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Design and construction of anchored diaphragm walls and dewatering system for the excavation of a two-level basement car park at the Al Ghazala Hotel Intercontinental in Tripoli—Libya

C. Asioli, M. Agostini & T. Minotti
Trevi S.p.A., Italy

ABSTRACT: This paper covers the adopted design criteria and the main results obtained in the design of the multi-anchored diaphragm wall and the dewatering system for the excavation of a two level basement car park at the Al Ghazala Hotel Intercontinental in Tripoli, Libya. The following design aspects are particularly highlighted: site geotechnical conditions and adopted design geotechnical parameters, preliminary design carried out to check the conceptual design and to choose the most economic solution, final construction design of diaphragm walls, anchors and dewatering system. Moreover, construction aspects concerning diaphragm walls, ground anchors and dewatering wells are also illustrated.

1 INTRODUCTION

From July 2007 until March 2008 TREVI S.p.A. was engaged in the design and construction of anchored diaphragm walls and dewatering systems for the excavation of a two-level basement car park at Al Ghazala Hotel Intercontinental in Tripoli, Libya.

The investor of the project was MAGNA PROPERTIES GROUP Ltd.—Libya and the Main Contractor was MAN ENTERPRISE S.A.L.. Design and construction management was performed by DAR ALHANDASAH Engineering.

Currently, the basement is completed and the building is under construction.

Figure 1. The site.

2 PROJECT DESCRIPTION

The project's location is in the town centre of Tripoli, Libya, approximately 150 m from the Mediterranean coast, near Al Ghazala round about.

The project dealt with a prestigious, multi-use development in the bay area of Tripoli overlooking the Mediterranean Sea. The development includes a two-level basement car-park, which is extending across the whole area of the site.

The site area had an almost flat shape of approximately 195 m × 70 m and the ground levels varied from 3.4 to 7.0 m from mean sea level.

For the raft foundation construction it was necessary to excavate the area down to −5.0 ÷ −8.0 m elevation. The excavation depth, measured from the ground level, is in the range of 8.3 ÷ 12.0 m.

Temporary support of the soil was achieved by multilevel anchored diaphragm walls.

The diaphragm wall, 600 and 800 mm thick, with a depth up to 18 meters, provided a continuous temporary structure to prevent soil collapse and water inflow during the digging works. The excavation was carried out by using a cable suspended, hydraulically operated clamshell. Two/three anchors' levels with 3 and 4 strands were adopted.

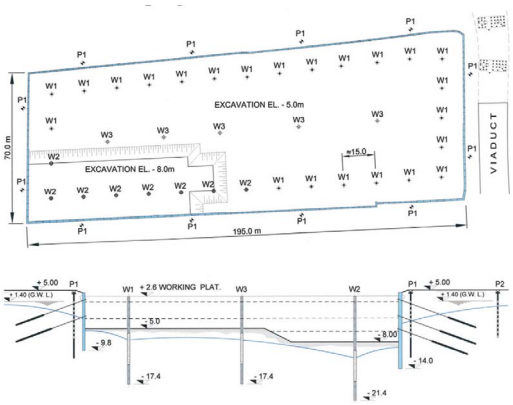

Figure 2. General layout and typical section.

For the deepest level, in order to avoid high inlet water flows through the wall, a preventer device was applied directly to the anchorage reservation installed in the diaphragm wall.

In order to control ground water level, 31 wells with a length of 20 ÷ 24 m were installed inside the excavation area 10 m far from the diaphragm walls. Longitudinal wells spacing was approximately 15 m. 5 additional wells were installed in the middle of the excavation area and were also utilized to check the groundwater level during dewatering stages. All the wells had a drilling diameter of 800 mm with an internal 355 mm diameter steel casing and a slotted length of 7 m.

Submersible pumps having discharge capability between 90 to 420 l/min with a minimum prevalence of 25 m were installed inside the wells.

Figure 2 shows the general layout and the typical section of the project.

