Paralimni

MARINA

Hydrogeological Conditions and Dewatering Methods

NOVEMBER, 2019

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Applied Geology – Geotechnics – Materials Testing

Environmental Engineering

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27/11/2019
X\Geotech19/Pernera-Marina

Messrs PMV MARITIME HOLDINGS LTD,
Att.: Mr Anthoulis Kountouris,
Paralimni.

Dear sirs,

RE: PRELIMINARY ASSESSMENT OF HYDROGEOLOGICAL CONDITIONS AND POSSIBLE DEWATERING METHODS OF THE EXCAVATION AT PARALIMNI MARINA

We are glad to advise you that the above study is completed and three hard copies are forwarded.

The results are based mainly on desk study, long year experiences in geotechnical investigations and a lot of information obtained from similar studies in similar geological/hydrogeological conditions.

We remain at your disposal for any clarifications or further information on this subject.

Sincerely yours,

Andreas Shathas
(Geologist - Managing Director)

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1 Introduction

PMV Maritime Holdings Ltd are planning to construct a marina with the necessary breakwater, two ten storey towers, villas and supporting facilities at the site of the existing fishing port in Louma – Pernera area of Paralimni. The project site is located on the south-eastern coast of Cyprus, just north of Golden Coast Hotel and covers an area of approximately 0.12 km^2.The location of the project is shown on figure 1 below.

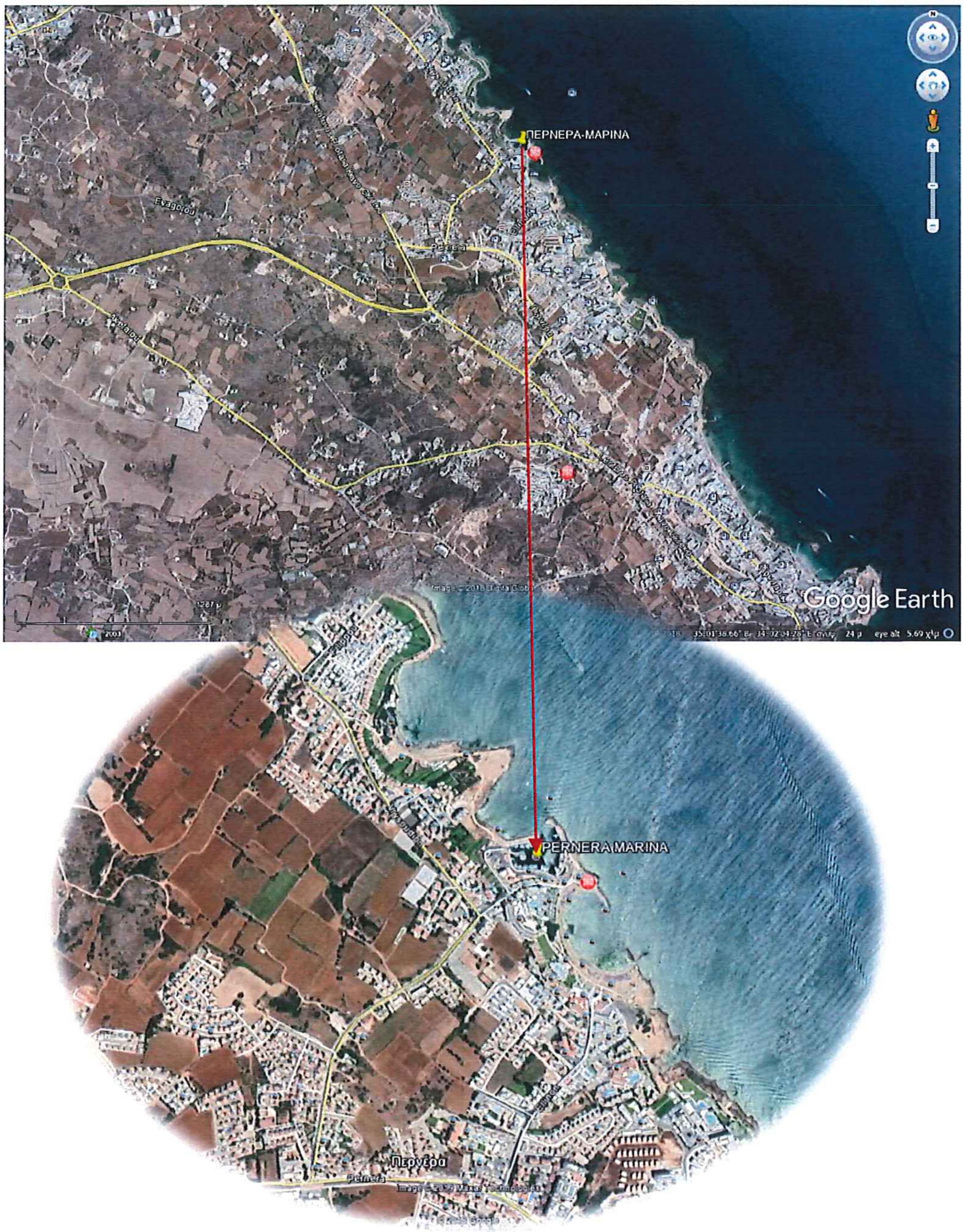


Figure 1: General Location of Project area

The two 10-storey buildings (at the Northwestern and Southwestern part of the Marina) will be built with car parking areas. Several small single storey structures and parking areas will also be constructed within the Marina area, as well as fuel storage area to the southeast corner. The total buildings area in plan (building footprint, Figures 2 and 3), is approximately 8,800 m^2. Part of the buildings area is to be excavated (inland) and part backfilled (offshore), as shown on above figures and on cross section, figure 6.

