Failure of a Hybrid Flexible Shoring System for a 30m Excavation: Exploration of Causes and Remedial Measures

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ABSTRACT

One of the largest development projects in the City of Beirut involved the excavation of an area of 14,000m^2 down to a depth of 30m below street level. The site is in an urban area and bound by major traffic arteries and a multi-storey office building. The shoring approach adopted for the excavation boundary walls consisted of a hybrid flexible system with multiple rows of pre-stressed anchors, followed by rows of passive nails at varying spacings and lengths. A reinforced shotcrete facing was provided across the full depth of the excavation. Upon reaching the final excavation grades across the whole site, significant movements were recorded along one of the site boundaries (approximately 120m long) adjacent to the main traffic artery. These deformations at the face were accompanied by longitudinal cracks up to 20m away from the excavation boundary along the main road, with differential downward movements on the order of 5 to 10 centimeters. The pattern of deformation and location of cracks suggested an impending deep seated failure. This resulted in the closure of all adjacent roads to traffic and emergency backfilling measures to shore the compromised wall. At this stage third party forensic failure analyses were initiated in which we were involved.

In this paper, the background related to site-specific sub-surface characterization efforts, along with design choices and options adopted are be presented and discussed. Post-movement analyses and monitoring results are used to identify the reasons behind the failure. Finally, remedial measures implemented are described and discussed in detail along with lessons learned.

INTRODUCTION

The development project subject of this case study extends over an area ~14,000 m^2 and includes a thirty one storey tower rising above apportion of the with an commercial center extending over the balance of the surface area and across four basement levels. The remaining basement levels were to serve as parking and service areas.

The project is bounded to the West and North by major traffic arteries and to the East by a 5 storey office building approximately 12m away from the site boundary. The southern site boundary is defined by an old two storey structure and an empty lot. The approximate depth of the excavation works associated with the MCC project is 28m extending over the whole site.

The shoring system adopted by the excavation contractor was a hybrid flexible system consisting of a combination of pre-stressed active anchors towards the top of the excavation, and passive nails further down, with a reinforced shotcrete layer ~15cms thick over the whole excavation face.

As the excavation was reaching the final grades, significant deformations and displacements were recorded along a significant portion of the shoring system involving a 100m long shored section. The deformations across the facing which were monitored by fixed point survey techniques were on the order of centimeters (exceeding 7cms at some points) and were both horizontal (towards the excavation) and vertical (downwards).

The distress and deformations in the shoring system itself were accompanied by the development of longitudinal cracks in the major road at the western boundary, at distances of ~8m, ~13m and ~ 17m away from the edge of the excavation, with a perceptible differential downward movement across some them.

The relatively rapid development of the deformations in the shoring system and the associated cracks in the road caused great concerns regarding the overall stability of the excavation shoring system and the imminent loss of the main traffic artery adjacent to the site through a “deep-seated” failure involving very large volumes of soil/rock. A schematic plan of the site which shows the mapped cracks (in red) is provided in Fig. 1.

At this point, emergency remedial measures were put into effect. These included the closure of the main road to traffic and the initiation of round the clock backfilling operations along the affected side of the excavation. The backfilling operations proceeded to eventually reach a height of about 20m from the base of the excavation. The cracks along the road were "sealed" to prevent the infiltration of rain waters which could further destabilize the failing mass. The system was stabilized and no further significant movements were observed, either along the portion of the facing which remained exposed (top 8m) nor in the cracks along the road to the west.

The effort described in this paper was initiated to establish the possible causes of the failure of the implemented system and potentially assign liabilities which would cover the loss and cost of remedial works. To that purpose, a new campaign of subsurface reconnaissance was initiated to assess the conditions along the affected section along with other sections of the site. Further, analyses and assessments of the original site exploration campaign and recommendations, shoring system conception, design and implementation were carried out. The results and findings of these efforts are presented in this paper.

