THE MAGAZINE OF THE DEEP FOUNDATIONS INSTITUTE MAY/JUNE 2017

# DEEP FOUNDATIONS

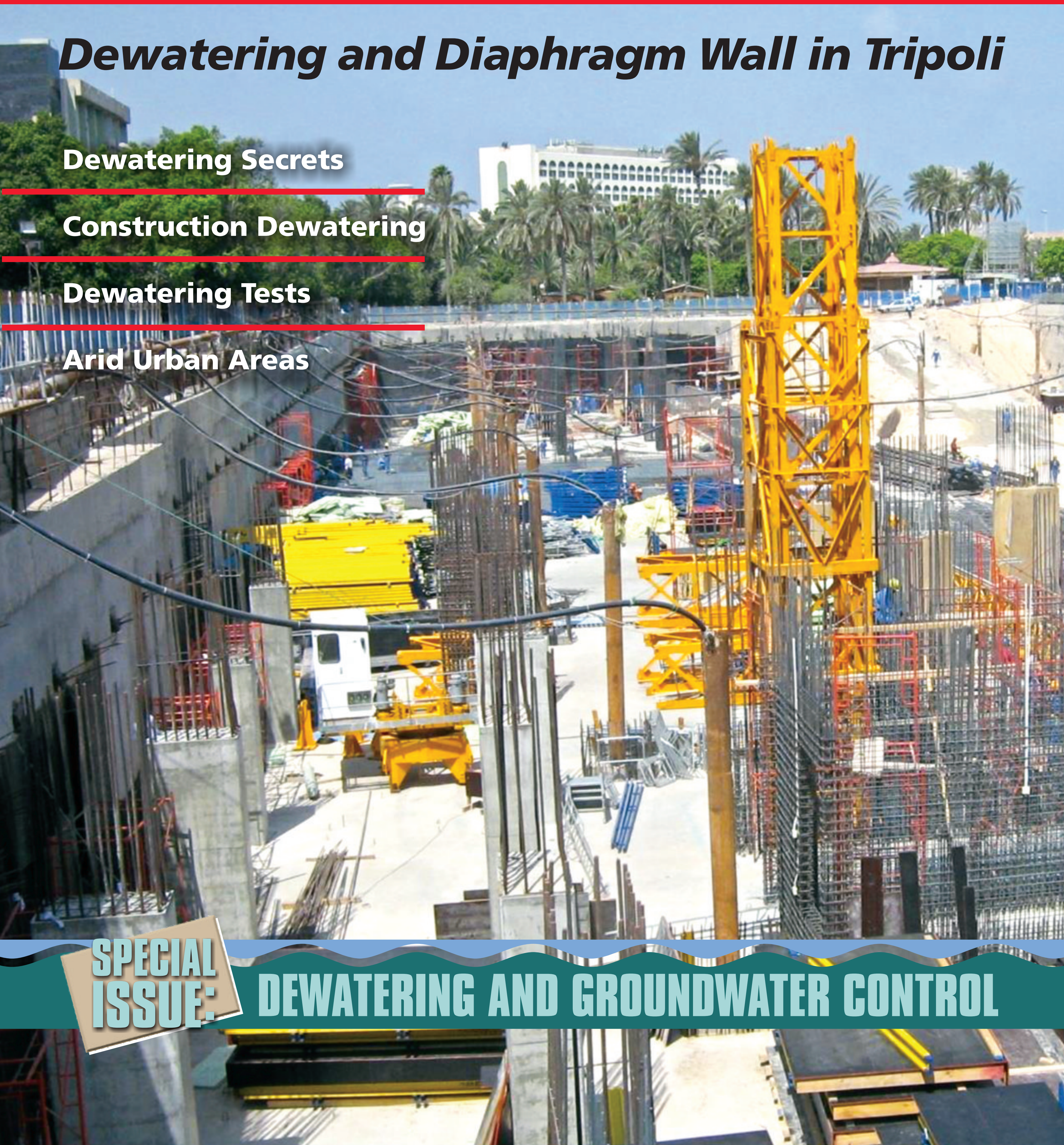
## *Dewatering and Diaphragm Wall in Tripoli*