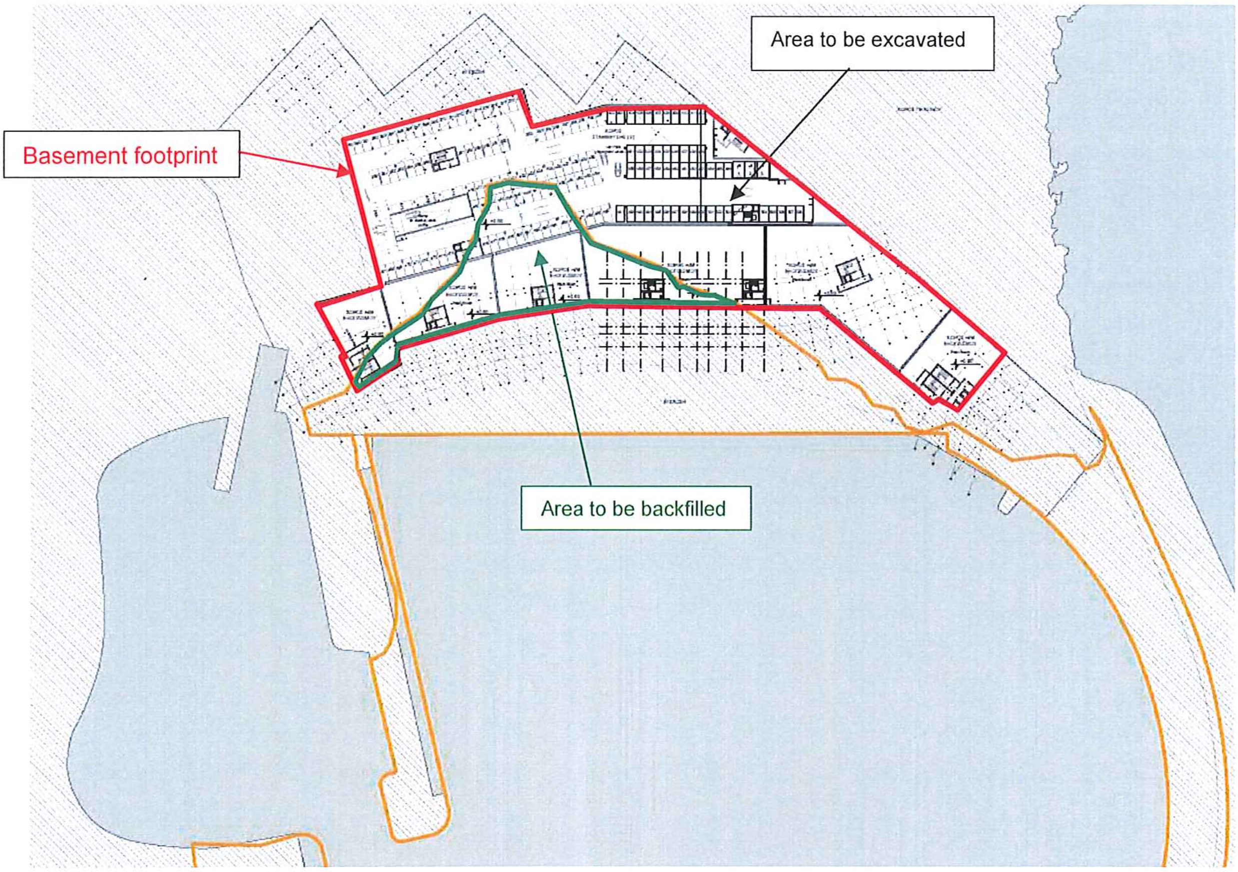


Figure 2: Basement Plan (Kythreotis Architects)

The Ground floor elevation of the North and South buildings are approximately 2.00-3.00m and 1.50-2.50m above mean sea level respectively. Since a basement is envisaged, the final upper level of buildings foundation is planned to be at +0.6m as shown on cross section of figure 6. The maximum depth of excavations, so that the foundation can be constructed, will reach -2m (2m below mean sea level). Reduction of the water level down to at least -2.50m, therefore, should be achieved, so that the base of the excavation is kept dry and the construction works can be performed. It is anticipated that a peripheral diaphragm or secant pile wall will be constructed upon completion of excavations and reclamation works, which will form a positive impermeable cut-off barrier, as shown on cross section of figure 6.

Figure 3: Basement Plan (in red)

