

Chemical Grouting for Water Control in Nearshore Poorly Cemented Sands

L. Liersch¹, G. Abdo², W. Choucair³, S. Sadek, M. ASCE⁴

¹ Project Designer, Technical office, Bauer Spezialtiefbau GmbH

² Director, Bauer Lebanon Foundation Specialists sarl

³ Principal, Turba ltd., Geotechnical and Environmental Consultants

⁴ Professor, Dept. of Civil and Environmental Engineering, American University of Beirut

ABSTRACT: The construction of a new beach resort along the southern shoreline of the city of Beirut comprised a 29m deep excavation, which extended about 6m below sea level. The size of the lot (16.000m²), its direct proximity to the sea and the nature of the subsurface materials presented an unusual and extremely challenging water control context. Further constraints regarding local materials, expertise, and practically and economically pumpable water quantities, compounded the difficulties. This paper presents the results of the subsurface investigation campaigns and the solutions adopted which included secant pile cutoff walls and a 2m thick chemically grouted base plug. The problems faced, given the nature and variability of subsurface materials, and lessons learned in relation to grouting strategies and solutions, are presented along with actual trial and production pumping test results.

INTRODUCTION

One of the largest developments currently under construction in the post-war Beirut, Lebanon involves the rebirth of one of the city's iconic sea resorts. The Summerland Hotel was for most of the nineteen seventies and eighties, the symbol and center of Beirut's touristic, cultural and social activities. The old hotel and resort have since suffered through the ravages of time and wars. It has now been demolished and is scheduled to be rebuilt on the original grounds with significant improvements, expansion and ground reclamation works.

The project includes a number of challenging engineering aspects. The tearing down of the old structures involved exposing cut sections up to 29m in height which required shoring. In some instances, parts of the old basement walls slabs and columns were retained and incorporated in the shoring provisions. The final grades of the excavation are at 6m below mean sea level. Figure 1 shows a plan of the new development with an East-West cross-section. It presents some of the elements of the shoring and water control solutions envisaged. This paper will focus on the water control measures, specifically on the chemically grouted plug designed to reduce water ingress from the considerable site area to pumpable limits set at < 1000 m³/hr.