SUBSURFACE CONDITIONS

An original site investigation campaign was conducted prior to the start of the works. This effort was aimed at establishing the subsurface conditions for the selection and design of a proper foundation system and to establish strata characteristics to be used in the design of the shoring system. In total 22 boreholes were executed on site and are shown in Fig. 1 as BH-1 through BH-22. The majority of these boreholes ended up being approximately 2m above the final foundation level, due a change in project grades which came after the completion of the exploration works! The in-situ testing component consisted mainly of standard penetration tests (SPT) in both the surficial soil layers and the more competent base materials, local Marls. The SPT results indicated refusal in the Marl and varied significantly in the upper soil strata (N=20 to 80). The geologic context and possible structural features were not mapped and the potential implications of geological structural features (strike and dip of the rock beds) on the excavation works and shoring provisions were not explored.
The stratigraphic model presented in the original report indicated the presence of the following layers:

- Layer 1: Superficial CLAY deposits, stiff to v. stiff.
- Layer 2: MARL, friable to very weak, fractured and weathered.
- Layer 3: LIMESTONE base of "very poor" quality with some cavities.

The suggested design parameters for the above profile (we will refer to this as Profile 1 in this paper) were listed as follows:

- Layer 1: γ=19kN/m^3; c=75 kPa; ϕ=0^o.
- Layer 2/3: γ=20kN/m^3; c=100 kPa; ϕ=30^o

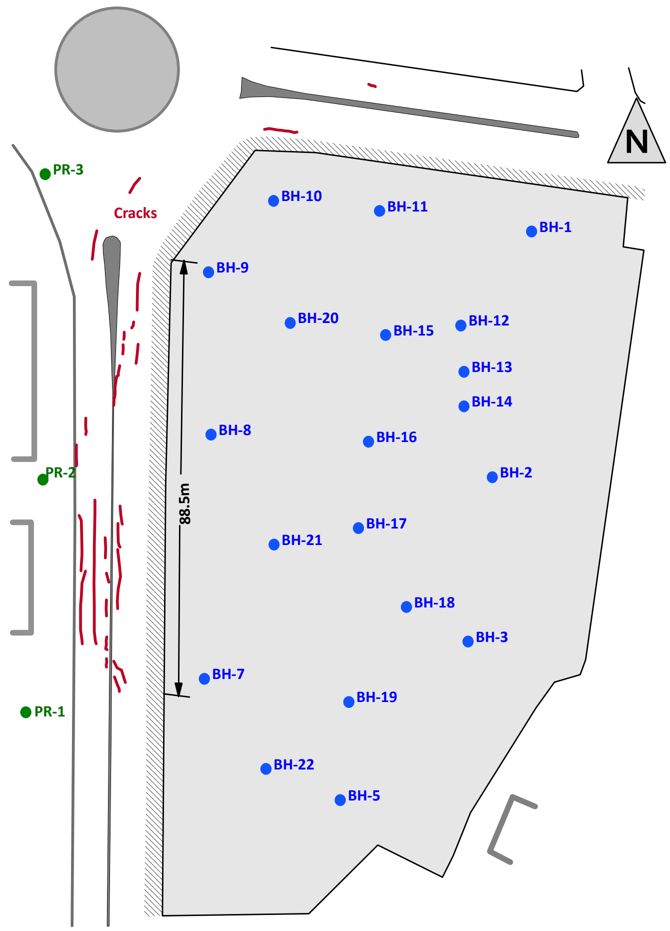

Fig. 1. Plan view of the site showing the cracking in the roads

An additional and closely supervised exploration effort was undertaken following the observed deformations and cracking along the road as part of the failure analysis works. A set of boreholes were advanced around the periphery of the site. Great care was taken in locating the boreholes in order to minimize any interference with the executed shoring system elements, specifically the post-tensioned anchors. The boreholes which fall in the western boundary are indicated as PR1 through PR3 on Fig.1. The new exploration holes included both SPT and pressuremeter measurements, along with laboratory tests for classification and evaluation of the remolded (disturbed) strength characteristics of the strata. The in-situ test results from the second campaign are presented in graphical summary form in Fig. 2.

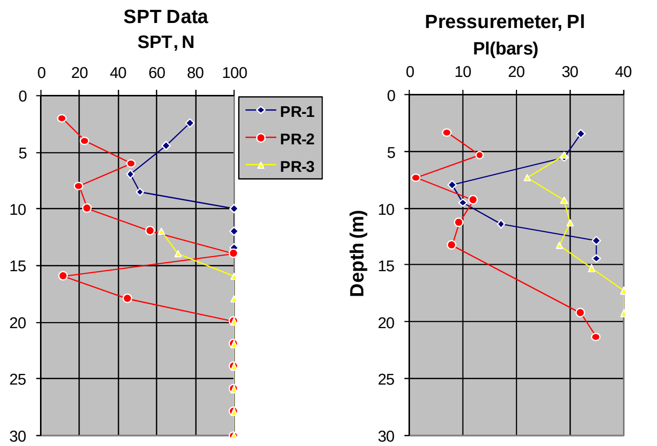

Fig. 2. Field test results from the second investigation effort

A number of significant differences can be identified between the earlier geotechnical investigation and the one carried out in the days following the shoring system distress. The new subsurface model (Profile 2) consists of three "layers" or strata assigned the following engineering characteristics:

- Clayey Sand: γ=19kN/m^3; c=25 kPa; ϕ=25^o.
- Silty Marl/Marl: γ=20kN/m^3; c=50 kPa; ϕ=25^o.
- Marly Limestone/Marl: γ=22kN/m^3; c=75 kPa; ϕ=25^o.

The difference in the strength parameters as reported in the original and second report can be attributed to a number of possible factors which include the likely different thicknesses and varying composition along the tested zones, which is possible given the complexity of the dominant geological unit. Also, the presence of water in the soil/rock block which was a significant factor contributing to the weakening of the susceptible materials.

In summary, it can be established that the subsurface materials encountered were predominantly weak, altered and subject to change within small distances and are potentially affected by changes in water content and unloading.

EVALUATION OF THE SHORING SYSTEM

The shoring system adopted by the contractor is defined in the CLOUTERRE 91 recommendations as a "mixed-structure" shoring system in which active pre-tensioned anchors are combined with passive nails and a reinforced shotcrete facing. The location of the anchors is typically at or near the top of the excavation to limit the movements close to sensitive structures and/or utilities. By their very nature such systems will result in deformations, both vertical and horizontal, along the supported cut, during the excavation stages prior to placement of anchors or nails and after placement of the passive nails, as they strain to reach their working capacities. This is not to say that the choice of this particular shoring approach is not legitimate in the case of the project at hand, provided that applicable norms and codes are closely adhered to, and that design considerations be implemented that account for the complex geological formations and structure and local but significant variabilities in subsurface profiles and material characteristics. Further, it is our opinion that with this particular project, the "case for caution" is strengthened by a number of elements:

- Variable subsurface conditions, with alternating sequences of hard and softer materials, including a "dominant" silty Marl substratum which is particularly vulnerable to changes in confinement and increased moisture levels.
- Very large area of the site (~14,000 m^2) and corresponding shoring perimeter (~450m) combined with very deep excavation levels (~28 m).
- Time pressures. The execution time frame for all shoring and excavation works was set at four months. As a result the necessary time for monitoring and possible implementation of timely remedial measures was not available.

A thorough and comprehensive review of the original designs submitted by the contractor and revisions suggested and/or mandated by the consultant was conducted. The review suggested that on a number of occasions short cuts were taken, changes made and deviations allowed from established norms which when combined, led to a final shoring design which was optimistic and did not meet the "case for caution" suggested earlier, for instance:

- The value of unit skin friction, q_s, along the nail / anchor fixed length in the Marl layer was taken as 350 kPa. This value is considered as high in weathered marls (which dominate the Marl/Marly limestone profile CLOUTERRE 91-Ch.3; T.A.95-p.149; BS8081 and PTI section 6.7.2.4).
- The partial factor of safety adopted for the unit skin

friction Γ_{qs} = 1.5 is lower than the recommended 1.80 standard in case the q_s values are obtained from charts or correlations. The 1.5 value may be used if q_s is determined through field pullout tests which was not the case here. BS8081 even suggests that a factor of safety of 2.0 should be applied on the grout/tendon or bar interface, even for the case of temporary anchors where no serious consequences may result from failure (Section 6.2 Table-2).

- As was indicated in the section on subsurface conditions, the original parameters recommended for use in design did not represent the most critical conditions. They could conceivably represent a higher-bound interpretation of the subsurface conditions. Despite this fact, partial safety factors lower than those recommended by CLOUTERRE 91 were used: Γ_c = 1.15 and Γ_ϕ = 1.0 were used for the cohesion and friction angle terms, respectively.
- The nail "density" or number of nails per square meter of facing was well below the recommended ranges. The executed design called for a spacing of 3m horizontally and 4m vertically (i.e one nail in ~ 12m^2) contrasted with the recommended limit of to 1 nail per ~ 6m^2.
- The vertical spacing between the nails / anchors of about 4m fell outside the recommended norms for free-stand up unsupported heights for construction phases in soil nailed walls and mixed walls. Typical values are on the order of 2 to 3m.