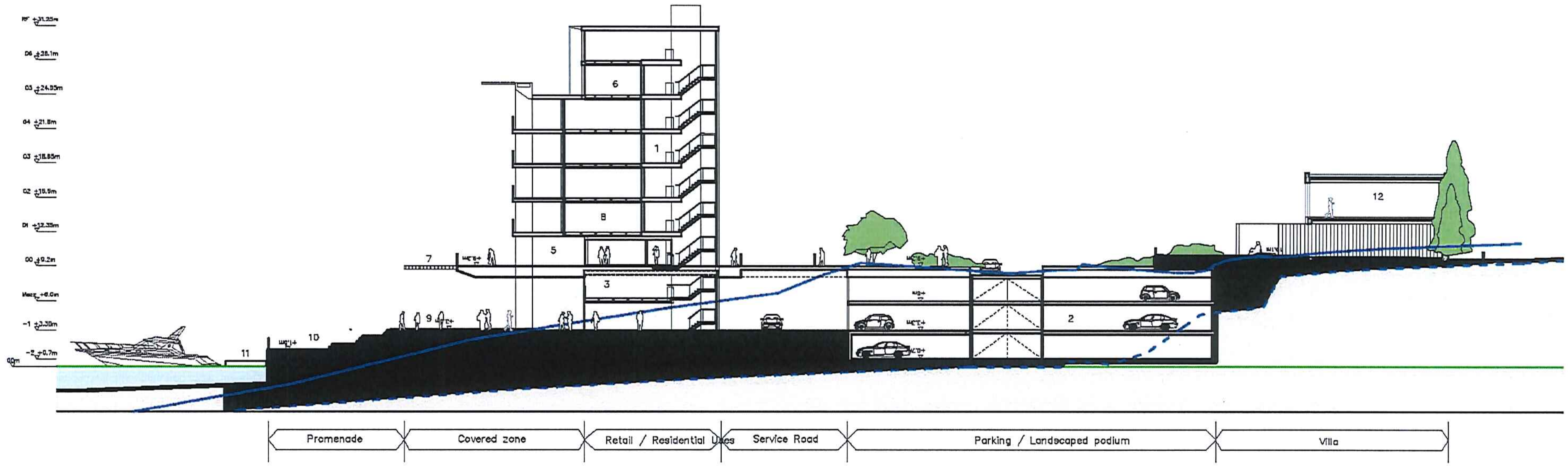


Figure 4: Cross section of the buildings area (W-E)

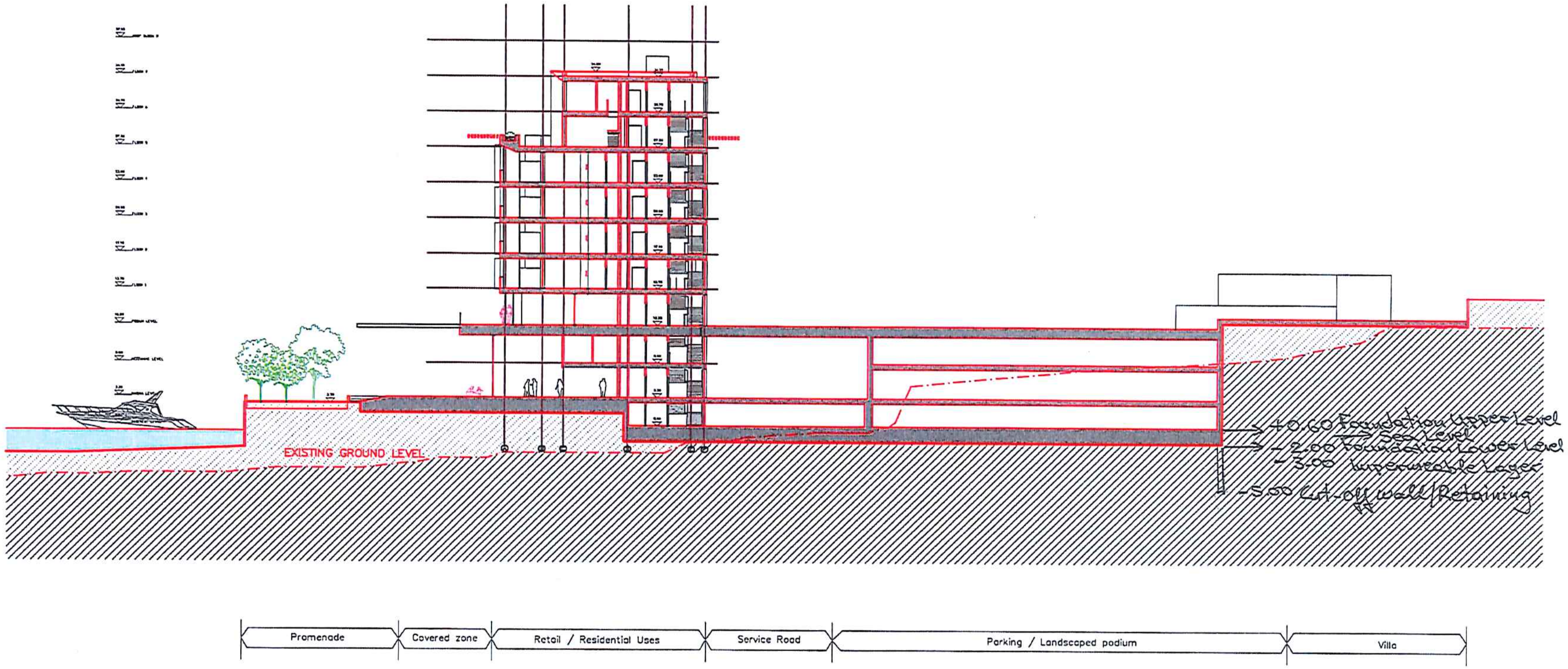


Figure 5: Cross section of the buildings area (N-S)

CROSS SECTION 1– 1'

