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Deep Excavations in Difficult Soil Conditions - A Case Study

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Abstract

The structures located at the shores of Marmara Sea in Istanbul are subject to hazards of near fault tectonic events. It is known that North Anatolian Fault (NAF) is located at about 10 kilometers south of shoreline within the Marmara Sea.

A hotel structure is planned to be constructed near the shore at the town of Buyukcekmece. The ground water table is located very close ground surface and the subsoil consists of very weak fine sands and non plastic silts, indicating the possible hazards under earthquake loadings.

The structure is planned to have various basements for various activities, therefore deep excavation has to be done at very difficult conditions having neighboring structures and infrastructures and considering the encountered poor subsoils and high ground water table.

Therefore, a very special construction system and the procedure for retaining structure is designed and implemented. The continuous retaining wall constructed cast-in-situ using slurry technique which was tied back with two levels of pre-stressed anchors is planned to be permitted to carry out the excavation to a level of 12.5m below the surface. However, it was not possible to execute second row of anchors due to very weak non plastic silts encountered during excavation contrary to the results of initial soil investigations performed prior to the design. Instead, a special top down construction technique is developed and implemented below this level in order to overcome the encountered difficulties.

Keywords: *deep excavations, soft soils, top down, anchors, monitoring.*

1 Introduction

A case study is presented for a hotel structure to be constructed in a very unfavorable ground conditions and high seismicity. The structure is planned to have three basements requiring soil excavation to a depth of -12.5m below ground surface with a high GWT. The site is located at the vicinity of North Anatolian Fault (NAF) located within the Marmara Sea near Istanbul. NAF is expected to produce major earthquake of M=7.0+ during near future,(Durgunoglu and et al. 1982).

It is clear that design and construction of both foundation and retaining structure systems require special attention. In this difficult case, an additional complication is occurred due to the misleading results of the initial soil investigations. In fact, it has become compulsory at some stage of the construction to change the supporting system after certain elevation in order to overcome the unforeseen conditions and difficulties. As a result, a top down construction scheme is implemented after certain elevation. Various design and construction details and considerations are discussed and evaluated within the case study.

2 Structure

The hotel structure planned to be constructed to an area of 2450 sqm in plan having three basements below the existing ground surface, Figure 1. Reinforced concrete structure will have eight floors above the ground, as entrance floor + meeting floor + six normal floors. The top elevation of the third basement – parking is given as -12.50 m with reference to +0.00 existing ground surface elevation. The total weight of the superstructure is estimated to be approximately 53,200 tons giving a uniform foundation base pressure of about $\sigma_b = 22$ t/m^2.

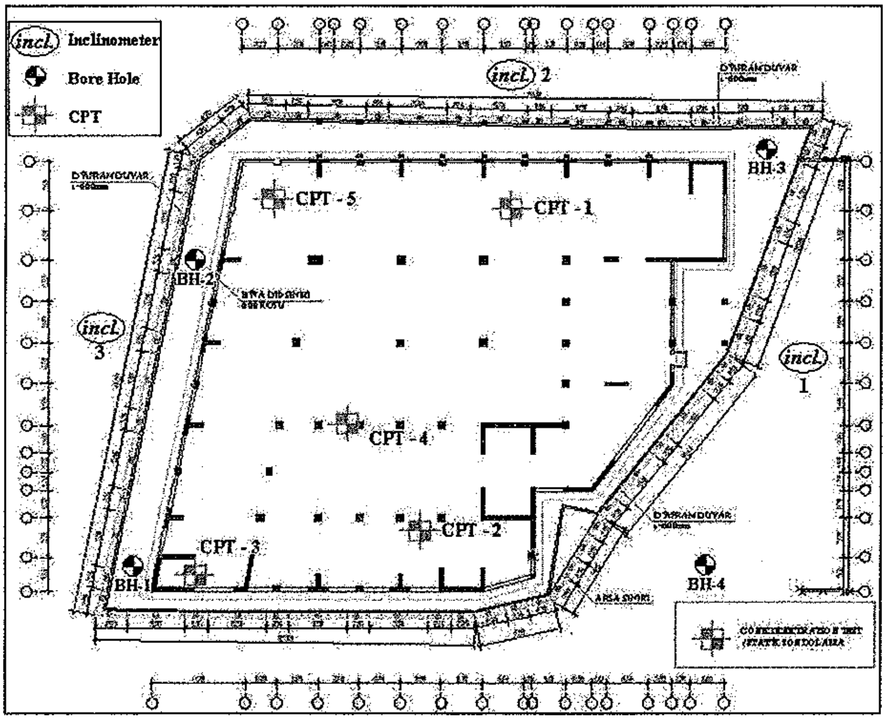

Figure 1. ZETAŞ – CPT / BH / INCL Investigation Site Plan

3 Seismicity

The structures located at the shores of Marmara Sea in Istanbul are subject to hazards of near fault tectonic events. It is known that North Anatolian Fault (NAF) is located below the sea of Marmara at a approximate 10 kilometers south of the shore line. The subject of the case study hotel structure is planned to be constructed at the town of Buyukcekmece which is about 50 kilometers west of center of Istanbul near the shore line.