3 GROUND CONDITIONS

The soil investigation, carried out by Al-Tamier Engineering Consulting Bureau, was carried out in three phases from June 2005 to October 2006. It comprised a total of 14 boreholes drilled to depths varying from 24 m to 55 m.

Standard Penetration Tests (SPT) were carried out in coarse grained deposits and in fine grained or completely weathered rock. Coring rock samples, where rock strata were encountered, were retrieved using Double Tube Core Barrel. Upon completion of boreholes, standpipes and piezometers were installed to monitor the ground water table. Rising head tests were performed into eight boreholes using a submersible pump and three pumping tests were also carried out into two boreholes after enlarging the borehole diameter. Physical, mechanical and chemical laboratory tests were carried out on soil samples and intact cylindrical rock core specimens taken from the boreholes.

The subsoil conditions encountered in the drilled boreholes indicated that in the first 3.0 m the subsoil consisted of dark brown to grey sands with some silts and some gravels, followed by a 27.0 m thick layer of light brown fine sands with some silts and some gravels (Pleistocene aged Gargaresh Formation). Thin layers of highly weathered limestone with roughly a 0.5 m thickness were encountered during the drilling. At a depth of 14.5 ÷ 15.5 m, the sand became cemented with very high SPT values.

This layer is followed by a layer of rounded boulders and fine sands whose thickness varied from 2.0 m to 4.0 m.

Finally a layer of slightly to highly weathered limestone, calcarenite and claystone (Middle Miocene Al Khums Formation), which continued till the end of boring, was encountered at a depth of about 35.0 m.

Ground water table is approximately +1.4 m from mean sea level.

Soil is mostly fine to medium sand (63%) with gravel (17%) and silt (20%). Prevalence of non plastic material is also confirmed by Atterberg Limits.

Relative density (Dr%) was evaluated from SPT test by means of the Skempton (1986) correlation for NC fine sands, angle of internal friction has been assumed according to Schmertmann (1977) suggestion, and elastic modulus was estimated according to D'Apollonia (1970) for NC fine sands.

Back analyses of pumping tests were carried out using the well theory of free aquifer, in order to estimate the average soil permeability.

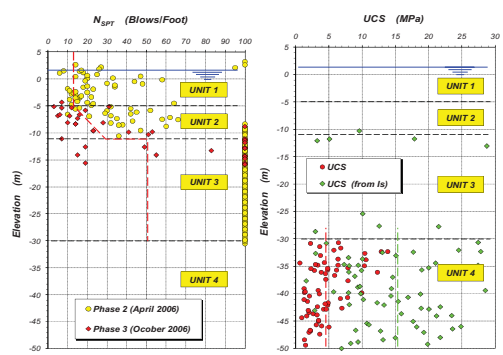

Figure 3. SPT values and UCS resistance.

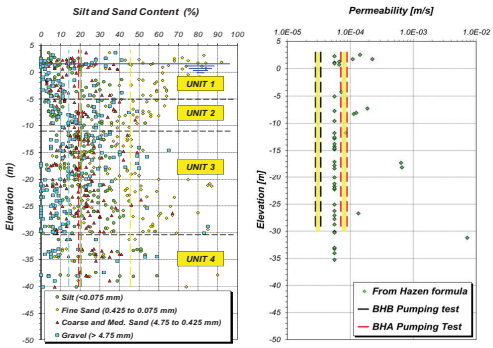

Figure 4. Fine content distribution and permeability.

Soil permeability was in the range of 7.1 ÷ 8.9 10^{-5} m/s for test A and in the range of 2.8 ÷ 3.4 10^{-5} m/s for test B. Figure 4 shows permeability values estimated by sieve analysis, using Hazen formula, and by pumping tests.