**Dewatering Secrets**

**Construction Dewatering**

**Dewatering Tests**

**Arid Urban Areas**



**SPECIAL ISSUE: DEWATERING AND GROUNDWATER CONTROL**

COVER STORY



COVER STORY

SPECIAL ISSUE: DEWATERING AND GROUNDWATER CONTROL

# Anchored Diaphragm Wall and Dewatering System for Libya Hotel

The project's goal was to construct the Al Ghazala Intercontinental, a multi-use development in the bay area of Tripoli overlooking the Mediterranean Sea. The site of the project is located approximately 500 ft (150 m) from the Mediterranean coast, near the Al Ghazala roundabout in the town center of Tripoli, Libya. The investor of the project was Magna Properties Group, Ltd. – Libya, the main contractor was Man Enterprise S.A.L., and the design and construction management was performed by Dar Al-Handasah Engineering. Trevi S.p.A. was contracted to perform the design and construction of the anchored diaphragm walls and dewatering systems for the excavation of a two-level basement car park, which extends across the full footprint of the site.

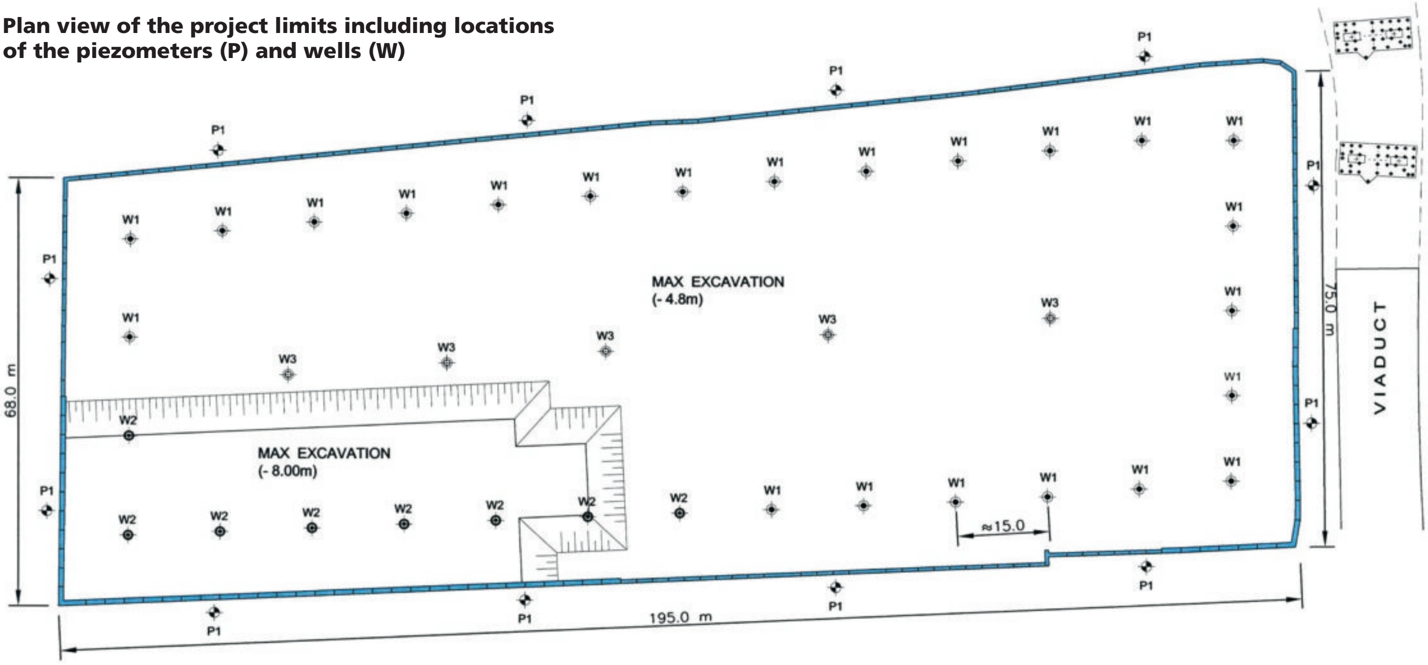

The site was approximately 640 ft by 230 ft (195 m by 70 m) in plan and nearly flat. The groundwater level typically varies from about 11 ft to 23 ft (3.4 m to 7.0 m) above mean sea level. To construct the required raft foundation, it was necessary to excavate the area down to El. -15.7 ft to -26.3 ft (-4.8 m to -8.0 m), which resulted in an excavation depth ranging from about 27 ft to 39 ft (8.3 m to 12.0 m) below the existing ground surface.

