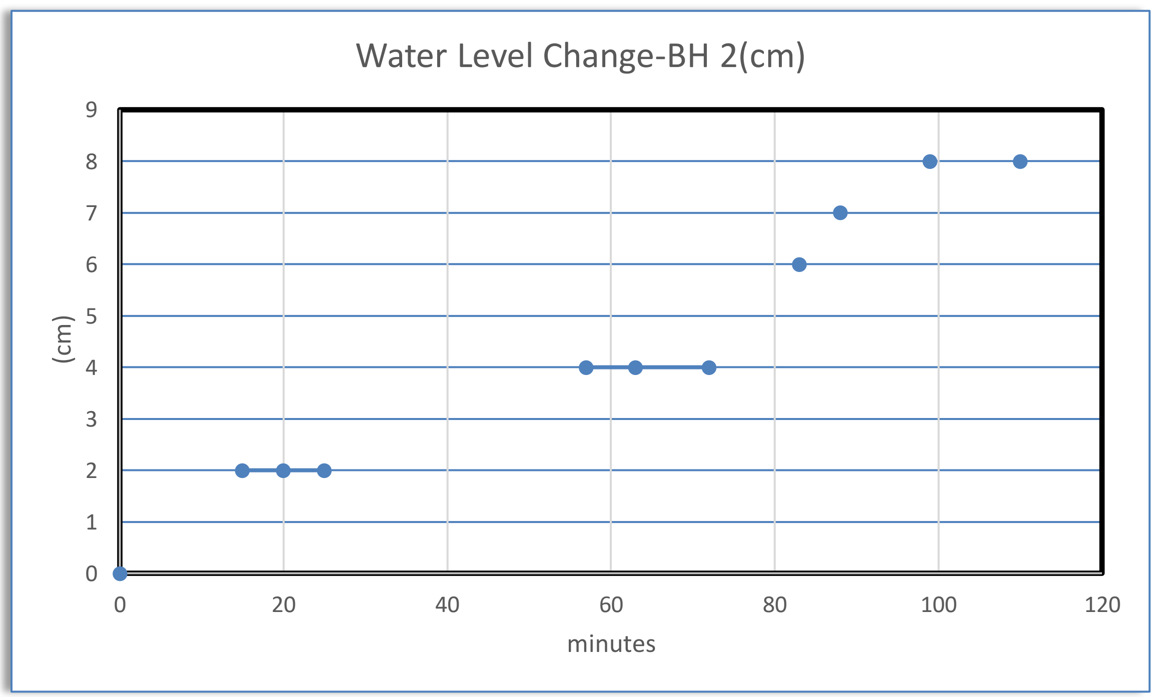
It is important to note that, although a manometer was installed at the top of the borehole for measuring pressure, the tests were performed without the application of pressure, at all stages.

Time (min.)	Average Supply (Lt/min.)	Pressure at Borehole Head (bar)	Water Level in BH2 (m)	Water level change in BH2 (cm)
0	0	0	3.85	0
1	124	0		
2	125	0		
4	124.5	0		
5	124	0		
10	123.2	0		
15	123	0	3.83	2
20	121	0	3.83	2
25	120	0	3.83	2
30	120	0		
31	181	0		
35	187	0		
40	185	0		
45	194	0		
46	230	0		
47	225	0		
49	223	0		
52	226	0		
53	245	0		
57	244	0	3.81	4
63	243	0	3.8	4
72	240	0	3.8	4
77	240	0		
82	234	0		
83	328	0	3.79	6
84	315	0		
86	344	0		
87	361	0		
88	363	0	3.78	7
92	357	0		
97	362	0		
99	340	0	3.77	8
100	355	0		
105	355	0		
110	355	0	3.77	8