In summary, the design soil stratigraphy and the geotechnical design parameters adopted in the calculation were:

Unit 1: from ground level to −5.0 m, sand with silt and gravel. ($\gamma_n = 18.5$ kN/m³, $\varphi' = 30°$, $E_{50} = 5.5$ MPa; $E_{ur} = 14$ MPa; $k_h = 1.0 \times 10^{-4}$ m/s).
Unit 2: from −5.0 to −11.0 m, fine to medium dense sand. ($\gamma_n = 19.0$ kN/m³, $\varphi' = 33°$, $E_{50} = 13$ MPa; $E_{ur} = 32.5$ MPa; $k_h = 1.0 \times 10^{-4}$ m/s).
Unit 3: from −11.0 m o −30 m, very dense/cemented sand. ($\gamma_n = 20.0$ kN/m³, $\varphi' \geq 35°$, $E_{50} = 24.7$ MPa; $E_{ur} = 62$ MPa; $k_h = 6.0 \times 10^{-5}$ m/s).
Unit 4: below −30 m, weathered limestone ($k_h = 1.0 \times 10^{-5}$ m/s). This unit has been considered only for dewatering analysis.

4 PRELIMINARY ANALYSIS

In order to evaluate the dewatering effects on the adjacent structures, select the wells position and check the wall's dimension quoted in the conceptual design, a preliminary calculation was carried out.

In order to take into account all the aspects involved with the dewatering system and with the excavation phases (settlement of the adjacent building, position and number of wells, stresses on the diaphragm wall and anchors, bottom excavation stability, global stability, etc.) a FEM analysis has been carried out using Plaxis® code.

A Hardening Soil Model, with a different stiffness for primary loading or unloading-reloading stress paths, was adopted to schematize soil layers.

The three following different models were analyzed, with different pumps position and wall embedded length, to find out the most economic solution.

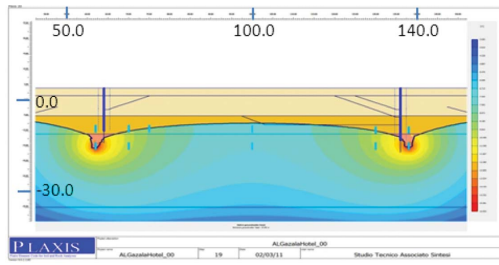

Figure 5. Model A—G.W. Head (Red = −15 m; Blue = 3 m).

Model A: dewatering by external deep wells; wall embedment length 5 m.
Model B: dewatering by internal deep wells; wall embedment length 5 m.
Model C: dewatering by internal deep wells; wall embedment length 9 m.

Figure 5 shows the position of the water level after the water table lowering in the Model A (by external wells). In order to maintain the water table inside the excavation area below the final excavation level, the required pumping rate was 80 m³/day/m and the outside water table was lowered to −13.0 m elevation.

The maximum vertical settlements were in the range of 26 ÷ 30 mm with a very wide influence area.

In models B and C, adopting three internal wells, the external water table was lowered to a −5.0 m elevation and the maximum vertical settlements were in the range of 20 ÷ 24 mm.

Required pumping rates, to maintain the water table below the final excavation level, were respectively equal to 45 and 35 m³/day/m.

The 4 m more diaphragm wall length of model C didn't bring big advantages, hence the inside position of the wells and the diaphragm wall embedment length of 5 m were chosen (model B) in order to have a smaller water discharge and keep the external water level as high as possible.

Wall thickness of 600 and 800 mm and anchor levels quoted in the Conceptual Design were confirmed by the preliminary analysis.

5 DIAPHRAGM WALLS' FINAL DESIGN

Due to different external surcharges and geometries, six different sections were analyzed in order to achieve the most economic solution for the retaining wall. The deepest section geometry (type 2) is illustrated in Figure 6.

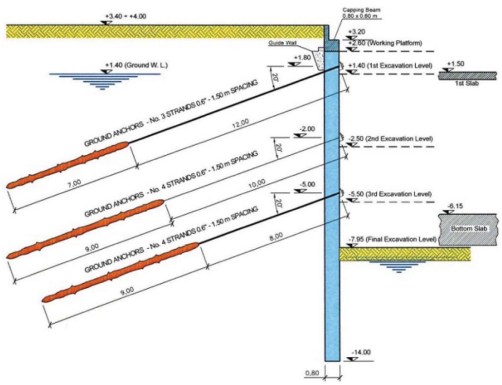

Figure 6. Section type 2.