## Subsurface Conditions

The subsurface site investigation was performed in three phases by Al-Tamier Engineering Consulting Bureau. During Phase 1, a preliminary investigation was undertaken during the early stages of the project to obtain information about the general stratigraphy and groundwater level at the site. Phase 2 was performed to obtain subsurface information and design parameters, and Phase 3 was performed as a supplementary ground investigation to investigate unexpected ground conditions identified during Phase 2. In total, 14 boreholes were drilled to depths ranging from about 80 ft to 180 ft (24 to 55 m). The generalized subsurface profile is quite heterogeneous, and is predominantly composed of sand with silt and gravel that is underlain by limestone, calcarenite and claystone.

AUTHOR Claudio Asioli, Trevi Group - Design, Research and Development Department

Plan view of the project limits including locations of the piezometers (P) and wells (W)




| Thickness | Description |
|---|---|
| ±10 ft (3 m) | Sands with some silts and gravels; contains thin layer of peat |
| ±88.5 ft (27 m) | Fine sand with some silt and gravel; contains thin layer of highly weathered limestone and thin layer of cemented sand |
| ±1.6 ft to 13 ft (0.5 m to 4 m) | Rounded boulders with fine sands |
| ±16.5 ft to 23 ft (5 m to 7 m) | Calcareous siltstone; contains fine sand |
| 115 ft to 180 ft (35 m to 55 m) | Slightly to highly weathered limestone; calcarenite and claystone encountered |

**Generalized subsurface profile**

**Photograph of the extents of the project site**

Blow counts were obtained using the Standard Penetration Test (SPT) in the coarse-grained and fine-grained soils and in the completely weathered rock. Core samples of the underlying limestone were obtained using a double-tube core barrel that was about 4-3/8 in (112 mm) in diameter. Physical, mechanical and chemical tests were performed in the laboratory on soil samples and intact cylindrical rock core specimens obtained from the boreholes.

| Depth | Design Parameters |
|---|---|
| 0 to ±16 ft (0 m to 5m) | SAND with silt & gravel. $SPT_{avg}$ = 15 blow/ft; $\gamma_{tot}$=115 to 118 pcf (18.0 to 18.5 kN/m$^3$); $\phi'$=30°; $E'$=1450 to 2175 psi (10 to 15 MPa) |
| ±16 ft to 36 ft (5 m to 11 m) | SAND. $D_r$=40 to 60 %; SPT incr. with depth; $\gamma_{tot}$ = 118 to 121 pcf (18.5 to 19.0 kN/m$^3$); $\phi'$=33°; $E'$=2175 to 4350 psi (15 to 30 MPa) |
| ±36 ft to 98 ft (11 m to 30 m) | Very dense/cemented SAND. SPT shows refusal (where available); $\gamma_{tot}$ = 121 to 127 pcf (19.0 to 20.0 kN/m$^3$); $\phi'$≥35°; $E'$≥7250 psi (50 MPa) |
| >98 ft (> 30 m) | Weathered LIMESTONE (unit considered for dewatering analysis only) |

**Geotechnical design parameters**

Standpipes and piezometers were installed to monitor the changes in groundwater levels. Daily readings of the groundwater level in each standpipe and piezometer were made and recorded during the site investigation and for a period of nearly one month after the completion of the site investigation. Rising head tests were performed in eight boreholes using a submersible pump, where the water level was drawn down to a maximum depth of about 18 ft (5.5 m) from the ground surface and was then returned to its original level in about 2 to 3 minutes. In addition, three pumping tests were conducted in two boreholes, where the pumping rate varied from about 80 gpm to 240 gpm (5 l/sec to 15 l/sec) during the tests.

To estimate the average hydraulic conductivity of the soil, back analysis of the pumping test results were performed using well theory for an unconfined aquifer. The hydraulic conductivity of the soil ranged from about 7.1x$10^{-3}$ cm/s to 8.9x$10^{-3}$ cm/s for the tests in borehole BH-1 and about 2.8x$10^{-3}$ cm/s to 3.4x$10^{-3}$ cm/s for the tests in borehole BH-2.

## Preliminary Analysis

A preliminary analysis was performed using Plaxis, a finite element software program, to evaluate the effects of both the dewatering on adjacent structures and the conceptual design for and stability of the diaphragm walls and ground anchors. Three different models were analyzed using a soil-hardening model with different stiffness for the primary loading or unloading-reloading stress paths. The models considered different positioning of the wells (external vs. internal) and diaphragm wall embedment (16.5 ft vs. 29.5 ft [5 m vs. 9 m]).