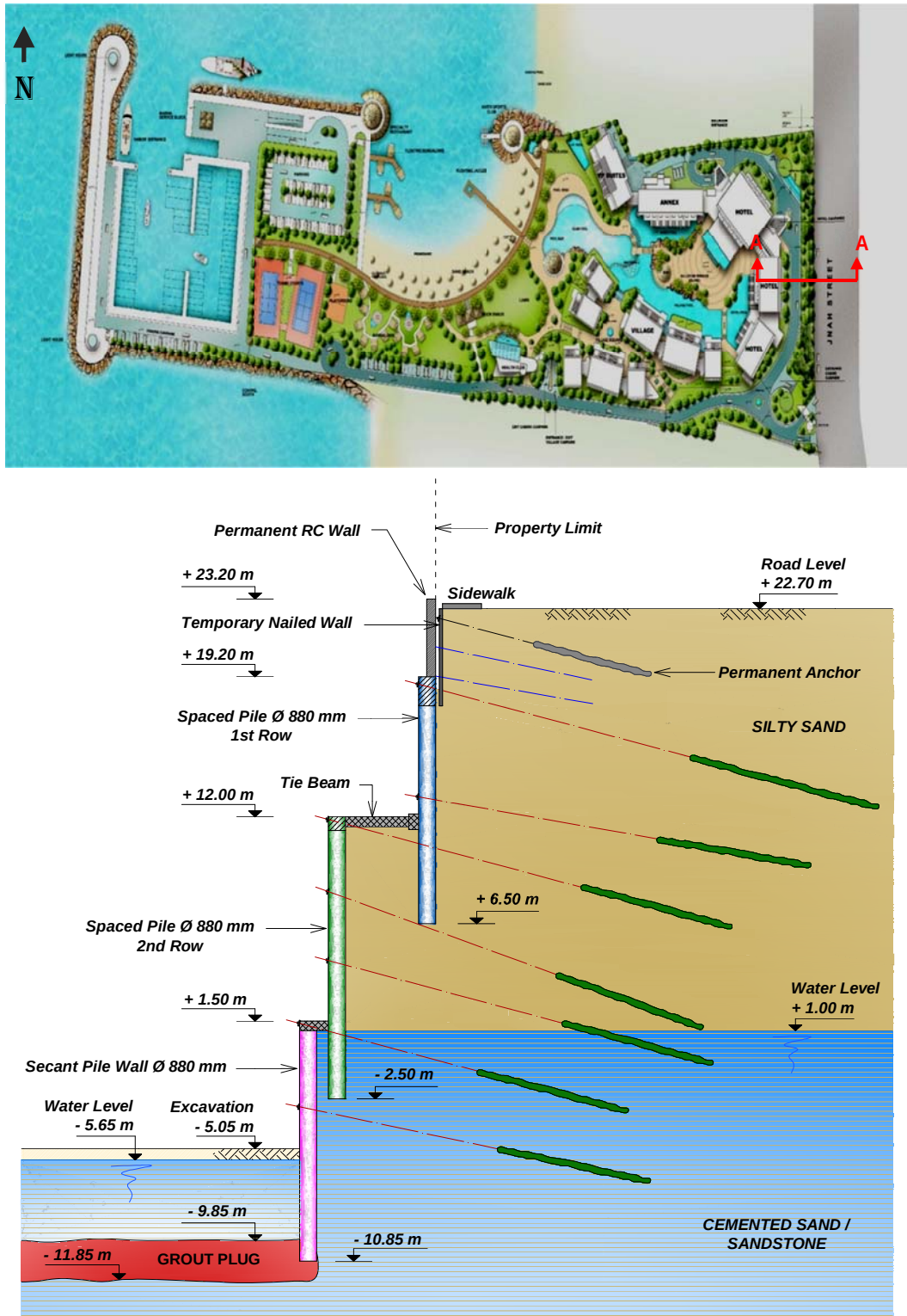


FIG.1 Site plan and cross-section (A-A) along the eastern site boundary.

SUBSURFACE CHARACTERIZATION

In preparation for the project, a detailed soil investigation program was designed and implemented by TURBA Engineers. The subsurface exploration consisted of advancing 18 boreholes to depths varying 20 to 30.5m below existing grades across the site and included besides laboratory testing and characterization:

- Continuous coring with detailed description and logging of rock properties.
- Monitoring of drilling parameters (rate of penetration, flushing water pressure, thrust, torque).
- In-hole constant head (Lefranc) and packer (Lugeon) water permeability tests (BS 5930-199).
- Two pumping tests with piezometer monitoring for field-scale permeability estimation.

The findings of the initial geotechnical investigation indicated that the subsurface materials in the depths explored consisted of cemented sands and/or weathered sandstone. This stratum is also characterized as follows:

- The sandstone is highly fractured with core recoveries of 0 to 60 % (in line with varying levels of cementation-typical of the deposits in the area).
- No clear cavities and/or fractures were evidenced during coring/drilling.
- The field pumping tests indicated an estimated subsurface “average” permeability of $\sim 10^{-3}$ m/sec.
- Given possible anisotropic characteristics of the stratum, the tests conducted do not provide clear indications of vertical vs. horizontal permeabilities.
- Lugeon tests yielded values between 105 and 170. These lugeon numbers are typical of fractured sandstone and indicate that the material is groutable/treatable with bentonite-cement grouts.
- The results of the logs and tests indicated the presence of significant variation across the stratum, with alternating highly pervious layers with less permeable horizons. This observation will impact significantly the implementation of the grouted plug.

Based on the initial site exploration campaign, the geotechnical specialty contractor (BAUER) and the consultant established the following as part of a baseline for the design of the water control scheme/grouted plug:

- The subsoil would be considered as a mixture of weak sandstone and sand.
- The maximum acceptable post-treatment permeability of the grouted plug was set at 2×10^{-6} m/sec.
- Where present, Sandy zones/horizons (slightly silty, fine sands) may only be treatable with chemical grouting. The considerable presence of fine sands ($D_{15} < 0.3\text{mm}$) precludes the use of microfine cement grouts.
- Open fractures and/or cavities can be treated with gravity grouting using a bentonite-cement grout.
- A second-phase, more detailed/targeted soil investigation would be needed.
- Field testing/proof of any grouting scheme/design in tests quadrant or pits prior to full field implementation.