The ground water table is near the ground surface and the subsoil conditions are saturated weak alluvial deposits leading to various geotechnical hazards under structural loads as well as under earthquake loading. The soil amplification due to thick weak alluvial deposits, together with soil liquefaction of saturated loose sands and strength and stiffness loss of saturated soft fine grained soils are the main concerns under the seismic loading. It is known that the probability of an earthquake of M=7.0+ to occur along the NAF in the Marmara Sea is above 65 percent within 20 to 30 years period of time. Under this tremendously high risk of seismicity civil engineering structures at the area must be designed and constructed with up most care against earthquake loads and hazards.

4 Soil Conditions

The soil conditions are determined by means of four borings to a depth of 29 to 32 m during the planning stage in July 2004. Systematic SPT blow counts N_{60} were measured together with soil sampling. The soil profile determined as a result of this campaign is given in Figure 2. It is seen that loose to medium dense sands with average $N_{60} \approx 15$-20 are located to a depth of 12.0 m below the ground surface. Normally consolidated medium stiff to stiff clays are encountered below the sand layer. The GWT is indicated at a level of approximately -5.5 m below the ground surface.

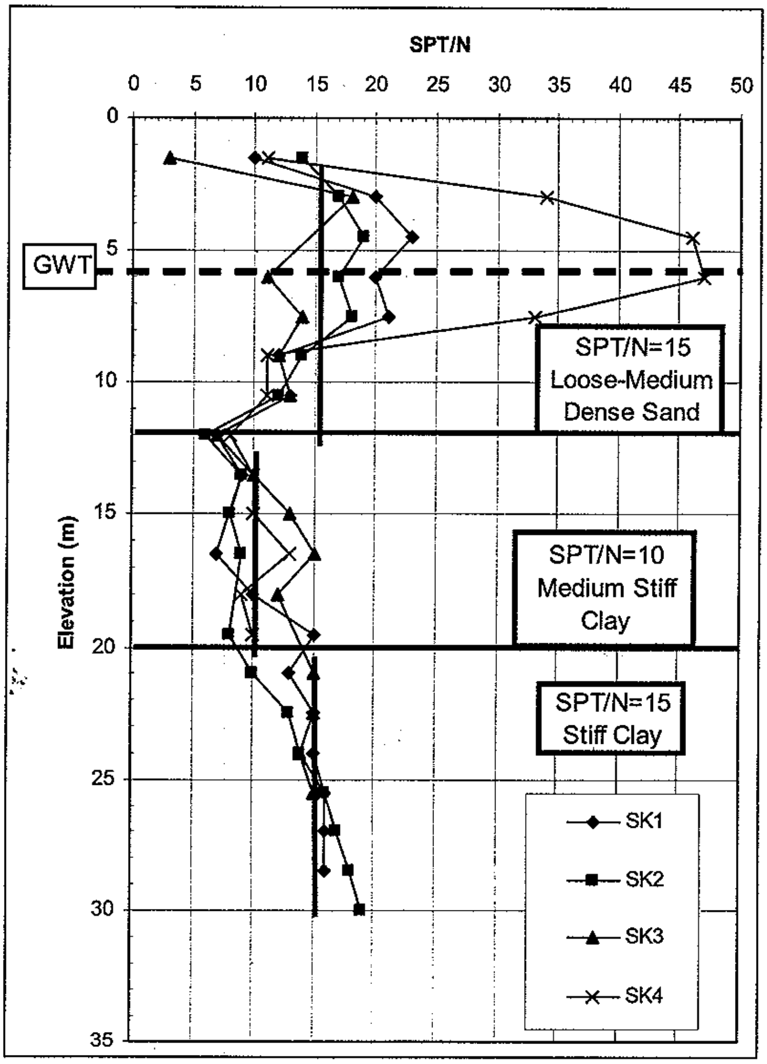

Figure 2. Initial Borings - Change of SPT/N values with respect to depth.

During the excavation of slurry diaphragm walls of 60 cm thick and 17.5 m in length for the tie back retaining system, it was seen that; the GWT is higher and subsoil conditions are different and weaker than the initially determined ones as a result of first geotechnical site investigations given in Figure 2. At this stage, it was decided to implement additional soil investigations to verify the difference and to obtain new and more reliable geotechnical data which was necessary to be used for the reevaluation of the stability and safety of the retaining structure. For this purpose, five CPT tests are performed from the existing ground surface to a maximum depth of 30.0 m. One of the typical results of CPT is presented in Figure 3.