Based on the results of the simulations, an embedment depth for the diaphragm walls of about 16.5 ft (5 m) was selected, as greater embedment did not result in big advantages for the performance of the constructed wall system. Depending upon the depth of excavation in a given area, the results indicated that a wall thickness of either about 24 in or 32 in (600 mm or 800 mm) with two or three anchor levels, respectively, would be adequate. To maintain the water level inside the excavation below the base of the excavation, three internal wells with a pumping rate between 117 and 150 gal/hr/ft (35 and 45 $m^3$/day/m) would be required. The maximum vertical settlement was estimated to be less than about 1 in (25 mm). Thus, it was decided to utilize internal wells with a spacing that would result in a lower discharge from the wells to keep the water level below the base of the excavation and the water level as high as possible external to the diaphragm walls.

## Design of the Diaphragm Wall

Considering the in-situ conditions, dewatering stages, external surcharges and different geometries, six different sections were analyzed to achieve the most economical solution for the diaphragm wall. To evaluate stresses in the diaphragm wall and ground anchors, the computer software program PARATIE was used, which can incorporate the soil-structure interaction and the different stages of diaphragm wall construction and excavation.

The diaphragm wall was modeled using one-dimensional linear elastic finite elements with a beam element bending stiffness. The constitutive model uses elasto-plastic springs to capture the behavior of the soil. The software program allows structural elements such as ground anchors and props to be modeled as elastic springs with an axial stiffness. The soil-structure interaction was analyzed using the Winkler model, which subdivides the soil into independent strata.

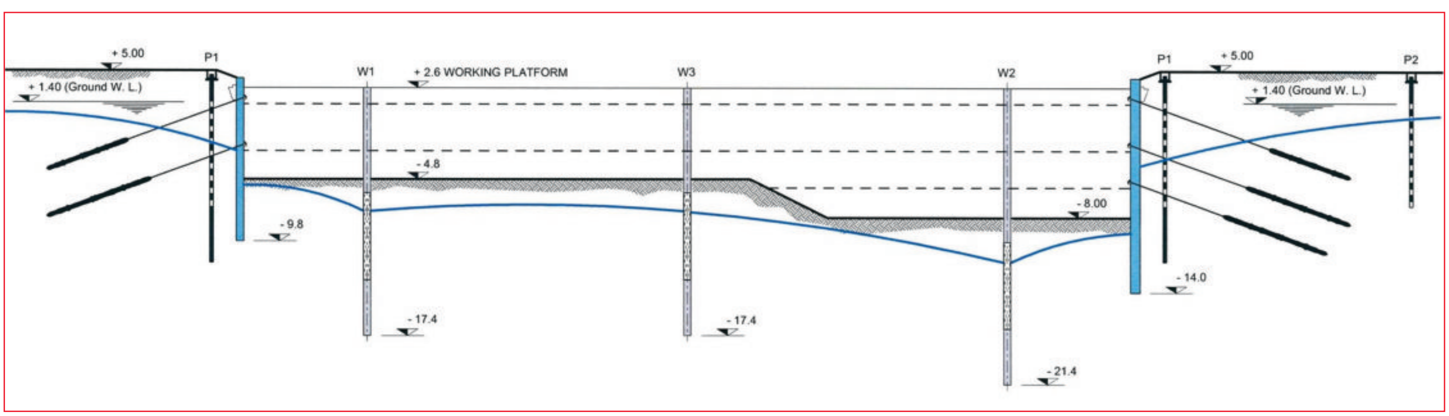
The model was analyzed for the deepest section of the excavation and for the worst case for the hydraulic condition (i.e., original external water level and internal water level about 1.6 ft (0.5 m) below the final excavation level). The following 11 steps were modeled to represent the construction sequencing:

1. Excavate to El. +4.6 ft (El. +1.4 m)
2. Install and tension first level of ground anchors
3. Perform first dewatering stage (draw down internal water level to El. -9.8 ft (El. -3.0 m)), and excavate down to El. -8.2 ft (El. -2.5 m)
4. Install and tension second level of ground anchors
5. Perform second dewatering stage (draw down internal water level to El. -19.7 ft [El. -6.0 m]) and excavate to El. -18.0 ft (El. -5.5 m)
6. Install and tension third level of ground anchors
7. Perform third dewatering stage (draw down internal water level to El. -27.9 ft [El. -8.5 m]) and excavate to El. -26.2 ft (El. -8.0 m)
8. Construct bottom slab
9. Detension third and second levels of ground anchors
10. Construct top slab
11. Detension first level of ground anchors