Scale: Approximate

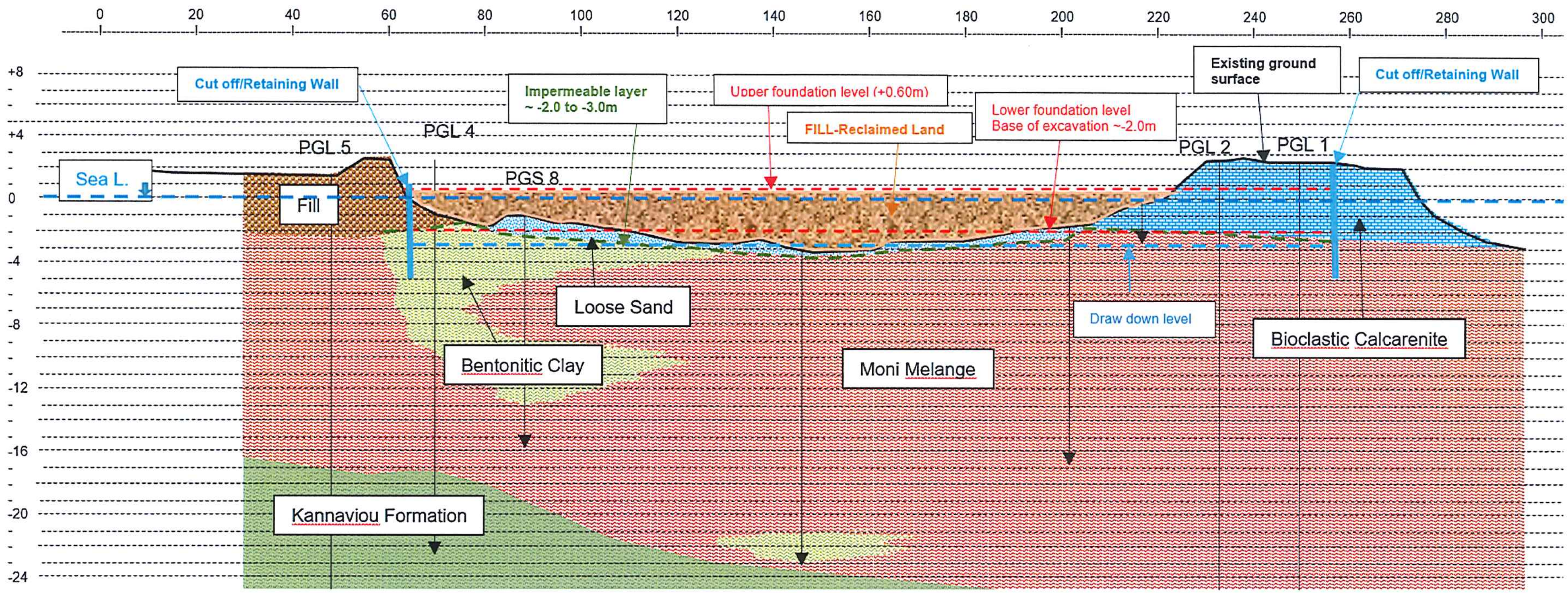


Figure 6: ***Geological Gross Section (N/S direction) showing details of foundation, excavation levels, required drawdown and cut off walls***

2 General Geological Setting

The broader area is covered by various types of soils and rocks and almost the whole of the circum-Troodos sedimentary cover can be found all over the broader area. At the particular site under study, however, the younger marine sedimentary formations form a thin cover of up to 5.00m over the much older geological Formations. Starting from the younger to the older the following geological formations were encountered during the field investigation works carried out in 2018.

The Athalassa member of Nicosia Formation of Pliocene and Pleistocene age (about 2 million years old), represented by calcarenites, calcareous silts and sands, which are capped inland by a <1 m thick top soil and offshore by few cm to 4 meters thick layer of recent, marine, superficial, sandy deposits. They have a maximum thickness of the order of 4.0-4.50 meters in the study area and were encountered both onshore and offshore, but not all over the whole of the area. They are of variable cementation, usually highly porous and fragmented due to relatively close spacing of the horizontal discontinuities and cracking along various directions and in places slightly karstified. The cementation is usually weak, occasionally moderate and in places very poor, turning thus to grain supported silty sands. Strong cementation was observed randomly.

The Moni Melange Formation, of upper Campanian to Maestricthian age, which consists of reddish brown and purple silt and clays, acting in most of the area as matrix to variably sized and shaped fragments/clasts of mudstone and siltstone, or even igneous rocks (serpentinite and lavas mostly) generated from the erosion of Mamonia Complex rocks.

They are of marginally high to high plasticity and highly fissured.

The Kannaviou formation of Upper Campanian to Maestricthian age (about 70-75 million years old), which is represented by dark greyish and bluish green to greenish and bluish grey bentonitic Clays incorporating radiolarian mudstones and volcaniclastic siltstones and sandstones, randomly other rock blocks like quartzitic sandstones, lavas and serpentinites common in southeastern Cyprus. At the particular site, they are found within the Moni Formation as irregularly distributed layers or patches, indicating, most probably, a combination of valley filling and slope deposition and that after and/or during the deposition of the Melange on Kannaviou, sliding and creeping were taking place and the characteristic syngenetic current texture features were developed.

Variably thick, a few cm to about 3-4 m, recent, superficial sea bed, sandy deposits can be observed offshore on top of calcarenites or directly on top of Moni Melange. The thickness of this unit is of importance, since it provides very easy dredging conditions as well as a highly permeable layer.

From the geotechnical point of view, the above formations can be grouped into 4 geotechnical units as shown on the geological cross section of figure 6.

A. The Superficial Sands
B. The Calcarenites
C. The Silt and Clays of Moni Melange and
D. The bentonitic Clays of Kannaviou formation.

3 Hydrogeological Conditions

The hydrogeological conditions prevailing at the building site have been assessed and presented in Geoinvest, August - November 2018 report, on the basis of the results of 23 investigation boreholes, drilled within or close to the area of the buildings down to a maximum depth of 35 m, with associated disturbed /undisturbed sampling and, in situ and laboratory testing. More information obtained also from desk study sources, based on existing information obtained from previous available geological/geotechnical investigations and construction sites in the broader area and correlated with the information obtained from the particular site.

From the hydrogeological model established during the geotechnical investigations carried out by Geoinvest, 2018, the following hydrogeological data may be deduced:

- Ground water was encountered in all onshore boreholes at variable depths, depending on the ground elevation.
- Static water, level recorded during the investigation campaign, coincides with the sea level and is expected to be subjected to seasonal and daily variations depending on the tide level (± 0.30cm).
- The aquifer at the site is phreatic and limited within the calcarenites.
- No groundwater was encountered at deeper sections.
- The calcarenites are of high permeability and a good and high yielding aquifer could be developed if they were thicker and if recharged adequately. This is not happening and not much groundwater is moving along the interface of the calcarenites and the underlying clays.