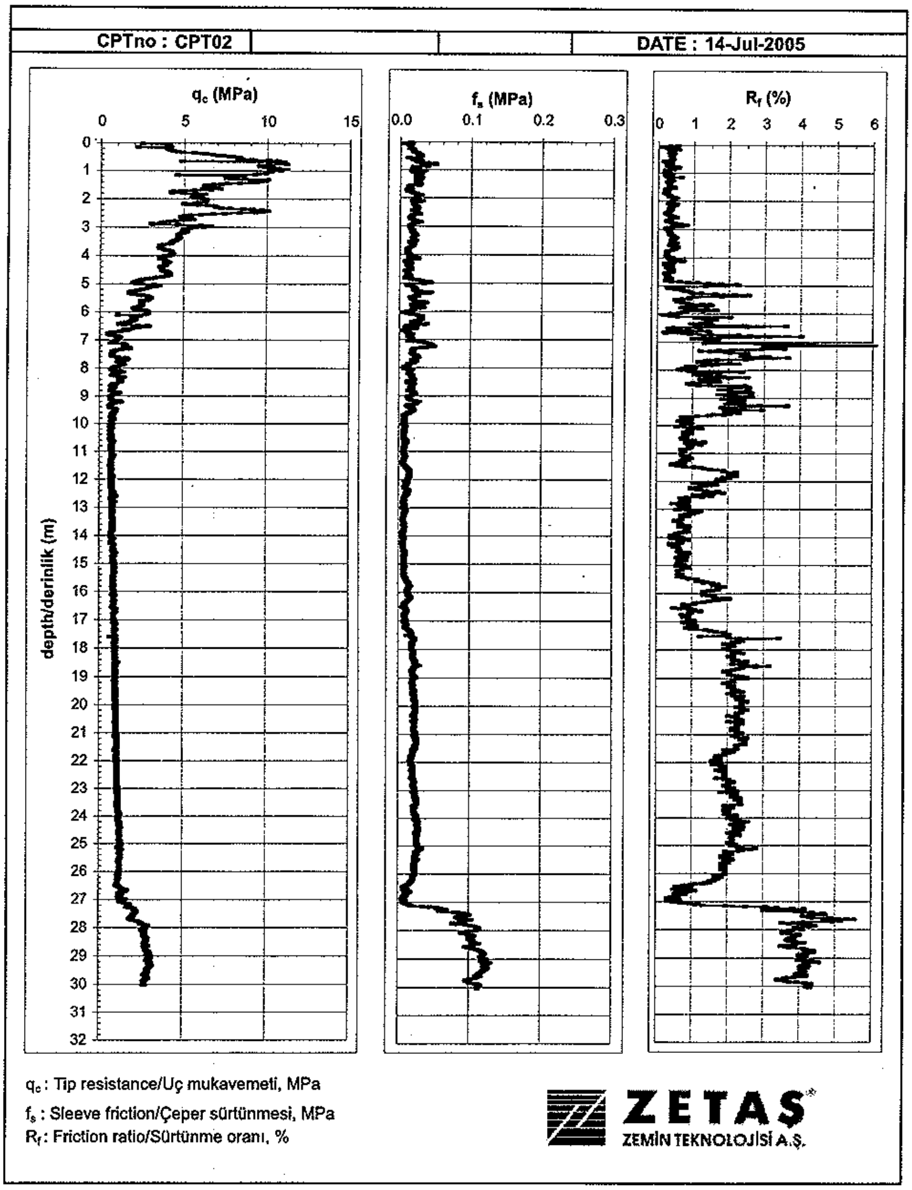

Figure 3. CPT 02 - Change of q_c, f_s and R_f values with respect to depth.

As a result of these additional soil investigations the followings are observed:

- The thickness of the top sand layer is limited to about 5.0 to 6.0 m instead of initially indicated value of 12.0 m.
- The relative density of the sand decreases with depth, and friction angle is estimated to vary between 30° to 40° within the layer.

- Below the sand layer, silty – clayey sand with a very low $q_c \approx 2 - 3$ MPa values is encountered to a depth of 8.0 m to 10.0 m.
- The lower layer is extremely weak normally consolidated silt or clay with q_c value even smaller than 1 MPa. The undrained shear strength of this layer is estimated from the CPT results and found to be $s_u = 20 - 40$ kPa, Figure 4.
- The GWT is located 2.5 m below the ground surface instead of 5.5 m.