Extended Soil Investigation Campaign and Laboratory Testing

In order to further explore and establish the bounds of the permeability characteristics of the layered/mixed material, an approach involving laboratory testing on retrieved cores in areas representative of high recovery, best quality zones, along with empirical estimates using worst-case conservative estimates, was adopted.

The “best case scenario/quality cores” were taken from an area with > 80% recovery and tested for water flow characteristics in a triaxial permeability cell, under gradients of 5 and 10 respectively. The results of these tests indicated permeability values of 1×10^{-6} m/sec to 5×10^{-6} m/sec.

Conservative estimates were obtained using the sand grain size distribution curves and the Hazen D_{10} correlation (Hazen, 1911). This approach yielded k values of 5×10^{-4} m/sec to 7×10^{-4} m/sec. In addition, packer field permeability tests in an area of the site showing recovery of 20% indicated values of permeability ranging from 5×10^{-4} m/sec to 1×10^{-3} m/sec. Additional permeability testing results for areas with different soil composition confirmed the initial values of approx. 3×10^{-4} m/sec to 1×10^{-3} m/sec recommended in the initial soil investigation report.

Figures 2 to 4 show the degree of variability from three boreholes in the zone/depth of the intended grouted plug (marked by the red lines). Based on the results of the initial investigation, the quality of the material within this zone shows significant variability. It indicated that a well-cemented sandstone could be expected at the plug depth in the central site area, with poorly cemented to uncemented materials closer to the waterfront to the west. The additional complementary exploration campaign and tests confirmed the general findings of the original site investigation and allowed for a more refined and comprehensive assessment of the subsurface strata.



Figure 2. Borehole SI-04 Elev. -7.5m to -12.0 m MSL (ref. Fig. 5 site location)



Figure 3. Borehole SI-06 Elev. -7.5m to -12.0 m MSL (ref. Fig. 5 site location)



Figure 4. Borehole SI-08 Elev. -7.5m to -12.0 m MSL (ref. Fig. 5 site location)

FIELD GROUTING TESTS AND TRIAL TEST PIT

Given the somewhat limited experience and published literature on grouting for water control in weakly cemented/fractured sandstones and sand mixed strata, all parties concerned realized very early on that any adopted method would have to be field test/proven in a contained location prior to implementation on a site scale. As such, a grout test location/zone GT-3, along with a “test pit” were selected for the application and testing of the bentonite-cement grout and soft gel grout, respectively. The bentonite cement grout consisted of the following mix: 300kg/m³ cement; 50kg/m³ bentonite; 890 l water (decantation at 60' <10%; Marsh time 30”-32”). The Soft gel grout consisted of the following mix: 220kg/m³ sodium silicate; 9.4 kg/m³ sodium aluminate; 835 l (initial viscosity ~4CP; gelling time 45-60mn).

Figure 5 present the layout of the western section of the site along with the location of boreholes SI-4, 6 and 8, grouting trial GT-3 and large scale test pit trial area.