In summary, there is a small, phreatic aquifer, the base of which follows the interface between the calcarenites and the Moni Melange. Very small amounts of ground water were observed during drilling. The static water level was recorded during the drilling campaign and found to be at the same level as the mean sea level. Quite natural, since the boreholes were drilled only a few meters from the shore line. This water is in fact sea water mixed with brackish water found within the calcarenites. The amounts of the groundwater depend on the precipitation mostly, but possible leakages from water supply and sewage pipe lines could also contribute to the ground water recharge.

3.1 Permeability

No in situ permeability tests (Lefranc, Magg or constant head) were performed during the 2018 ground investigations. In the frame of this study, three laboratory falling head permeability tests, in accordance with BS5930, were performed on selected samples from calcarenite and Moni Clay with the following results:

- Calcarenite: 1.2×10^{-3} to 2.5×10^{-3} or 1.04 to 2.16 m/day. It should be noted, however, that in situ permeability should be quite higher, due to the presence of cracks and other discontinuities of the rock mass. For calculations, permeability of the order of 5m/day was used. The permeability of the compacted fill could be considered to be of the same order.
- Moni Clay: 1.57×10^{-7} cm/s, or 0.0135 cm/day or 1.36×10^{-4} m/day, practically impermeable.

Table 1. Permeability, K, of the soils at the site, used for calculations

Group	Permeability, K cm/s (m/day)	Depth Range(m)
Fill and Calcarenite	5.78 $x10^{-3}$ cm/s, or 5.78 $x10^{-5}$ m/s, or (5m/day)	From +0.60 to -3.0m
Moni Melange	$1.57x10^{-7}$ cm/s or $1.57x10^{-9}$ m/s, or ($1.36x10^{-4}$ m/day)	from ~ -3m >30m

4 Inflow of water into the excavations during construction

The ground water level will have to be reduced at least 0.50m below excavation bottom, i.e. down to -2.5 m. To control and prevent or minimize the ground water seepages into the excavation, a permanent cut off wall acting as a retaining wall as well (diaphragm or secant piles) has to be constructed to a depth of at least -5m. The cut-off wall 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 low due to the very low permeability of the underground layer, i.e. $*10^{-7}$ cm/s (silt and clay, bentonitic clay).

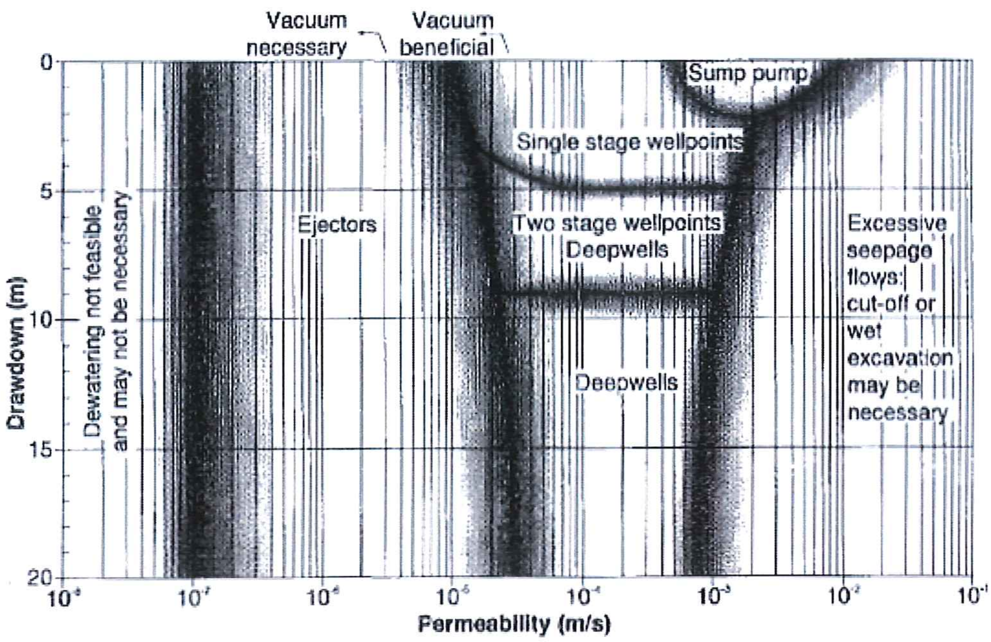
Considering the stratigraphy of the site, the ground water levels established, the depth of the proposed excavation, the peripheral wall barrier to be constructed, the test results obtained from the site investigations and laboratory tests, an estimate of the anticipated inflow of water into the excavation 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, and assumed that would be of the order of l/s and 0.087 l/s respectively, which is equal to approximately 4.5 7.5 m^3/day, in total 12 m^3/day. On the basis of infiltration tests results, carried out by Geoinvest in clayey marls (more or less of similar permeabilities with the substratum of the particular site), the infiltration rate might be of the order of 0.005 to 0.001 m^3/m^2/day. With an area of 8800 m2, the inflow might be of the order of 8.8 m^3/day and in the worst case scenario 44 m^3/day.