Verification Analyses-Limit Equilibrium (LE)

The executed shoring system design was evaluated at a first stage by using limit equilibrium analyses. The behavior of the system was explored in reference to a target global safety factor sing un-factored soil and material parameters. We considered that an appropriate target global factor of safety for conditions such as the ones relevant in the project under consideration would be FS~1.5. The analysis tool used was the specialized software SLIDE by rocscience. The results of the runs and limit equilibrium analyses conducted using both Profile 1 (original subsurface parameters) and Profile 2 (strata characteristics and parameters as established in the later site investigation effort) are presented in Figs. 3 and 4. The minimum global safety factors obtained for profiles 1 and 2 using non-circular failure surfaces, were 1.209 and 0.921 respectively. These numbers indicate that the original design was not satisfactory even with the subsurface model and associated characteristics used and that the second profile with its relevant the strength parameters indicates failure conditions (FS<1) as observed. It is interesting to note that the mobilized mass indicated by these analyses appear to correlate well with the observed crack patterns and distress.

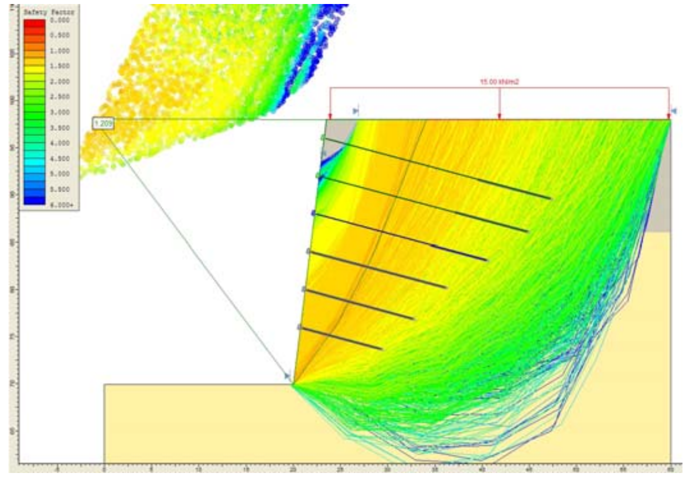

Fig. 3. LE analyses Profile 1. With all surfaces. FS_{min}~1.21.

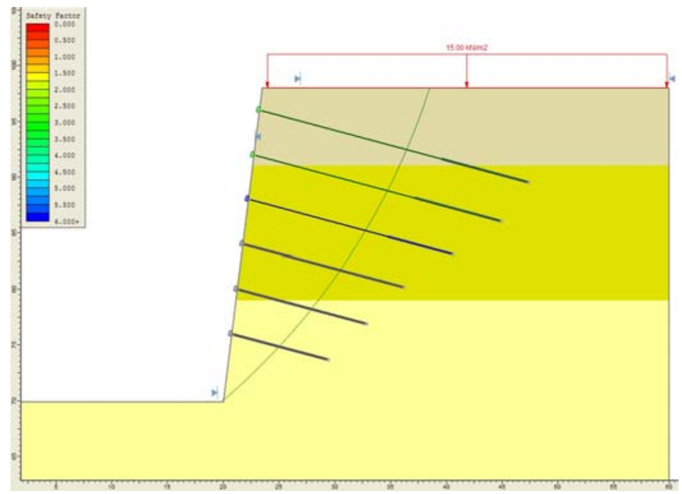

Fig. 4. LE analyses Profile 2. Critical surface FS_{min}~1.21.

Verification Analyses-Finite Element Method (FEM)

The executed excavation and shoring sequence was reproduced in a finite element model of the soils, supports, loads and geometries. The numerical process was conducted in a staged process mirroring the actual excavation and shoring steps. The software PLAXIS was used in these analyses. The soils were modeled using Mohr-Coulomb failure criteria combined with an elasto-plastic model. The FEM analyses were conducted for both profiles 1 and 2 with their corresponding recommended strata characteristics. As may be seen in Figs. 5 and 6 below, the sections below the finite element analyses capture the basics of the performance of the shored wall at all phases of the execution:

- Significant shear stresses and deformations develop at or near the excavation bottom at each stage of excavation.

- The zone between the last row of anchors and first row of nails experiences very high levels of shear stress ratios and the development of plastic points associated with significant deformations. This finding correlates well with the observed distress in the shotcrete facing at that particular level, prior to reaching the bottom of the excavation. This point is discussed further in the monitoring section of this paper.