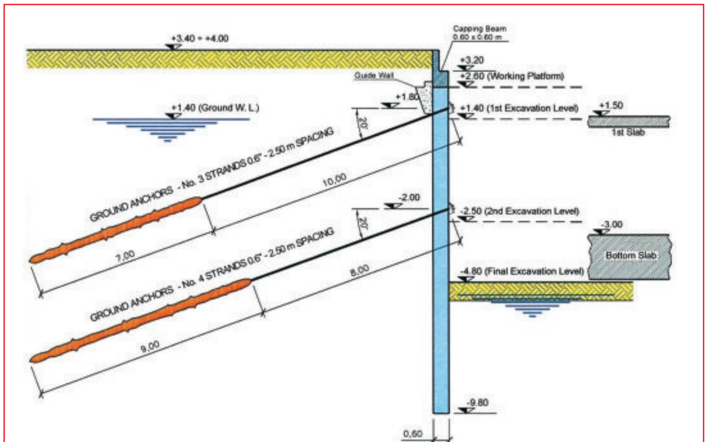
The structural design of the wall was performed according to Eurocode *EC2 - Design of Concrete Structures*. The ground anchors were designed according to $P_{lim}=\alpha\pi D q_s L_b$ where $P_{lim}$=anchor design load, D=drilling diameter (±6.2 in [160 mm]), where $L_b$=bond length (23 ft to 30 ft [7 m to 9 m]), and $\alpha q_s$ = grout-to-ground bond stress (±35 psi [240 kPa] for single stage, IGU injection [low pressure grouting] technique according to Bustamante-Doix). A factor of safety of 2.0 was used for the temporary ground anchors. A global stability analysis was performed using the c'/φ' reduction procedure available in Plaxis, and the minimum factors of safety computed were 1.68 and 1.55 for the deepest and shallowest excavations, respectively.

## Design of the Dewatering System

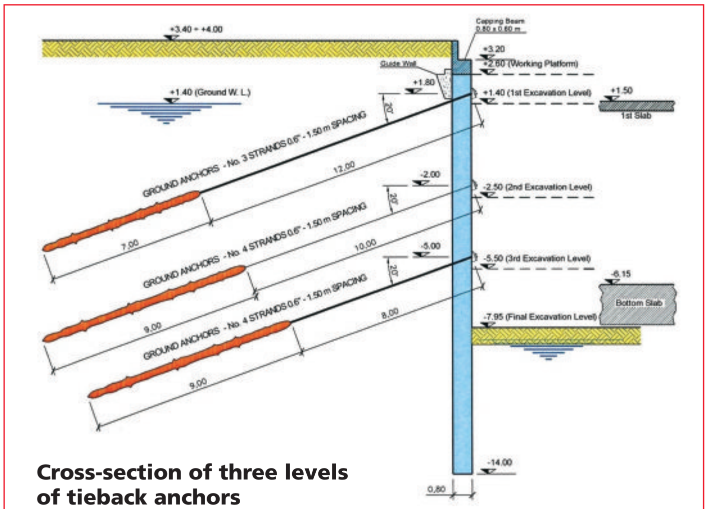
Analyses of the dewatering system were performed using Plaxis and considered two different permeability conditions: (1) isotropic soil permeability ($k_h = k_v$), which maximizes the water discharge, and (2) anisotropic soil permeability ($k_h = 10\ k_v$). The pumping wells were modeled inside the excavation area in two lines and were




**Typical cross-section across the project site**




**Cross-section of two levels of tieback anchors**




**Cross-section of three levels of tieback anchors**

located at a distance of approximately 33 ft (10 m) from the face of the diaphragm walls. The pumps were modeled at the mid-depth of the slotted length (i.e., pumps 1 and 2 were at about El. -33 ft [El. -10 m] and El. -47.6 ft [El. -14.5 m], respectively). The dewatering simulations were solved iteratively by altering the discharge until the piezometric level inside the excavation area was about 1.6 ft (0.5 m) below the base of the excavation. The total discharge for the entire excavation was estimated considering the perimeter that envelopes the wells. The total discharge was computed to be approximately 1,523 gpm (8,300 m$^3$/day). Using a factor of safety of 1.5, the pumping system was dimensioned for a total discharge of 2,293 gpm (12,500 m$^3$/day).