Therefore, it became compulsory, to reevaluate the tie back retaining structure design, together with the new design of foundation for the main structure.

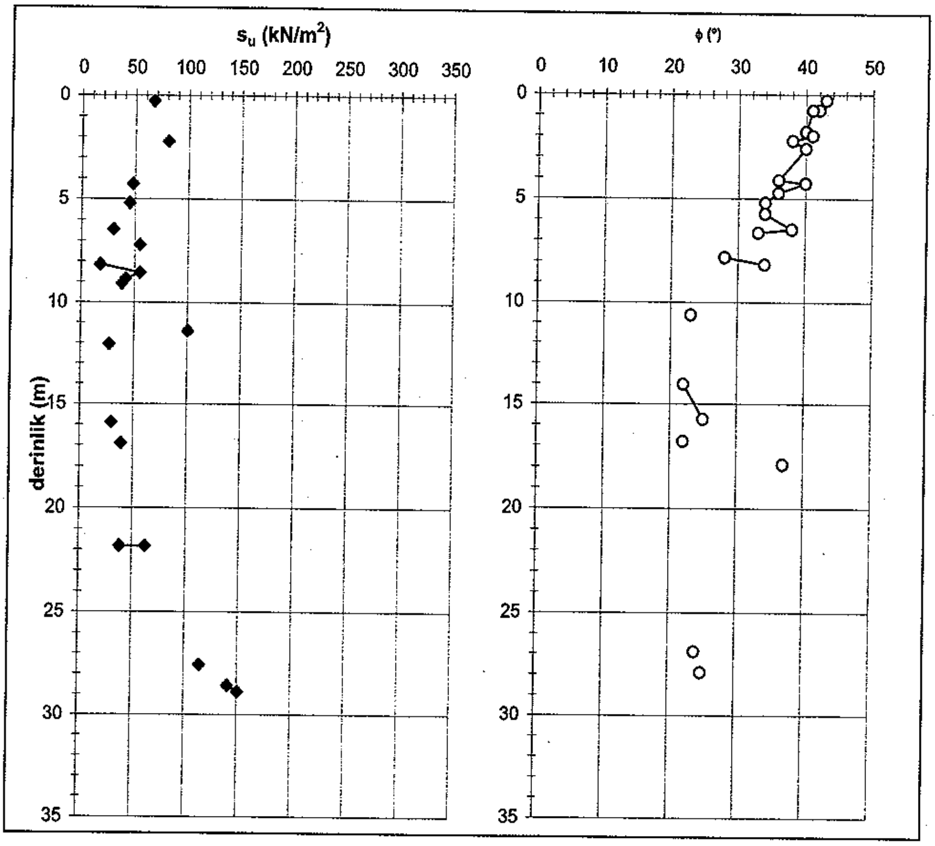

Figure 4. Geotechnical Parameters from CPT.

5 Foundation System

It is stated above that the top of the foundation slab is about -12.5 m. For the subsoil profile the vertical stress relief (unloading) due to excavation is estimated to be about 120 kPa. It is also estimated that the loads imposed by the superstructure will lead to foundation base pressure of 220 kPa. Therefore, the net normal stress on the foundation due to the structure after excavation will be about $\sigma_{bn} \cong 100$ kPa. The foundation will be located on weak soft normally consolidated clays and therefore, large consolidation settlements are foreseen. The ultimate consolidation settlement is estimated using Janbu's Tangent Modulus procedure and found to be $s_{cult} \cong 20 - 45$ cm using CPT data. Based on these values it is concluded that the mat foundation on insitu soils is not permissible. Under such circumstances, the mat either will be constructed on piles or the subsoil will be improved prior to the construction of the structure. The soil improvement option was preferred and implemented, because it has also provided positive additional support against lateral earth pressures imposed on the retaining structure during excavation together with the means of reducing vertical displacements due to softening of the clay subsoil under earthquake loadings.

6 Retaining Structure

6.1 Original Design

For the planned structure with three basements, an excavation to a depth of approx. 12.5 m will be carried out. As discussed above, the system for the retaining structure and the design of the individual members was initially performed based on the results of the initial soil investigations.

For this purpose, considering the presence of high GWT a tie back diaphragm wall with a thickness of 0.60 m is considered. The tip of the wall is located at -18.5 m below the ground the surface to overcome piping and to minimize the seepage quantity towards the excavation. Two rows of pre-stressed anchors with horizontal spacing of s_h = 1.25 m were designed to support both lateral earth pressure and the hydrostatic water pressure behind the retaining wall. The typical section of the retaining structure is given in Figure 5.