5 Ground Water Control

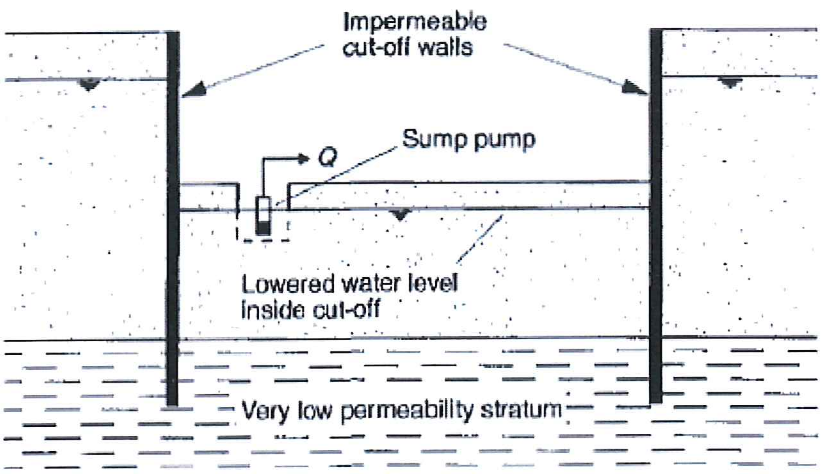
Techniques and methods to control ground water flow into the excavations of construction sites are provided by publication by CIRIA 515 (Ground Water Control, Design and Practice). The selection of the technique appropriate to the particular project at the particular site depends on various factors, but mainly on:

- the lithology,
- permeability of the soils,
- depth and dimensions of the excavation,
- the required drawdown and
- the extracted water management.

The proposed ground water control method has been selected, partly, from figure 1.10 and 1.8c of this publication (see below).



Considering the necessity to lower the water table at least 0.5m 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 as seen below. The total number of wells depends on the estimated total inflow of water and the pump capacity.



c) Excavation completely protected by a physical cut-off wall (retaining walls toeing into an impermeable stratum)

In the case under study, where the cut-off will be driven into a very low permeability stratum, the combination of pumped wells and sump pumps may be used. As already mentioned above, most of the inflow will originate from the base of the excavation and not from behind the cut-off wall. A number of drainage, large diameter boreholes and/or sump wells has to be distributed in the excavated area. They have to be drilled down to relatively shallow depths, near the interface with the Moni Clay (~ -3m), which is of very low permeability. The number of boreholes depend on the permeability, the thickness of the required drawdown, the area of the excavation and the time needed to reach the required drawdown. The time might be reduced if more boreholes and sump wells are constructed but the amounts of the water to be extracted daily, will be larger. On the other hand, the amounts of the daily extracted groundwater have to be such that the necessary treatment can be made before disposal. The design of the de-watering system is usually adjusted to suit the project's time schedule, the equipment available and techniques preferable to the contractor. It is standard practice, to adjust the de-watering system on the basis of the first results and after re-evaluate all factors considered, if necessary.

The boreholes might be 1000 mm in diameter with a 300mm perforated plastic pipe installed at the center. 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 natural aggregate 10 to 20 mm.

5.1 Dewatering Calculations

Specialized calculations have been carried out using DC-Dewatering v5.20 Software. For the calculations the following were taken into account:

- strata permeabilities,
- porous components,
- unit weights,
- groundwater level,
- lowering aim,
- side walls and base inflows as well as
- the use of cut-off wall and a tight base with very low permeability.

Considering the total footprint of the basement area, it seems that the excavation of the basement could proceed in one stage. The volume of the water to be pumped to reach the required drawdown is of the order of ~6.500 m^3 including the inflow from the base and walls.

It was calculated that in case of using 11 pumping wells of 1000mm diameter, 2.92 l/s or 252.6 m^3/day could be pumped out. The water volume was calculated to be of the order of 6547.0 m^3, thus 653.2 h or 27.2 days might be needed to pump the pit dry. Of course the pumping capacity can be increased lowering consequently the pumping time if the diameter and number of the wells are increased. More details can be found in appendix 1.

5.2 Lifts' Wells

The excavation of the Lifts' wells extend 1 m below the general excavation level. In this case a sump well in the close vicinity of the lifts might be adequate to keep the base of the excavation dry. If, surprisingly, not, a separate perimeter 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.

5.3 Mitigation of Environmental Effects

5.3.1 Dewatering Consenting

Extraction of groundwater for excavation dewatering and its disposal into water bodies (sea in this case) can be carried out only if the necessary Environmental Permits are granted by the Authorities. The Engineer and Contractor must fully understand the conditions for activities covered by the permits and prepare the necessary method statement and associated quality assurance system.

5.3.2 Discharge to the Environment

The usual practice is to dispose the dewatering water into the environment. The following specific factors need to be addressed in this case:

- Siting of the discharge,
- the effects on the discharge location and
- the ability of the discharge environment to accept the volume of discharge.

It should be born in mind that if the extracted water is contaminated, the contamination will be drawn into the system at the disposal site. Furthermore, sediments will be mobilized by ground water flow. The appropriate treatment and monitoring should be therefore implemented aiming to:

- treat the discharged water so that adverse discharge of suspended solids or contaminants is avoided,
- minimize the loss of fines from in-situ soils and avoid ground settlement.

In this case, the latter is not of importance, since there are no buildings in the immediate vicinity of the area.

In similar cases, the common practice adopted to dispose the water from dewatering activities is to pump it into the sea, at a point satisfying certain criteria and at a distance from the beach predefined by the Authorities.

In order to avoid all related impacts at the disposal site, as well as costly pumping to several hundred of meters into the sea, the following possible solution could be considered:

➢ Return the extracted sea water to the sea at the immediate vicinity of the excavation either through the calcarenites or through the reclaimed land (fill) outside the excavation area, through suitably constructed recharge wells.

The water will be filtered both through the pumping and recharge wells and through the natural calcarenite or fill and then will enter the sea in the same area, free of fines and possible contamination.