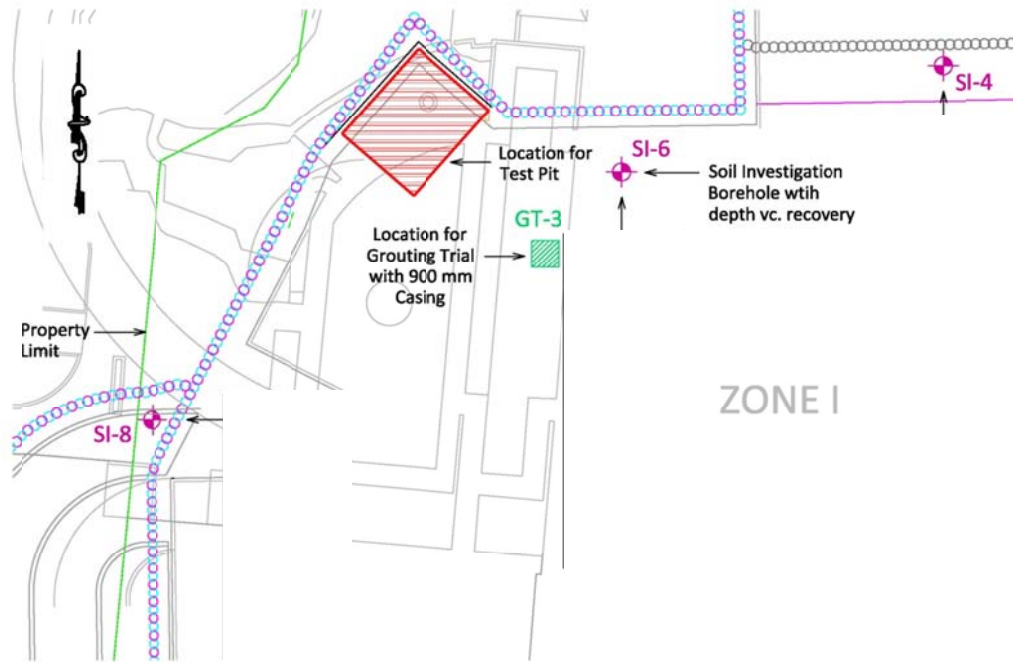


Figure 5. Location of Boreholes, Grout Trial and Test Pit Locations

The grouting test program consisted of two stages or phases. The first phase involved several small field tests to get some measure and indication of the grouting parameters and effective practically applicable materials. The second phase involved a relatively large scale implementation in a test pit, to revise/confirm the permeability of the whole system, as well as establishing minimum required grout quantities. The phase-1 small field tests included several squared grouting grids with spacings of 3.0m and 2.12m which were installed and grouted with bentonite-cement and chemical grouts, respectively. These initial spacings were adopted based on the contractor’s in-house experience. A typical

test installation is shown in Figure 6. The phase-1 treated trial zones were tested by installing a 900 mm temporary steel casing with a cemented toe plug, cored with a 840 mm drilling bucket.

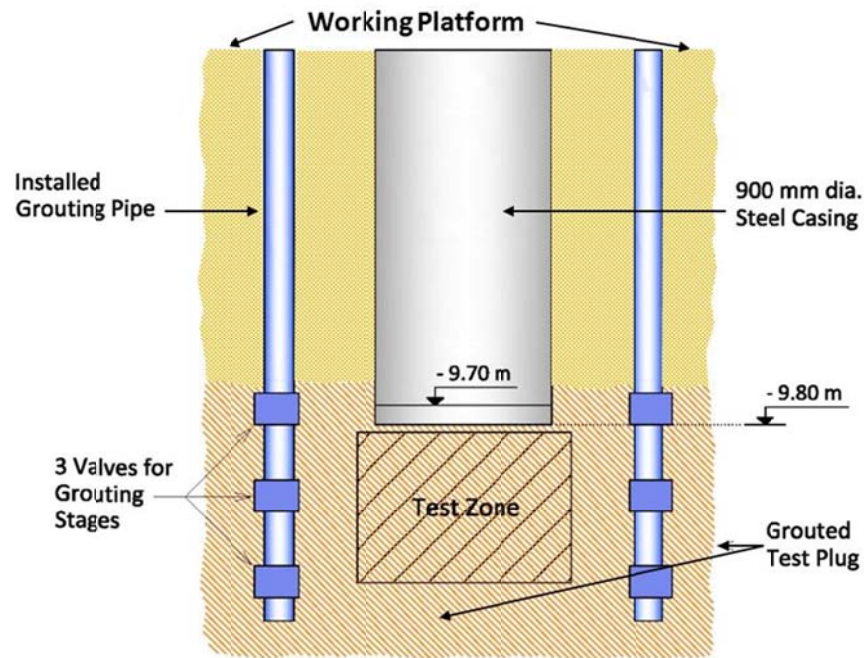


Figure 6. General installation scheme for phase-1 grouting trials

Lefranc permeability tests were executed before grouting. Several falling head and rising head tests were performed post treatment in the test zone. Furthermore, constant head tests with flow rate measurements were conducted on the same. Several areas were tested with packer water pressure tests before and after treatment. Packer tests were performed in independent test boreholes in the treated zone. Results are summarized in Table 1 below.