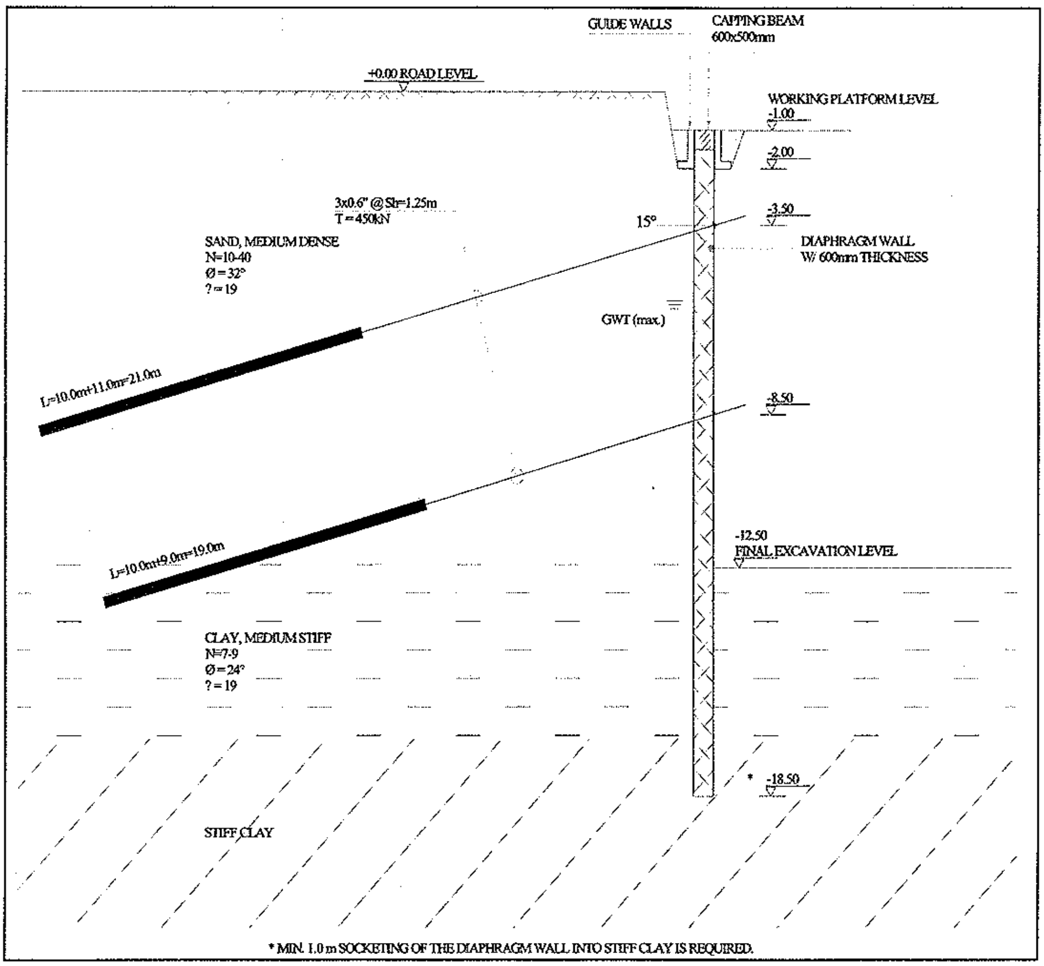

Figure 5. Typical section of the initially planned retaining structure.

The elevation of the first row of anchors was -3.50 m, having inclination of 15° with horizontal. The length of the first row was l = 21.0 m and consequently the grouted length of the anchor were partly in sand and silty clayey sand leading to minor difficulty in construction and obtaining the design loads of T = 450 kN using top hammer and continuous casing during drilling.

However, due to the presence of very weak and soft saturated silts and clays below -8.0 m determined during construction by means of CPT's, ($s_u \cong 20$ kN/m^2), it was impossible to drill for the construction of second row of anchors.

6.2 Alternatives for Change in Design

At this stage it was compulsory to change the design of the tie back retaining wall system. First alternative was, to construct central part of the r.c. structure and brace the retaining wall at an elevation of -9.00 m using horizontal steel tubes. Numerical analysis is performed using Plaxis and estimated that 40 – 60 mm additional horizontal displacements will occur, leading to decrease in first anchor load to value of T = 440 kN, Figure 6.