Table 2 – Infiltration Test Borehole 1



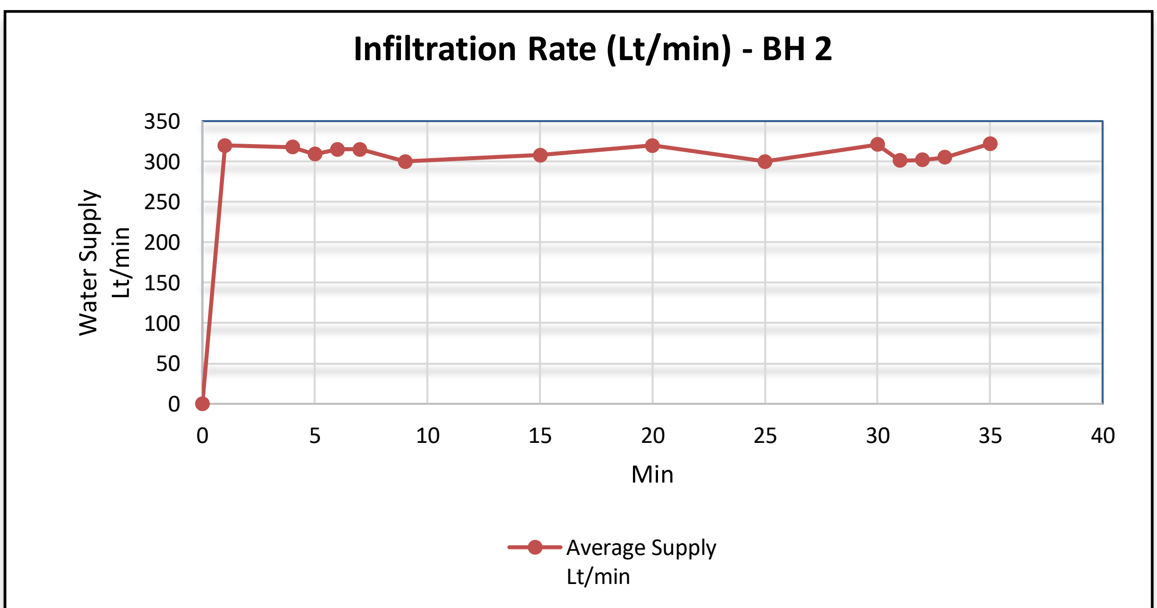
Graph1 – Infiltration Rate Borehole 1



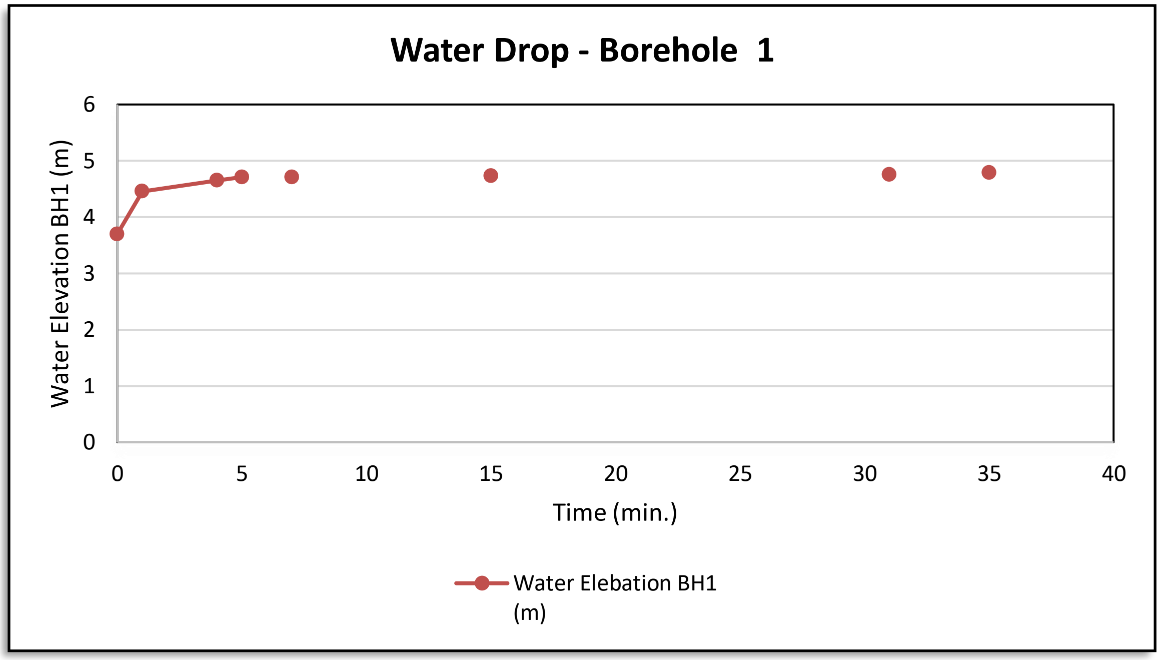
Graph 2 – Water Drop Borehole 2

Time (min.)	Average Supply Lt/min	Water Elevation BH1 (m)
0	0	3.7
1	320	4.46
4	318	4.65
5	309	4.71
6	315	
7	315	4.71
9	300	
15	308	4.73
20	320	
25	300	
30	321	
31	301	4.76
32	302	
33	305	
35	322	4.79

Table 3 – Infiltration Test Borehole 2



Graph 3 – Infiltration Rate Borehole 2



Graph 4 – Water Drop Borehole 1

4.5 Analysis of results

From the analysis of the data obtained during the infiltration tests carried out in boreholes 1 and 2 it may be 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 230 m^3/hour (case of total excavation), approximately 7 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.

Recharge wells may be located in the proposed public green area of the building site.

4.6 Design of Recharge Wells

A 900 mm diameter boreholes will be drilled down to a minimum depth of 11 m. 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.

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. The perforated pipe shall commence 2 m below the overlying unsuitable strata (Fill).

After installation of the lining tube, rounded 5 mm to 10 mm 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 meter above and below the interface with the overlying unsuitable material (Fill). Care should be taken to prevent contamination of the gravel.

The well will be equipped with a down pipe or 'falling main', to feed the water down into the well, which prevents the recharge water from cascading into the well and becoming aerated (aeration of the water may promote biofouling and other clogging processes and should be avoided). Air vents shall be provided at the top of the well and pipework to purge air from the system when recharging commences, and to prevent airlocks in the system.

Only clean water will be used for testing purposes in order to prevent clogging. If required, water will pass through a sediment removal device as a settling tank, prior to injection, in order that water used will have total suspended solids (TSS) not exceeding 30 g/m^3.

Appendix 1
Infiltration Test - Photographs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ΝΕΡΟ ΔΙΑ ΠΙΣΙΝΕΣ & ΕΠΙΧΩΜΑΤΩΣΕΙΣ