Table 1: Comparison of testing parameter for grouting trials

Grid	Grout Type	Rock Recovery	Permeability		Improvement Ratio
			before Treatment	after Treatment	
		(%)	(m/sec)	(m/sec)	before/after
3.0 x 3.0	Cement	20	1.5×10^{-3}	1.5×10^{-3}	1
3.0 x 3.0	Cement	70	1×10^{-3}	1×10^{-3}	1
2.12 x 2.12	Cement/Soft Gel	20	5×10^{-4} to 1×10^{-3}	4×10^{-5} to 2×10^{-4}	5 to 10
2.12 x 2.12	Cement/Soft Gel	7	5×10^{-4} to 1×10^{-3}	1×10^{-5} to 4×10^{-4}	10 to 20

The initial phase-1 test results presented in Table 1 clearly indicated that effective treatment could only be achieved through the application of chemical (soft gel) grout. Still, the post treatment permeabilities were above the maximum contractual specification of $k_{plug} < 2 \times 10^{-6}$ m/sec. Furthermore, at this stage

uncertainties regarding the testing procedure and grout quantities were still not completely resolved.

It is then that an additional phase was added to the field trial stage. This further trial was designed and approved to get more reliable treatment and permeability results. The “phase-3” field trial consisted of the construction of a test pit with an area of 100 m². The pit area was chosen along the seaside, near bore hole SI-6 where the core recovery was less than 20 % indicating a problematic zone. The objective of the trial pit field test was to establish the optimal grout quantities required to achieve meet the post treatment specifications. The pit/trial area itself would be part of the constructed site area, however for the purpose of the test and in order to provide lateral containment, secant pile cutoff walls were constructed at its boundaries (Figure 5).

Grouting points were installed in the trial pit using a squared grid pattern 2.12m by 2.12 m. Each grouting point was equipped with three injection valves positioned within the required plug zone at elevations of -9.45m, -10.15m and -10.8m below mean sea level, respectively. Initial grout estimations were done considering the following grouting sequences and quantities:

- 1st sequence: Upper valve 200 or 250 l alternated cement/soft-gel grout
Middle valve 800 or 1300 l alternated soft-gel grout
Bottom valve 200 or 250 l alternated cement/soft-gel grout
Note: for the above that the volumes of grout injected through each of the valves varied according to the estimated radius cover of the injection point in reference to its position in the grid.
Pressures: Opening pressure 12 to 40 bar
 Grouting pressure 10 to 25 bar
Flow rate: 8 to 10 l/min
- 2nd sequence: Drilling of additional holes and re-grouting of edges with 400 l/point.
- 3rd sequence: Re-grouting of points with 1500 l/point.

The following grout mixes were used:

- Sleeve and Bentonite-Cement Grout: 50 kg Bentonite+300 kg Cement 42.5N+890 l water.
- Soft Gel Grout: 220 kg Sodium Silicate SN40+9.4 kg Hardener Sodiumaluminate SA+835 l water.

The approach adopted as part of the first sequence of grouting was based on the desirability of using the upper and lower valves to seal macro-cracks and relatively large fissures with bentonite-cement grout, while the valve at the center would then be used to complete the plugging of fine sand zones.

After every sequence of grouting a pumping test was conducted in wells executed for the purpose. The dewatering flow rate and resulting water level drop recorded and evaluated according Figure 7. Table 2 provides a summary review of all the grouting parameters and the evaluation of pumping tests, with achieved system permeability. Based on the measured and evaluated permeability values estimates/predictions of the flow rates required for the whole site to reach the target construction WT level were made.

Table 2: Evaluation pumping tests with estimated permeability & flow rates

Sequence No.	Grout quantity	Estimated pumping rate	Estimated pump rate total site	Permeability achieved
1st	333 l/m ²	47 l/sec/1000m ²	approx. 2700 m ³ /h	9.4*10 ⁻⁶ m/sec
2nd	333 l/m ²	42 l/sec/1000m ²	approx. 2400 m ³ /h	6.9*10 ⁻⁶ m/sec
3rd	670 l/m ²	21 l/sec/1000m ²	approx. 1150 m ³ /h	3.0*10 ⁻⁶ m/sec