- As the final excavation step is reached, a large concentration of plastic points develops at the toe of the slope and extends upward and outwards, without developing full "failure conditions" in case Profile 1 model characteristics are used, whereas clear failure patterns are evident for Profile 2. Interestingly, tension points are noted at or near the ground surface, starting at a distance of ~12-15m behind the face of the excavation, which correlates well with the observed cracking and distress patterns shown in Fig. 1.

- Finally, for the case of Profile 2, the outwards horizontal deformation patterns at the facing match (in form) and even to some extent the general magnitudes (on the order of ten cms as a maximum) the patterns observed in the survey monitoring data, a typical sample of which is shown in Fig. 7.

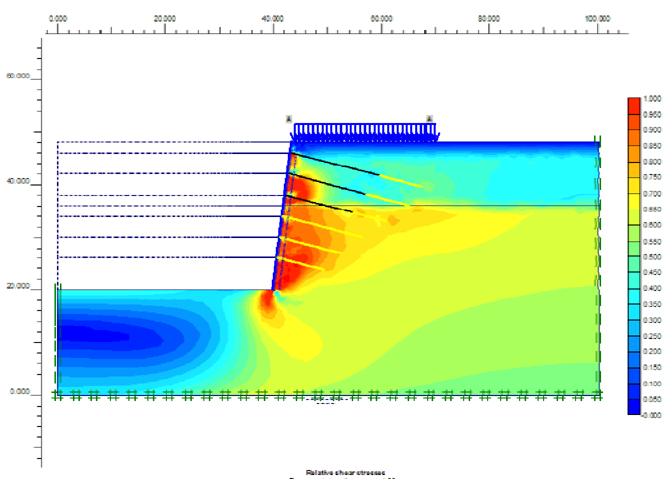

Fig. 6. FEM Profile 1. Relative shear stresses $\tau/\tau_{max(m-c)}$.

Additional FE analyses were run in an attempt to resolve an on-going debate between the consultant and contractor regarding the fact that, had a fourth layer of anchors been included in the system the failure would have been averted. The results suggested that the presence of the additional layer of anchor did not significantly improve the overall stability and maximum deformations for the final excavation step. However, the deformations at the interim steps, particularly in the zone of transition between anchors and nails would have been lower.

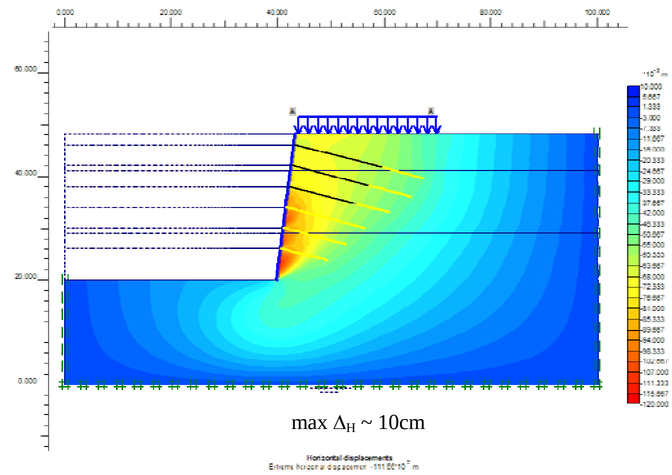

Fig. 7. FEM Profile 2. Horizontal displacement contours.

QUALITY CONTROL AND MONITORING PROVISIONS

A review of the construction methods, materials and quality control provisions revealed no concerns in this respect. The materials used, the placement and construction methods, drilling, construction of nails and anchors, grouting etc. met overall accepted codes and standards of practice. The materials quality control measures which were implemented on site were the following:

- Control tests on the shotcrete.
- Control tests on the grout used in the anchors and nails
- Control tests on tendons and reinforcements.
- Proof tests were carried out on all anchors to 125% of the working load during the pres-stressing phase. A large number of proof tests were taken up to 150% of the working anchor load. No load-unload-time (creep) tests were conducted, and no failure or pullout tests.