| Dewatering Stage | Well #1 Discharge | Well #2 Discharge | Total Discharge |
|---|---|---|---|
| 1 | 33.6 gal/hr/ft<br>10 m$^3$/day/m | 33.6 gal/hr/ft<br>10 m$^3$/day/m | 67 gal/hr/ft<br>20 m$^3$/day/m |
| 2 | 53.7 gal/hr/ft<br>16 m$^3$/day/m | 53.7 gal/hr/ft<br>16 m$^3$/day/m | 107 gal/hr/ft<br>32 m$^3$/day/m |
| 3 | 53.7 gal/hr/ft<br>16 m$^3$/day/m | 90.6 gal/hr/ft<br>27 m$^3$/day/m | 144 gal/hr/ft<br>43 m$^3$/day/m) |

**Results of the dewatering simulations**

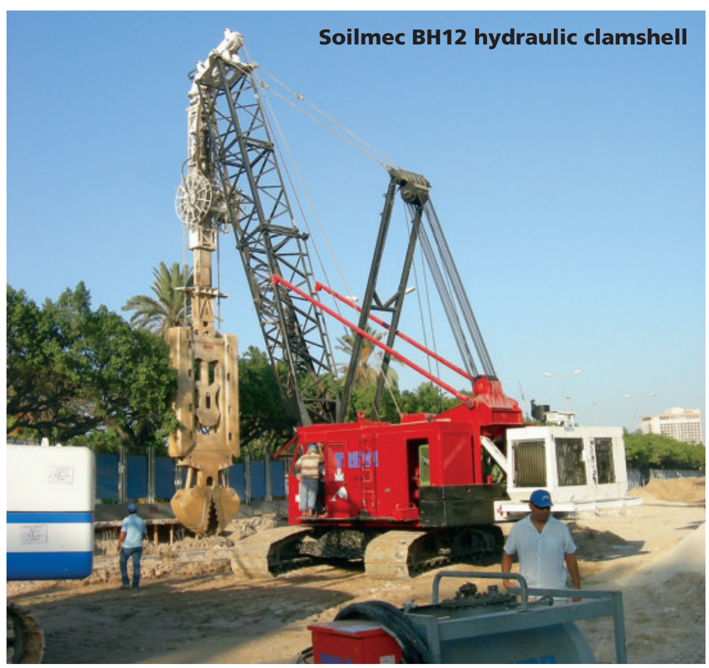
Therefore, to control groundwater, 31 wells were installed inside the excavation area at a distance of about 33 ft (10 m) from the face of the diaphragm walls. The spacing between the wells was approximately 50 ft (15 m) center-to-center. In addition, five wells were installed within the middle of excavation area to monitor the groundwater levels during the dewatering stages and to be activated only in the event of necessity.

Each of the wells was approximately 65 ft to 78 ft (20 m to 24 m) in length and about 32 in (800 mm) in diameter, and had an internal steel casing with a diameter of approximately 14 in (355 mm), and had a slotted length of about 23 ft (7 m). The maximum slot size was about 1/16 in (2 mm) with a minimum slotted surface of 10%. The submersible pumps had a discharge capability between 24 and 111 gpm (130 to 600 m$^3$/day). To monitor the groundwater level outside the excavation area, standpipes were installed all around the perimeter of the diaphragm wall.

## Construction Activities

The diaphragm wall was either 24 in (600 mm) or 32 in (800 mm) in thickness, and was installed to a depth of about 60 ft (18 m) using a Soilmec BH-12 hydraulically-operated clamshell suspended to a Link-Belt 318 crane. A separate 60-ton (55-tonne) service crane was used for the rebar cage and concreting lifting and placement operations. Bentonite slurry was used to provide trench stability during the excavation. Quality control was performed daily on the bentonite slurry and the concrete used in the diaphragm walls. Nondestructive integrity testing using the crosshole sonic logging (CSL) method was performed through access tubes (four tubes attached to the rebar cage per panel) on about 33% of the "regular" panels and on all "irregular" panels (i.e., corners and multiple panels).

The construction of the reinforced concrete diaphragm walls was sequenced using primary panels and secondary panels, where the excavation for a secondary panel could commence after the adjacent primary panels had set but not fully cured. Steel sheet pile end stops were installed at either end of a primary panel after its



**Soilmec BH12 hydraulic clamshell**

excavation was completed and before the installation of the steel rebar cage. These end stops were used to create a nonlinear contact joint between adjacent panels to increase the shear resistance of the joint. The end stops were extracted after placement of the concrete in a primary panel.