In conclusion, there are two options to dispose the pumped water, which in fact will be, mostly, sea water.

- Dispose to the sea at a distance to be predefined by the Authorities.
- Dispose at the site, through recharge wells after treatment including settlement of fines and filtering to further reduce the fines and possible contamination (see more details in next chapter).

In the first case, the necessary number of boreholes and/or sump wells can be used in such a way so that about 30 m^3/hour or 720 m^3/day could be pumped. In about less than 10 days, the water to be trapped within the cut off wall will be pumped out and the water level reduced down to the required depth.

In the second case, smaller amounts of water should be pumped so that they can be treated (reduce the fines through settlement tanks) and filtered through the discharge wells before disposal to the sea at the immediate vicinity of the buildings. Pumping of about 10.5 m^3 /hour will be necessary to reduce the water level down to the required depth in about 27 days. Afterwards, and provided that the cut off wall is successfully water proof, only negligible amounts (<10m^3/day and in extreme, unexpected cases <45m^3/day) of water will have to be extracted to keep the excavation dry.

5.3.3 Dewatering Discharge Quality – Suspended Solids

The dewatering water, either directly disposed in the sea or through suitably designed recharge wells should pass through sediment removal devices such as settling tanks, so that the suspended solids are reduced to less than 80 g/m^3. The number and dimensions of the tanks depend on the water to be extracted hourly or daily. Based on particle size and density of silt particles, it is estimated that a settling tank having dimensions of about 50-100 m^3 (smaller, if already, through the appropriately designed pumping well, the extracted water is filtered) will be required for the efficient removal of fines. From experiences, settling times are expected to be in the region of 1 hour.

The main consideration in this case is the possible clogging of the recharge boreholes, but it can be minimized with the appropriate design of both the pumping and recharging boreholes, as well as the settling tanks.

5.3.4 Ground Water Recharge System

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.

Such a scheme requires caution and careful planning. As a rule of thumb, for each abstraction well two or three 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. In this case this is not valid, since the layer from which the water is to be extracted is of small thickness and on the other hand the calcarenite, through which is to be returned to the sea is thick enough. Most probably the number of recharge wells might be the same as the number of the discharge wells, but this will be more precisely determined if a pumping/recharge test is carried out as mentioned below.

Recirculation will be kept to the minimum due to the cut off walls and the very low permeability of the soils at the base of the excavation.

5.3.5 Recharge Wells

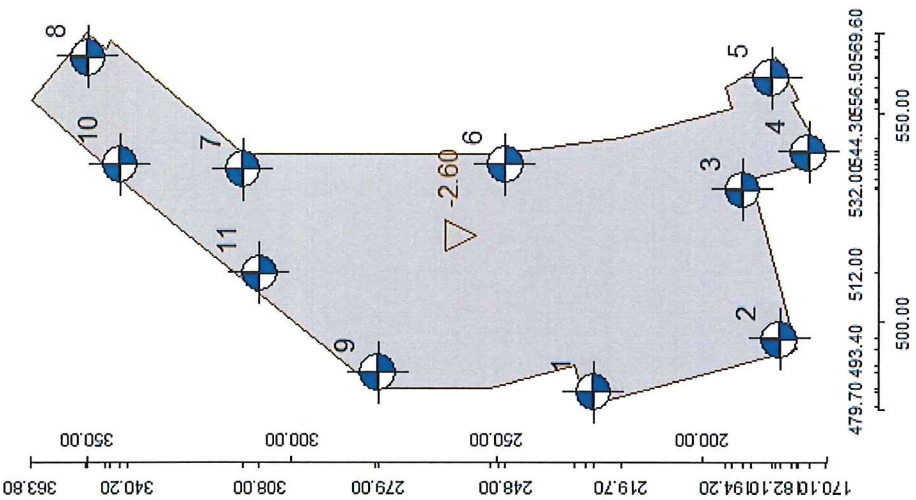
For the specific site under consideration, the wells can be taken down to a depth of about 4-5 meters into the calcarenite. The estimated permeability of the calcarenite is of the order of 2 to 6 $x10^{-3}$ cm/s with an average of the order of 3.5 m/day. On the basis of several in situ permeability tests in calcarenite carried out in several places in Cyprus, the infiltration rate was calculated to be about 14 m^3 per square meter per day in dry conditions. Through a borehole 4 meters long and one meter diameter having an area of about 14 m^2, 196 m^3/day could be infiltrated, at least at the initial stage before clogging and saturation takes place. A very rough estimate could give less than 30 days to reach the required drawdown with 3-5 number of recharge boreholes.

More precise estimates could be made only if a full-scale Pumping Test, in combination with a Recharge Test for recharging water back to the aquifer should be carried out in the immediate future, in order to further assess the in situ permeability of the various layers and infiltration/recharge capacity of the calcarenite into which the recharge could be effected.

The above tests will provide information regarding:

- The total discharge, flow rate and time required to achieve drawdown down to the excavation depth.
- Matters related with ground water control and point out any practical and environmental constraints.