Monitoring of Deformations at the Shoring Face

Fixed references were placed at some anchors heads locations and other points along the shored face. These references points were regularly monitored for displacements in the three directions. The relevant reference points along the section which is the subject of this paper are shown on Fig.8. On that same figure blue lines mark the locations which showed significant distress in the shotcrete facing as the works proceeded to the final grades. All the survey data made available to us for the relevant section were analyzed and plotted along various vertical alignments through the section. Typical horizontal (Y direction) deformation measurements along alignment 3 are presented in

Fig.9. It is very important to note that had the monitoring program not been implemented, the problems may have been identified much later, at a time when possibly it would not have been possible to stabilize the failing section through the emergency backfilling program put into effect.

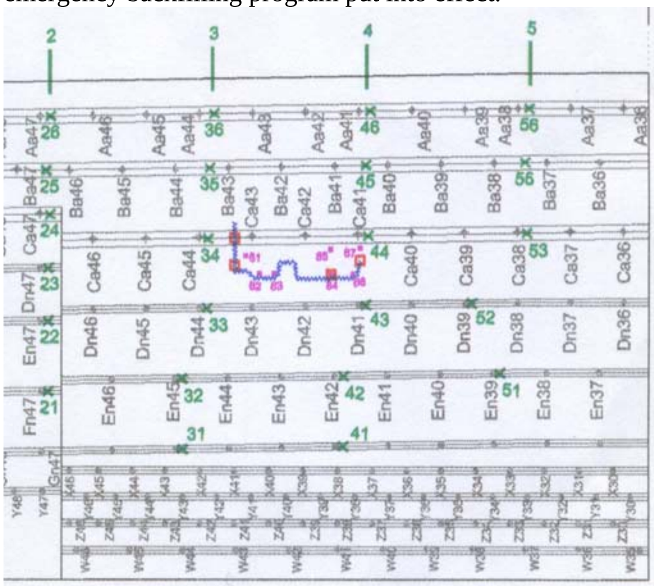

Fig. 8. Partial view of the monitoring points along the section (surveyed points in shown in green)

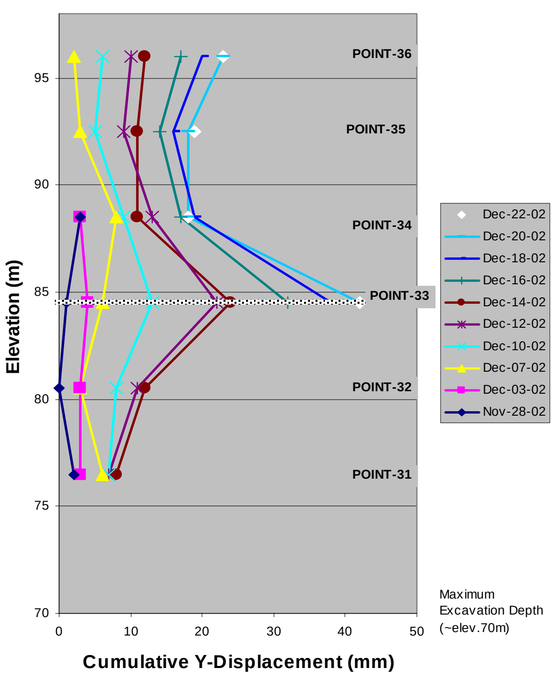

Fig. 9. Horizontal movement along alignment 3. The grey line marks the backfilling elevation obstructing further readings.

It is clear from these figures that the movements started to increase at an alarming rate from approximately December 10 and stabilized significantly after the remedial backfilling was implemented.

In order to investigate the effect if any of the rainfall which fell over the area on the observed patterns of movement, the results for the period spanning from Nov 28 to Dec. 22nd for the point of maximum deformation along alignment 3 (point 33) were plotted and they are presented in Fig. 10 along with the corresponding average rainfall data obtained from the Beirut International Airport weather station.