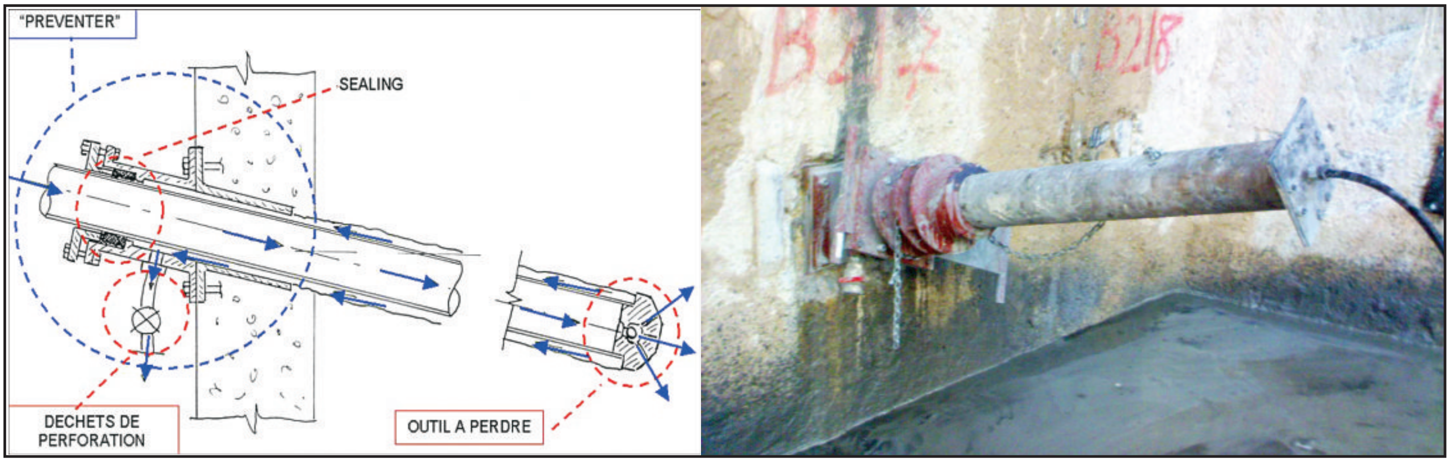
Depending on the depth of excavation in a given area, two or three levels of ground anchors (with either three or four strand tendons) were installed to provide the required lateral support to the wall. Two Soilmec SM 405/8 micro drilling rigs were used to install the ground anchors using the cased rotary drilling method, which were constructed in accordance with the European code EN 1537, *Execution of Special Geotechnical Work - Ground Anchors.* To avoid high water inflow through the wall perforations at the deepest anchor level, preventer equipment was used directly to the anchorage installed in the diaphragm wall. After placement of the anchor tendon, grout was injected in a single stage at low pressure (maximum net pressure of 72 to 87 psi [5 to 6 bars]) using a dedicated grouting pipe.

The dewatering wells and monitoring wells were installed following the construction of the diaphragm walls. After each hole was drilled, it was flushed out with clean water to remove any loose material, and then the well screen, temporary casing, and filter material were installed. The temporary casing was removed as the filter material was placed. Each well was pumped or airlifted for a minimum of 1 hour or until the discharge water was free of drilling mud and/or fines (admissible sand content was less than 100 ppm).

A 6 in (152 mm) diameter electric submersible pump was installed and suspended in each well with the pump inlet facing the base of the well. The discharge take-offs were installed at each well location and the main take-off was installed around the perimeter of the excavation. The take-offs were connected to each other using a flexible pipe. A sedimentation tank was not necessary. Discharge of the groundwater averaged approximately 2,642,000 gal/day (10,000 $m^3$/day) for one year.