APPENDIX 1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11
-2.60
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340.20
308.00
279.00
248.00
219.70
194.20
182.10
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479.70
493.40
500.00
512.00
532.00
544.30
550.00
556.50
569.60

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Analysis of ground water lowering (Herth/Arndts 1994)

Subsoil

Ground water depth	0.60 m
Depth of aquitard	3.60 m
Water level H	3.00 m
Storage coefficient p	0.20
Ground water situation:	Unconfined aquifer

Soil layer data

	FILL	**CLAY**
Layer height Δh[m]	3.60	26.40
Permeability k[m/s]	$5.78*10^{-5}$	$1.57*10^{-9}$
Permeability k dist.[m/s]	$5.78*10^{-5}$	$1.57*10^{-9}$
Porous component n[-]	0.30	0.45
Layer type	permeable	impermeable

Subsoil pit

No.	**Depth [m]**	**X [m]**	**Y [m]**	**Enclosure to [m]**	**Tight base top [m]**	**d [m]**
1	2.60	479.70	228.40	8.00	2.60	0.10
		489.60	231.10	**Wall - Flow - Base**		
		484.10	251.70	**[l/(s*m²)]**	**[l/(s*1000m²)]**	
		484.10	279.60	0.0001	0.01	
		532.40	340.20			
		553.40	363.80			
		569.60	350.30			
		565.80	345.80			
		567.60	344.30			
		540.30	312.20			
		540.30	249.60			
		544.30	219.70			
		551.50	192.90			
		556.50	194.20			
		563.50	182.10			
		553.90	176.60			
		553.00	178.10			
		539.20	170.10			
		534.40	188.10			
		493.40	177.10			

Series 1

Lowering = 2.50 m under static water level 0.60 m

Subsoil pit no. 1, Depth = 2.60 m:

Well Name	X [m]	Y [m]	Diameter [mm]	Depth [m]
1	483.37	226.29	1000	3.60
2	496.00	181.00	1000	3.60
3	532.00	190.00	1000	3.60
4	541.00	174.00	1000	3.60
5	559.00	183.00	1000	3.60
6	538.00	248.00	1000	3.60
7	537.00	312.00	1000	3.60
8	564.00	350.00	1000	3.60
9	488.00	279.00	1000	3.60
10	538.00	342.00	1000	3.60
11	512.00	308.00	1000	3.60

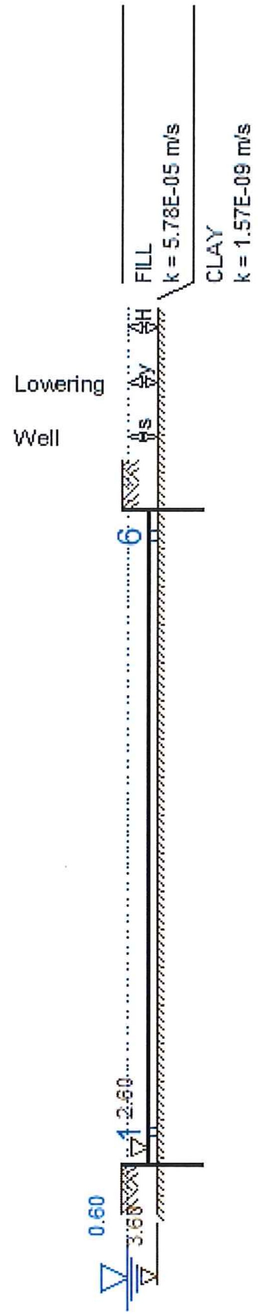
No	Water level in the well below surf. [m]	Lowering funnel s_{EB} [m]	Wetted filter height h [m]	Capacity q [l/s]
1	3.44	0.34	0.16	0.25
2	3.42	0.32	0.18	0.28
3	3.46	0.36	0.14	0.22
4	3.45	0.35	0.15	0.23
5	3.43	0.33	0.17	0.27
6	3.46	0.36	0.14	0.23
7	3.42	0.32	0.18	0.29
8	3.38	0.28	0.22	0.35
9	3.44	0.34	0.16	0.26
10	3.42	0.32	0.18	0.29
11	3.44	0.34	0.16	0.26

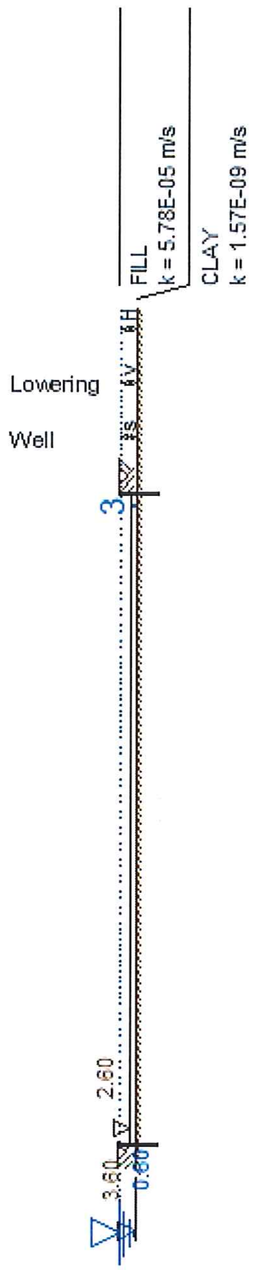
Required pumped quantity Q_0: 0.14 l/s, **Q max:** 0.14 l/s
Required: 1 wells
Available: 11 wells
Available pumped quantity Q: 2.92 l/s ***** sufficient *****

Maximum pumping capacity: 0.35 l/s
Required filter length: 0.22 m

Residual inflow through the wall: 0.052 l/s
Residual inflow through the base: 0.087 l/s
Inflow by rainfall: 0.000 l/s
Pumping capacity of the wells: 2.924 l/s
Water volume of subsoil pit: 6547.0 m³
Time to pump dry: 653.2 h

Minimum range acc. to Weyrauch (Bautechnik 7/2004): 71 m


FILL
k = 5.78E-05 m/s
CLAY
k = 1.57E-09 m/s
Lowering
Well
6
1
2.60
0.60
3.60


FILL
k = 5.78E-05 m/s
CLAY
k = 1.57E-09 m/s
Lowering
Well
3
2.60
3.60
0.60