5.1 *Method of analysis*

In order to evaluate stresses in the diaphragm wall and anchors, the computed program PARATIE v.6.2 (distributed by Harpaceas®), which gives reliable results despite its simple use, has been utilized.

The program takes into account the soil-structure interaction and is able to follow the diaphragm walls' behavior during all execution phases.

Diaphragm wall is modeled by means of a one-dimensional linear elastic finite element with a bending stiffness "EJ" (beam element); the behavior of soil is taken into account by elasto–plastic springs, jointed to the nodes of the beam element, having a stiffness "k" proportional to the soil's in-situ stiffness. Moreover, the code allows to model support structural elements like anchors and props, as elastic springs with an axial stiffness "EA".

The soil-structure interaction problem is solved adopting the Winkler's model which subdivides the soil into independent strata.

The elasto-plastic constitutive law used to model the behavior of the soil is defined by the active and passive pressure coefficient. When the analysis starts, the soil is in geostatic conditions i.e. the ratio σ'_h/σ'_v is equal to k_0 and the soil behaves elastically until the ratio σ'_h/σ'_v is in the range (k_a, k_p), when the ratio σ'_h/σ'_v reaches the limit value k_a or k_p stresses are set to active or passive pressure.

5.2 *Calculation phases*

According to construction sequences the following calculation phases have been considered:

1. Excavation up to elevation +1.4 m;
2. Installation and tensioning of the first anchor's level;
3. First dewatering stage (internal water level at –3.0 m) and excavation up to elevation –2.5 m;
4. Installation and tensioning of the second anchor's level;
5. Second dewatering stage (internal water level at –6.0 m) and excavation up to elevation –5.5 m;
6. Installation and tensioning of the third anchor's level;
7. Third dewatering stage (internal water level at –8.5 m) and excavation down to elevation –8.0 m;
8. Construction of the bottom slab;
9. Detensioning of the third and second level of anchors;
10. Construction of the top slab;
11. Detensioning of the first level of anchors;

Soil stiffness parameters have been calibrated on FEM results and are illustrated below:

- Unit 1: $E_{vc} = 37$ MPa, $E_{ur} = 93$ MPa;
- Unit 2: $E_{vc} = 40\ [(\sigma'_v/100)]^1$ MPa, $E_{ur} = 2.5\ E_{vc}$;
- Unit 3: $E_{vc} = 71\ [(\sigma'_v/100)]^1$ MPa, $E_{ur} = 2.5\ E_{vc}$;

To take into account the worse hydraulic condition, a hydrostatic water pressure distribution on the faces of the wall has been considered, adopting the original external water level and assuming that the internal water level would be 50 cm below the excavation level.

5.3 *Results*

Structural calculation of the steel reinforcement was performed according to Eurocode EC2 "Design of concrete structures", considering a minimum value of 1.5 for the partial safety factor on load (γ_f).

Since such factoring is incompatible with wall-ground interaction analysis as it introduces artificial yielding in the ground springs, design stresses were calculated applying the safety factor directly to the characteristic stresses.

Anchors bond length, executed with I.G.U. method (single stage grouting at low pressure), was calculated according to the well-known Bustamante-Doix proposal considering an ultimate adhesion between foundation and soil equal to 240 kPa.

Allowable capacity of temporary anchors was evaluated assuming a safety factor equal to 2.0.

As far as the deepest section's results are concerned, the PARATIE program has been compared to results from PLAXIS calculation. Figure 7 shows good agreement between the bending moment evaluated with the two different analyses.

5.4 *Global stability*

Global stability analysis has been conducted via c'/φ' reduction procedure available in Plaxis. The program reduces soil resistance parameters until failure is reached.