Steel reinforcement for diaphragm wall


Installation of ground anchors




**Equipment to prevent water inflow through wall perforations**

Groundwater within excavation (between dewatering stages)

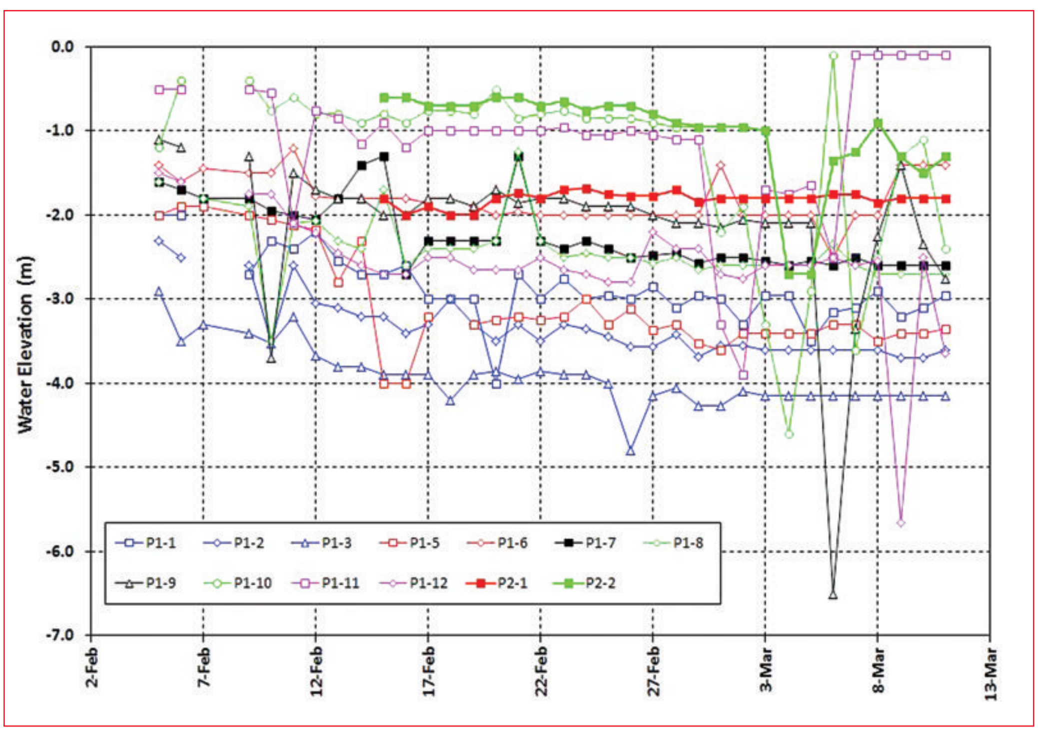
Conditions within excavation with pumps in operation

To monitor the groundwater level outside of the excavation, 14 observation standpipes were installed. During dewatering, the actual discharge flow rate was continuously monitored, and the results of the monitoring confirmed the assumed design flow rates. When the construction of the basement was completed, approximately one year after the installation of the pumps, the pumps were extracted and wells were grouted using cement mortar.

## Conclusions

A total of about 78,000 sq ft (7,250 sq m) of diaphragm wall was completed in 66 working days (11 weeks) using one equipment set up, resulting in an average production of about 1,185 sq ft/day (110 sq m/day) and a peak production of about 2,150 sq ft/day (200 sq m/day). Following the completion of the diaphragm wall installation, 36 dewatering wells were installed in 15 working days. The soil mass excavation (about 196,200 cu yd [150,000 cu m] in volume) and the installation of 575 ground anchors (total length of about 33,465 ft [10,200 m]) were completed in 66 working days using two drilling rigs, resulting in an average production of 26 ground anchors per week per rig.

This was a real teamwork project. Beginning during the design stage through the completion of the soil mass excavation, all of the issues were approached and rapidly resolved with the involvement of the engineer, main contractor and foundations specialist. A significant time savings resulted during the ground anchor construction phase through the excellent cooperation between the main contractor (responsible for the soil mass excavation) and the specialty contractor (installing the ground anchors). The dewatering design was particularly challenging since the dewatering design is typically managed by the main contractor, but was delegated to the specialty contractor on this project. The link between the dewatering system and the design of the diaphragm wall provided the opportunity to deliver an optimized and economical solution to the engineer and owner.




Groundwater level monitoring during 3 month period of excavation and anchor installation

## Acknowledgements

The authors wish to thank our consulting firm Studio Sintesi and the professional team of Man Enterprise S.A.L. and Dar Al-Handasah Engineering for their cooperation with the design and construction of the project.

*Claudio Asioli, M.Sc.C.E., is the director of design, research and development at Trevi S.p.A. His areas of expertise are designing and managing the execution of bored pile foundations, shoring using diaphragm and secant pile walls, ground anchors, dewatering systems, and soil improvement (jet grouting, soil mixing, low mobility grouting and artificial soil freezing).*