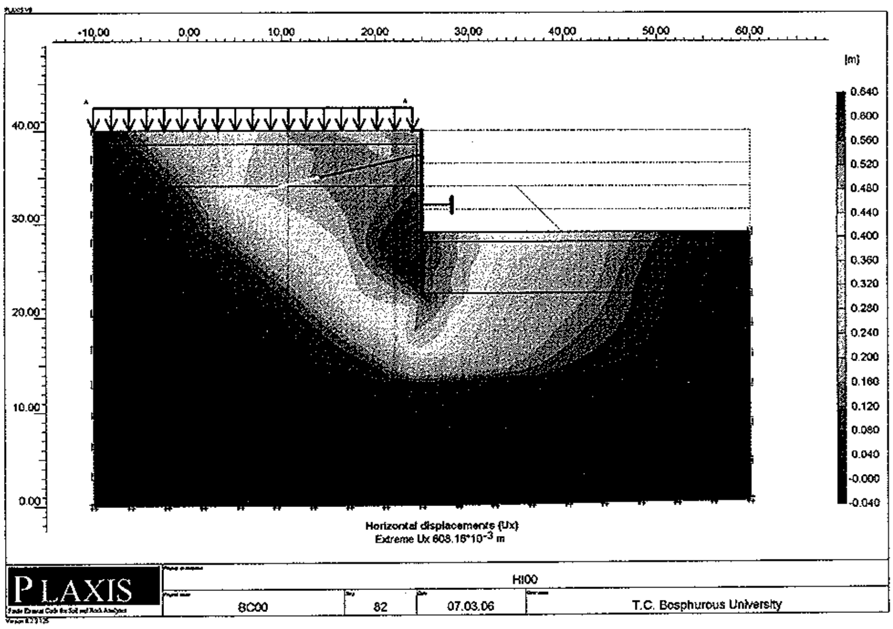

Figure 6. Horizontal displacements in case of rigid bracing in PLAXIS.

The second alternative utilizing the partial top down procedure was favored by the client was, to construct the r.c. floor slab of the hotel structure at an elevation of -9.0m in various segments around the perimeter and adjacent to the retaining wall. For this purpose, it was estimated that safe partial excavation could be performed to a level of -7.50m. Therefore, it was decided to construct temporary supporting cast-in-situ piles from this elevation at various locations. For this purpose, Φ= 80cm in diameter and 22.0m in length reaching to approximate elevation of -29.50m cast-in situ piles are proposed to be constructed. Thirty five number of piles are designed to be constructed at the central portion to support the reinforced floor slab, Figure 7. The maximum horizontal earth and hydrostatic water pressure was estimated to be ω= 230 kN/m leading to a maximum bending moment of M_{max}= 522 kNm/m at the dredge level. The 20Φ20 reinforcement used in the construction of 60cm thick diaphragm wall was sufficient. Due to the safety of the retaining system, the outer parts of r.c. slab were constructed in six segments one after the previous one.

6.3 Jet Grouting – High Modulus Columns

In both alternatives, it was necessary to implement an additional support below the dredge level in order to prevent plastic deformations during excavation below -9.0m.

For this purpose, jet grouting columns are implemented just below the dredge level. In fact, as explained earlier, the soil improvement by means of high modulus columns below dredge level were necessary to overcome three different foundation hazards, namely

1. To provide additional support to carry lateral earth pressure and hydrostatic pressure on the retaining system.
2. To provide vertical stiffness, in order to reduce excessive consolidation settlements under the mat foundation to tolerable values
3. To provide additional safety by means of high modulus columns against modulus degradation and strength loss of weak soft silts and clays under high shear stresses imposed by major expected earthquake loadings. It is well established recently that even fine-grained soils show significant vertical displacement under strong seismic loadings, Soncio and et al. (2003), Kalafat et al. (2003), Bray et al. (2004). Further, the utilization of jet grout columns to mitigate liquefaction induced hazards upon earthquakes have been successfully used in practice, Martin and et al. (2004).

The jet grout columns were designed based on their vertical load capacity under the structure. It is estimated that, Φ=80cm column with a length of l=16.0m will have a safe capacity of Q_s= 75 tons. For total structural load of 53200 tons, the number of columns will be $53{,}200 \div 75 \cong 710$ leading to total length of $710 \times 16 = 11{,}400$m. The total area of the structure is 2450m^2, leading to one Φ80cm column to an effective area of approximately 3.45m^2. This nearly corresponds to replacement area of $a_r = 0.5 \div 3.45 \cong 15\%$. Under this replacement ratio, it could be easily stated that, the additional vertical displacement of the mat foundation under earthquake loading will be low and expected to be below critical values due to the modulus degradation and strength loss of soft clays and silts under earthquake loadings (Durgunoglu, 2006).