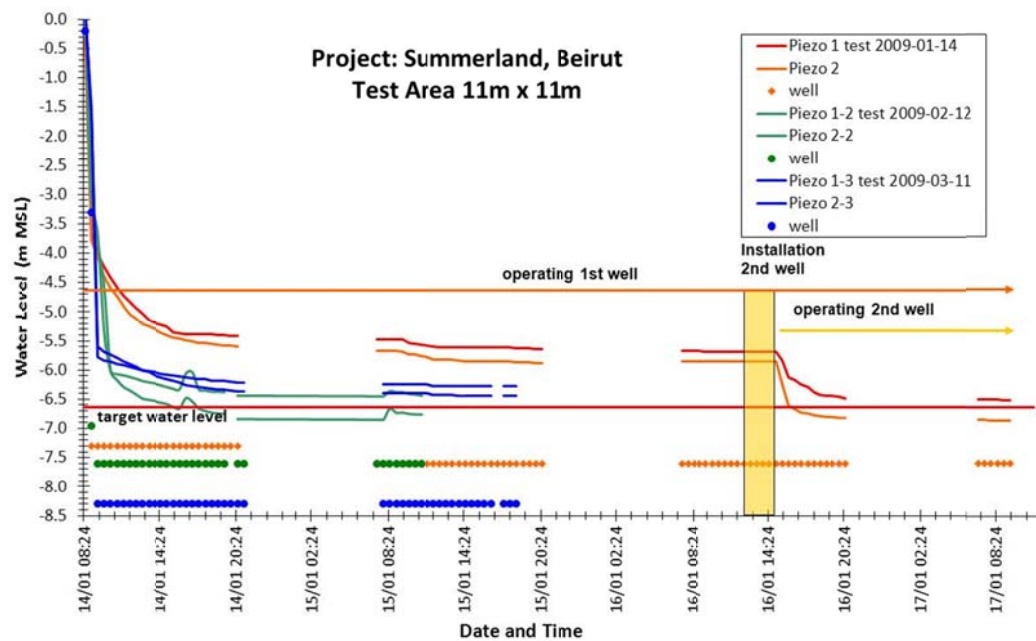


Figure 7. Graphical evaluation of WL readings at test pit during pumping tests

Reviewing the grout quantities pumped at the test pit, grout plug has to be considered as an integral layer without gaps. Furthermore, a long construction period of 2 to 3 years had to be considered. Balancing the results of test pit as well as the project constraints (durability, time schedule), a triangular grid of 2.0 by 2.0 m with the following grout quantities was adopted for implementation at field scale: Top valve-250 l; Middle valve-1900 l; Bottom valve-250 l

EXECUTION OF GROUTING PLUG ACROSS SITE

In preparation for final production, all tests had been executed with the same equipment, personnel and the same grouting materials. All activities were executed with local available materials and grouting equipment. Batching was done manually and mixing batch wise. All grouting treatment was done with hydraulic piston pumps and manual documentation of flow rate, pressure and total grout quantity. Already well experienced grouting site staff had to be introduced to a new technique with chemical grouting and trained during the tests.

Drilling was executed using UBW 08 and Klemm KR806 single rods with water flushing and sleeve mix filling. Grouting pipes were prepared on site and installed into the bentonite-cement slurry stabilized holes. Flushing material, progress and unusual occurrences were manually recorded by the operators. Approximately 7 to 14 days after setting, points were grouted with 10 piston pumps KP60 using manual flow meters and pressure gages and manual recording.

...Figure 8 through 11 provide views of the site at different stages of the works.



Figure 8: Equipment at working platform Figure 9: Drilling of grouting holes



Figure 10: Final excav. level Landside Figure 11: Final excav. Sea side

Grouting Documentation

Drilling and grouting parameters (drilling diagraphy, grout volumes, pressures) were monitored as part of an inspection and testing plan and evaluated over the whole area of construction. As such, the filled sleeve mix and the recovered soil quantities, as well as the possible grout intake for each grouting point were reviewed and evaluated. Using the pressure development during grouting, an overall interpretation of the subsurface soil characteristics was developed in line with the site investigation data and presented in Figure 12. Pressure category A (pure permeation) could be detected in pure sand areas, followed by pressure category B (overburden pressure) in cemented sand and grouting pressures category C (three- dimensional stress case) in the sandstone.