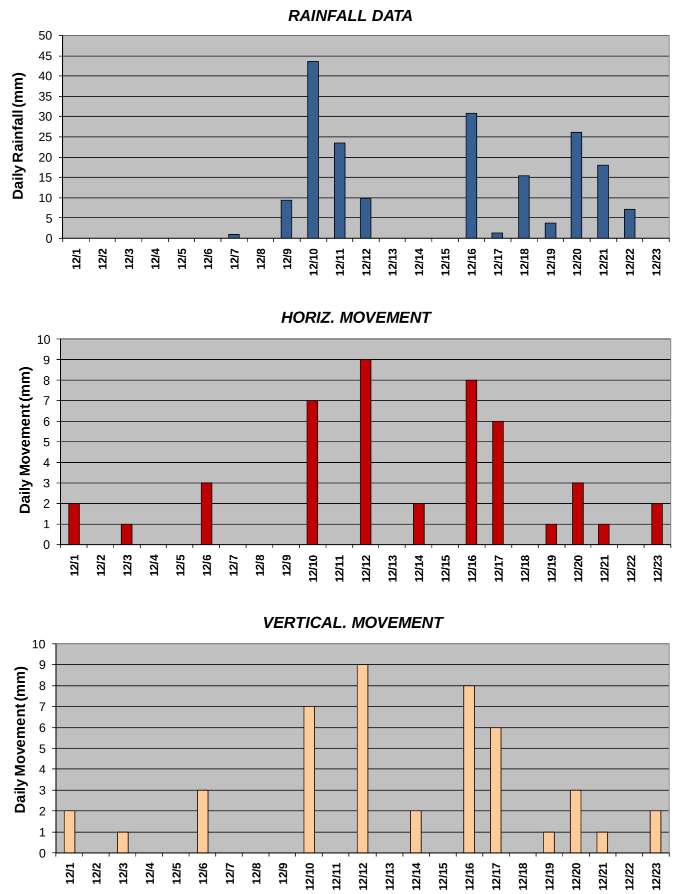

Fig. 10. Incremental horizontal and vertical movements at Point 33 with average rainfall data for same period.

SUMMARY OF FINDINGS-FACTORS CONTRIBUTING TO FAILURE.

Based on the background information presented and a thorough review of the design documentation and execution and monitoring data, along with through analyses of various models and scenarios to assess the executed works and identify the various possible contributors to the movements and distress observed in the shoring system, the following findings may be stressed:

- The initial site investigation report included a reasonable description of the nature and state of the various materials encountered. The report omitted to elaborate on and stress the importance of the geological structure (formation characteristics, bedding, strike and dip).
- The designs produced for the relevant sections were, below the acceptable standards of safety as defined through partial security factors in the CLOUTERRE 91 recommendations and Eurocode-7)
- The sole reliance on limit equilibrium methods of analysis for such shoring works maybe misleading in that local effects and deformations within the retained block are not considered.
- The execution of the works in terms of quality control and monitoring was adequate and may have contributed to avert the development of the full failure along the section.
- Water reaching the soil/rock strata along the distressed side was clearly a contributing factor to the observed failure, as the displacements are well correlated with increased precipitation and more importantly, resulted in measured very low strength in the post-failure exploration boreholes (particularly PR-2)

REMEDIAL WORKS

The scope of this paper does not include a full description or discussion of the various remedial options and alternatives along with the solution finally adopted. Further, works conducted along the other sections of the site to evaluate the stability of the executed shoring works in those areas are not presented in this paper. In short and following a long period which witnessed exhaustive debates and discussions and which involved both legal, financial along with technical considerations the following remedial/repair provisions were implemented and the project successfully completed:

- Backfilling up to ~3m below the existing road level along the failed section. The top of the backfill was set to provide a stable platform for piling rigs and construction equipment (cranes, loaders, etc.)
- A relatively rigid retaining system consisting of a row of "contiguous" cast-in-situ reinforced concrete piles, 0.8m in diameter was constructed through the backfilled material and in front/through the previous system (shotcrete facing, anchors and nails). The piles were extended well below the final excavation grades.
- The pile-wall was then supported by successive rows of pre-stressed anchors as the excavation of the backfilled material in front of the new shoring system proceeded.
- The fixed part of these anchors was designed and placed well beyond the failed zone and was tested with very stringent quality control provisions.
- The designs of the remedial works were based on the parameters established in the latter site exploration effort which were deemed to be more representative of subsurface conditions. In the failed zones, remolded strengths were used.

CONCLUSIONS

In this paper the case of the near total failure of a very large and deep excavation in Beirut was presented. The case is interesting from a number of perspectives: The envisaged excavation depth of 28m was amongst the deepest attempted in such geologic materials in the country; the system adopted by the contractor was a mixed or hybrid flexible system which in fact presented a number of challenges and significant disadvantages given the nature of the subsurface strata, geologic context. This was compounded by a number of optimistic design assumptions which went beyond accepted norms governing the design of such systems.

It follows that a number of very valuable lessons were learned by the local profession from this case and the reliance on such solutions is now more considered, studied and mindful of the potential weaknesses and disadvantages of these solutions in certain specific applications, along with their significant advantages associated primarily with cost and time savings.

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