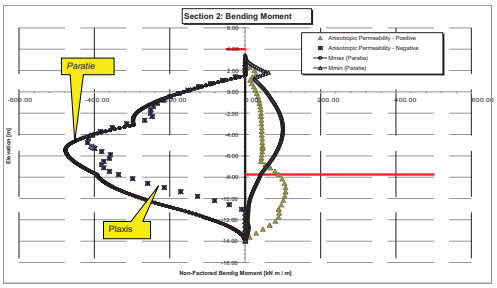

Figure 7. Results comparison.

Therefore the safety factor is:

$$FS = c'_{design} / c'_{failure} = \tan \cdot (\varphi'_{design}) / \tan (\varphi'_{failure})$$

The minimum safety factors obtained were 1.68 for the deepest section (type 2) and 1.55 for section 1.

Figure 8 shows the collapse mechanism in correspondence of the final excavation level, considering anisotropic condition for soil permeability.

6 DESIGN OF THE DEWATERING SYSTEM

Dewatering analysis was carried out by Plaxis® considering an isotropic soil permeability ($k_h = k_v$) which maximizes the water discharge quantity.

Pumping wells were located inside the excavation area in two lines, approximately 10 m far from diaphragm walls. Pumps elevation has been considered at the middle depth of the slotted length of wells: pump #1 (left side) was at –10 m and pump #2 (right side) was at –14.5 m.

Dewatering simulation was executed by an iterative calculation, changing the water discharge until the piezometric level inside the excavation area was 50 cm below the excavation level. In this way it was possible to find out the global water discharge necessary to lower the water for each dewatering stage.

Figure 9 shows flow field for the final dewatering stage.

At the final excavation depth, the maximum water discharge was 27 m^3/day/m for the pump #2, close to the deepest section, and 16 m^3/day/m for the pump #1 located where the maximum excavation reached –5.0 m. Considering the perimeter of the rectangle containing all wells, the total water discharge for the whole area has been estimated as follows:

$$Qt = 16 \times 350 \text{ m} + 27 \times 100 \text{ m} = 8300 \text{ m}^3/\text{day}.$$

Assuming a factor of safety equal to 1.5, the pumping system was dimensioned for 12500 m^3/day.

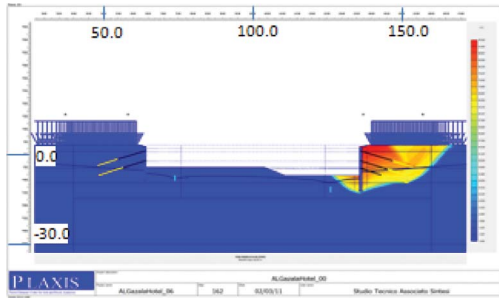

Figure 8. Failure mechanism for section 2. (Displacement label: Red = 10 m; Blue = 0 m).

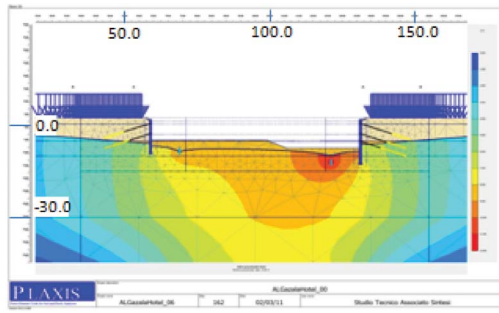

Figure 9. Piezometric head at final dewatering stage. (Red = –12.0; Blue = 0.0 m).

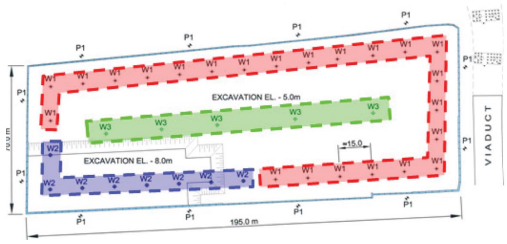

Figure 10. Wells distribution.