Confirmation Falling Head Tests

The performance of the grouting works was also verified by falling head tests performed in boreholes keying into the plug. A total of 14 boreholes 133/152mm in diameter were drilled down to elevation -10.85 m MSL, i.e. to the theoretical middle of the plug. The locations of the confirmation tests were selected to cover the whole site area. Perforated pipes with 55mm interior diameter were then installed into the borehole and the orifice filled with filter material. The casing was then retrieved up to the theoretical top of the plug and the transition zone at the elevation of the theoretical top of plug covered first with a sand layer as protection and sealed afterwards with a highly viscous bentonite cement mix with limited flowability. The falling head tests were then performed and evaluated according to EM1110-2-1901.

The majority of the confirmation permeability tests yielded results at or below the target specification, with ratios of improvement (pre/post treatment permeability) ranging from 3 to 30.

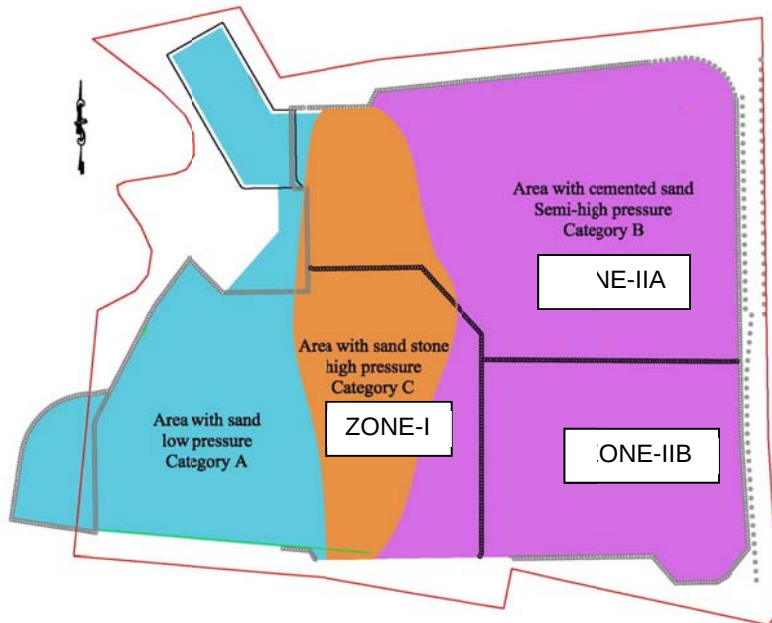


Figure 12: Evaluation of grouting pressure according grouting areas

Final Pumping Tests and Dewatering Works

After completing the piling and grouting activities in each of the zones indicated in Figure 12, pumping tests were conducted followed by the initiation of “permanent” dewatering of the area. Results of the dewatering works for the combined area of the site show an average quantity of approximately 950 m³/h. This is very close to the estimated values based on a plug permeability of 2×10^{-6} m/sec, set in the specifications. Considering the lowering of the water levels and final water levels inside the pit as well as the distribution of pumps over the whole site, it had been determined that the sand, and cemented-sand areas (category A and B, Figure 12) yielded significant lower water ingress compared to the sand stone area.

CONCLUSIONS

The introduction of a new technique to the local practice and the installation of a soft-gel plug in a heterogeneous and highly variable substratum of sands, weakly cemented sands and sandstone presented a set of very interesting and challenging problems. These may be summarized in the following:

- Soil Characteristics:
 - Heterogeneous and high spatial and depth variability
 - Very irregular groutability/permeability/compressive strength
- Technical and Contractual Constraints:
 - Adjustment of the method according local know-how/availability with the associated cost and time schedule implications
 - Soft gel prepared from a Sodium aluminate powder and pumped using standard hydraulic piston pumps
 - All documentation was manual; no digital logs for plug grouting

Lessons learned from this particular site suggest that weak sandstone with a recovery of more than about 60% was not groutable using soft-gel grouts due to low intake and a resulting lack of reduction of permeability. However, cemented sands with a recovery below ~40% were groutable using soft-gel grouts. The intake was high enough to result in sufficient sealing. The achieved permeability results for the treated materials were on the order of 5×10^{-6} m/sec. Furthermore, in cases involving such large sites with significant variability, the original site investigation complementary testing and proof of method/concept evaluation are critical for any grouting program/design. The reliance of the 1:1 scale pit implementation allowed the contractor and consultants to test/calibrate the methodology, materials and mixes prior to site-scale implementation. This and other factors noted in this paper were the key for the successful completion of the project.

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