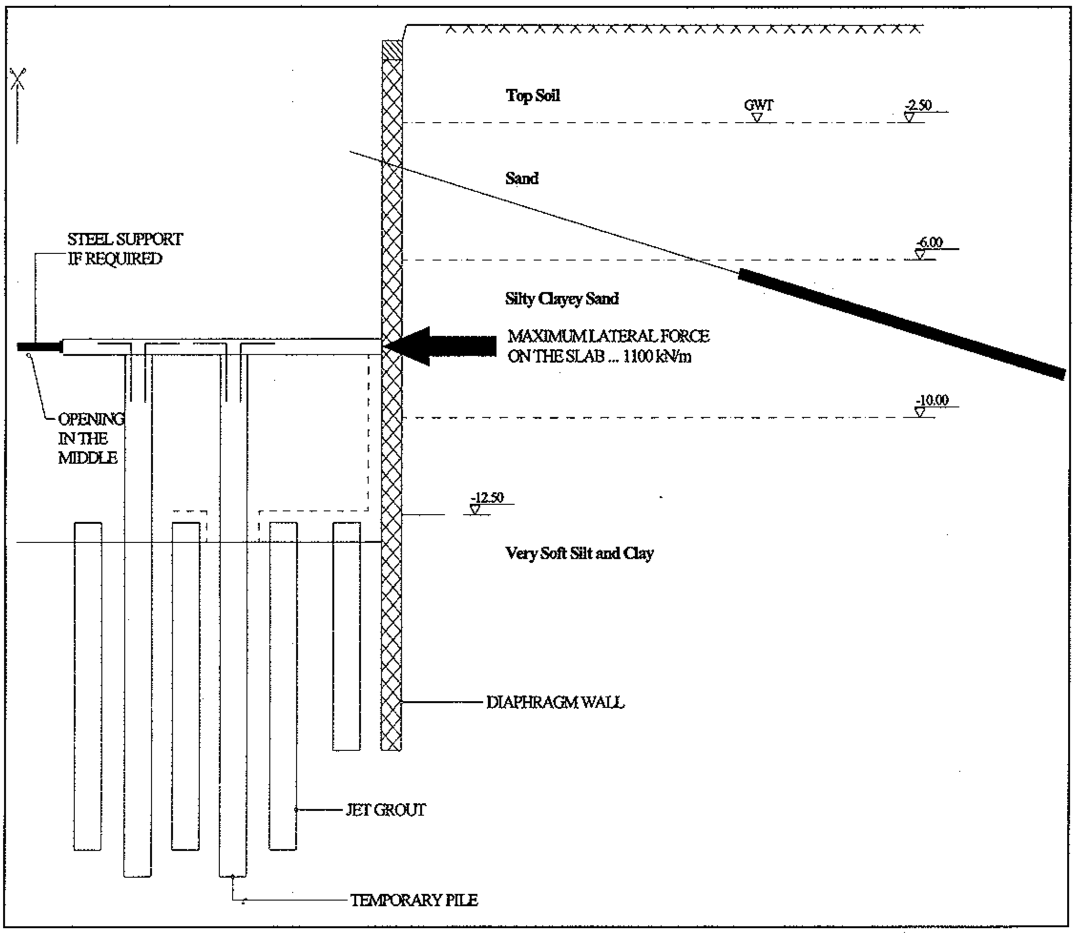

Figure 7. Section showing the retaining wall with r.c. floor slab and temporary bored piles.

6.4 Top Down Construction

The jet grout column were constructed from a working platform of -4.50m, i.e. working level of 1st row of anchors in order to reduce the lateral displacements and increase the safety of the retaining structure during partial excavation to a depth of -9.0m for the construction of reinforced concrete floor slab. The grouting is done only between the elevations of -12.50m to -28.50m, for a length of 16.0m.

Upon concreting the r.c. floor slab at -9.0m in various segments, it was possible to excavate the soil below the floor slab of 4.0m in width and evacuate the mucking from the space provided at the central part of the floor at elevation of -9.0m. After partial excavation below segments of the floor slab at -9.0m, it was possible to construct the mat foundation and exterior basement wall between the elevations -9.0m to -12.5m in segments as well. During the construction of mat foundation, it was also possible to construct the r.c. columns and shear walls between the elevations of -9.0m to -12.50m. After the completion of the superstructure between -9.00m to -12.50m, the temporary cast–in–situ piles are demolished. As a result, third basement parking between elevation of -9.00m to -12.50m having one floor slab, mat foundation, the r.c. columns and shear walls between are constructed from top to down procedure, due to initially unforeseen very unfavorable conditions. This procedure was adopted because of misleading results of initial soil investigations together with the existence of very high ground water table, 3.0 m above than what was foreseen, together with much weaker subsoil conditions especially below the level of -6.0 m. Photographs showing cast-in-situ pile, r.c. floor slab at -9.0m elevation and general view of the top down construction and the retaining system are presented in Figure 8a and 8b.