A total of 31 main wells were adopted, plus 5 additional wells, positioned in the middle of the excavation area, to be activated only in case of necessity.

Inside each well, a D = 355 mm steel pipe having a screen length of 7 m was positioned for pump installation. A maximum slot size equal to 2 mm with a 10% minimum slotted surface has been adopted.

Six inches submersible pumps, having a working discharge capability between 130 to 600 m^3/day (90 to 420 l/min) with a minimum prevalence of

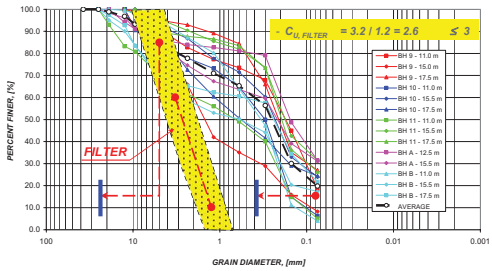

Figure 11. Grain size distribution of the filter.

25 m were selected. In order to monitor the ground water level outside the excavation area, 13 standpipes were installed all around the diaphragm wall perimeter.

6.1 *Filter material*

Filter grain size distribution has been designed according to U.S. Corps of Engineers (1955) "Drainage and Erosion Control" and Ciria report C515 "Groundwater Control—Design and Practice".

In particular, the following criteria were adopted:

- The D_{15} size of the filter should not be 5 times greater than the D_{85} size of the natural soil surrounding the filter (to prevent movement of fines from the aquifer).
- The D_{15} size of the filter should not be 4 times less than the D_{15} size of the protected material (filter more permeable than the aquifer).
- The maximum size of the filter particles (D_{100}) should not be bigger than 9.5 mm.
- Uniformity coefficient of the filter ($C_U = D_{60}/D_{10}$) should be less than 3 (to minimize risk of segregation during placement).

Figure 11 shows the adopted filter grain size distribution, evaluated by taking into account the average grain size distribution of the natural soil in correspondence of the slot part of the pipe.

7 CONSTRUCTION

7.1 *Diaphragm walls*

The diaphragm wall excavation was carried out by using a SOILMEC BH-12 hydraulically operated clamshell, suspended from a Link-Belt 318 crane. A service crane having a lifting capacity of 55 tons was adopted for cage installation and concreting. The trench stability during excavation was assured by the presence of stable bentonite suspension.

Figure 12. Soilmec BH12 clamshell.

Figure 13. Stop-end extraction by hydraulic jacks.

Figure 14. Anchors executions.

After completion of primary panels excavation and before steel cage installation, two steel sheet piles (stop ends) were installed at both ends of the trench. The stop ends were extracted after the concrete pouring, in order to create a non linear

contact with the concrete of the secondary panels and increase the shear resistance of the joint.

33% of regular panels and all irregular ones (corners and multiple panels) were tested for integrity using cross-hole sonic test. For each panel, 4 sonic pipes were installed with the steel cage.

7.2 *Anchors*

Temporary anchors were constructed in compliance with the European code EN 1537—"Execution of special geotechnical work—Ground Anchors".

Two Soilmec SM 405/8 drilling rigs equipped with a mast extension able to reduce the rods handling operations were utilized. The cased rotary drilling method was adopted to prevent soil collapse of the sandy soil.

For the deepest anchor level, in order to avoid high inlet water flows through the wall, a "preventer" device was applied directly to the anchorage reservation connected to the steel cage during the diaphragm wall execution.

Anchors were injected in a single stage at low pressure (max net pressure 5 ÷ 6 bar) using a dedicated grouting pipe right after the completion of the installation. The drilling fluid was displaced by the grouting pumped from the tip of the anchor.

Anchors' pre-stressing was carried out by single unit hydraulic jacks fed by a hydraulic power pack unit.

During stressing operations, the behavior of all anchors was tested up to 1.2 times the design load in order to compare it with the theoretical design behavior.

The effective free length (L_{ef}) had to be between the following limits:

$$0.9 \times L_{free} \leq L_{ef} \leq L_{free} + 0.5\ L_{bond}$$

Figure 15 shows some examples of the acceptance test.

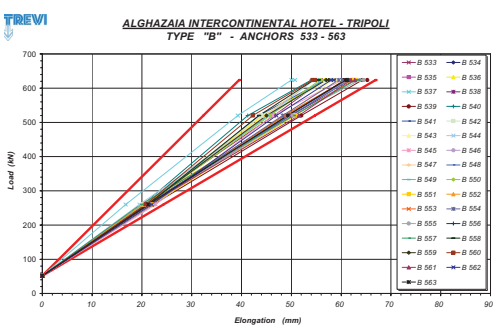

Figure 15. Examples of the acceptance test.

Figure 16. Situation before pumps activation.

Figure 17. Situation after pumps activation.

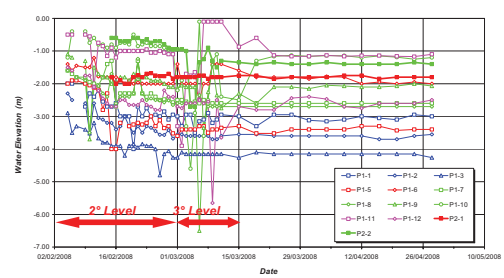

Figure 18. Water level monitoring.

7.3 *Dewatering*

Drilling of wells was executed without bentonite suspension, after all the surrounding diaphragm walls were completed. A temporary 800 mm casing, was installed by means of a Soilmec R50 hydraulic rotary rig. After each hole was drilled to its full length, it was flushed out with clean water to remove any loose material. The well screen and casing were then installed. The filter material was eventually introduced by gravity feed down the outer annulus. The temporary casing was removed as the filter sand was placed.

The well was then developed by airlift for a 1 hour minimum time or until the discharge water was free from drilling mud and/or fines. Admissible sand content after well development should be less than 100 p.p.m.

A 6" diameter electric submersible pump (type Lowara 6Z611/5 and 6Z621/2) suspended by means of a support rope, was installed in each well with the pump inlet located towards the base of the well.

The discharge's main take-off was installed around the excavation area by Main Contractors with take offs at each well location; the take-off was to be fitted with a non return valve.

Water discharge was managed by Main Contractor.

An average of approximately 10000 m^3 of water per day has been discharged for one year. Sedimentation tank was not necessary.

13 observation standpipes were installed for monitoring drawdown water levels outside the area. Records for a 3 month period, in correspondence of the soil mass excavation phases and anchors execution, show a water lowering just outside the diaphragm wall, varying from 2.5 to 5.5 m compared with the natural level.

When the construction basement was completed, approximately one year after pumps installation (February 2008–March 2009), pumps were extracted and wells were grouted by cement mortar.

7.4 *Production*

A total of 7250 m^2 of diaphragm walls was completed with a single equipment in 66 working days (11 weeks), from October to December 2007, with an average production of 110 m^2 per day and a peak of 200 m^2 per day.

After diaphragm walls' completion, 36 wells were executed in 15 working days.

The soil mass excavation of approximately 150000 m^3 and the simultaneous installation of 575 anchors (10200 m) were completed in 66 working days, from January to March 2008, using two drilling rigs with an average production of 26 anchors per week per rig.

8 CONCLUSIONS

This was a real "Teamwork Project". All parties involved did their best, bearing in mind the benefits for the Project.

Since the design stage, up to the completion of the soil mass excavation, all issues have been approached and rapidly resolved with the involvement of the Engineer, the Main Contractor, the Foundations Specialist and the Designer.

A remarkable time-saving result has been achieved during the anchors construction phase because the excellent cooperation between the Main Contractor, who was in charge of the soil mass excavation, and the Specialized Contractor avoided any loss of time.

The strong links between the dewatering system and the diaphragm walls design gave us the opportunity to provide to the Engineer with the overall project design.

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