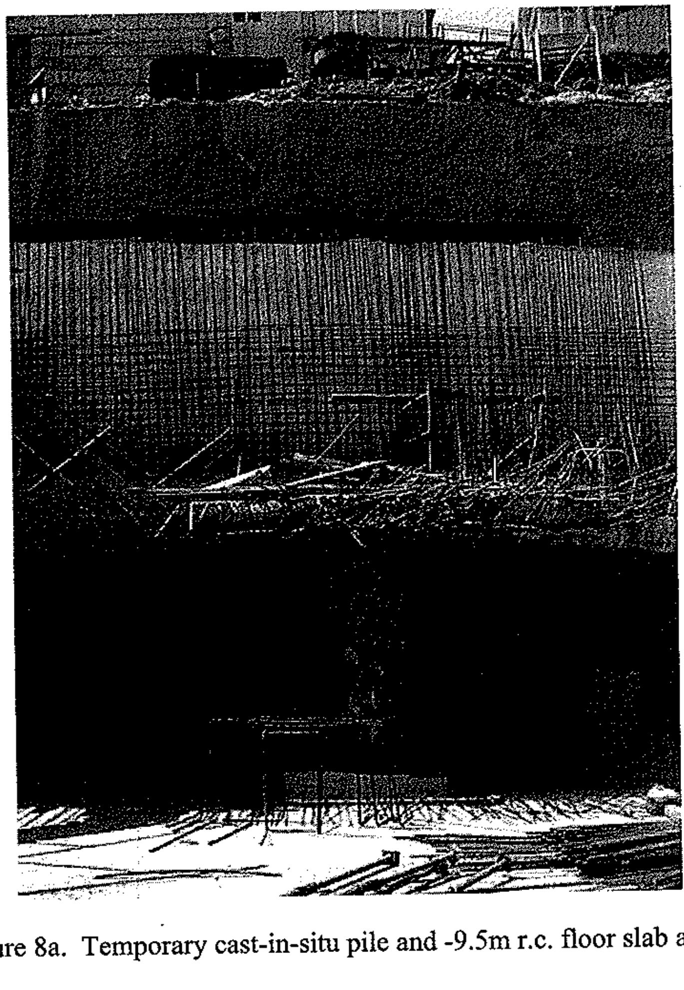

Figure 8a. Temporary cast-in-situ pile and -9.5m r.c. floor slab above.

Figure 8b. General view of the excavation and support system.

6.5 Instrumentation and Monitoring

Three inclinometers were placed at the beginning of the construction, to locations shown in Figure 1, for monitoring lateral displacements at different stages of excavation, construction of the retaining structure and during implementation of subsoil improvement by means of jet grouting. As seen in inclinometer no:2 in Figure 9, displacements showed a normal trend until the end of construction of the 1st level of prestressed anchorages at elevation -3.5m, having a displacement pattern towards excavation side. After that, during the works of soil improvement by jet grouting the lower part of the diaphragm wall displaced to the outer side with a significant amount. Following completion of the jet grouting and during partial excavation for slab construction at elevation -9.0m, it was observed that the upper part of the diaphragm wall continued to lean as expectedly towards excavation side, whereas the lower part displaced to opposite side due to resistance of the jet grout improved soil against very soft outer soil. This trend continued after construction of the slab at -9.0m, during additional excavation to reach -12.5m level for construction of the mat foundation with no significant change at the top of the diaphragm wall. It eventually stopped after construction of the mat foundation and establishment of final stabilization of the retaining system. Therefore, monitoring played a very important role for visual awareness of the phenomenon and taking step-by-step decisions.

7 Conclusions

A case study for a construction of a retaining structure in a difficult subsoil conditions under high seismicity is presented. From the paper, it is evident that:

1. The quality and especially reliability of the subsoil modeling towards the design has the prime importance.
2. It is always advisable to verify the subsoil conditions prior to construction if possible, especially if they are obtained by a different party.
3. High modulus columns using jet grouting technique implemented was very cost effective, since they were utilized for various purposes to overcome the geotechnical and seismic hazards:

- to increase bearing capacity,
- to reduce settlement,
- to minimize vertical displacements during earthquake loadings due to modulus degradation of the clay subsoils,
- to provide support under dredge level to prevent plastic lateral deformations upon excavation and
- to reduce the predominant period of the subsoil.

4. Application of partial top-down construction technique for the utilization of permanent part of the r.c. floor slab to provide the necessary lateral support below the first row of anchors was very effective.
5. Monitoring lateral displacements by means of inclinometers have provided excellent data for the engineering decisions to be implemented at various stages of the construction. Consequently, it was possible to overcome the possible hazards and failures that may occur due to the misleading geotechnical model obtained during the initial phase of the soil investigations and complicated subsoil conditions.

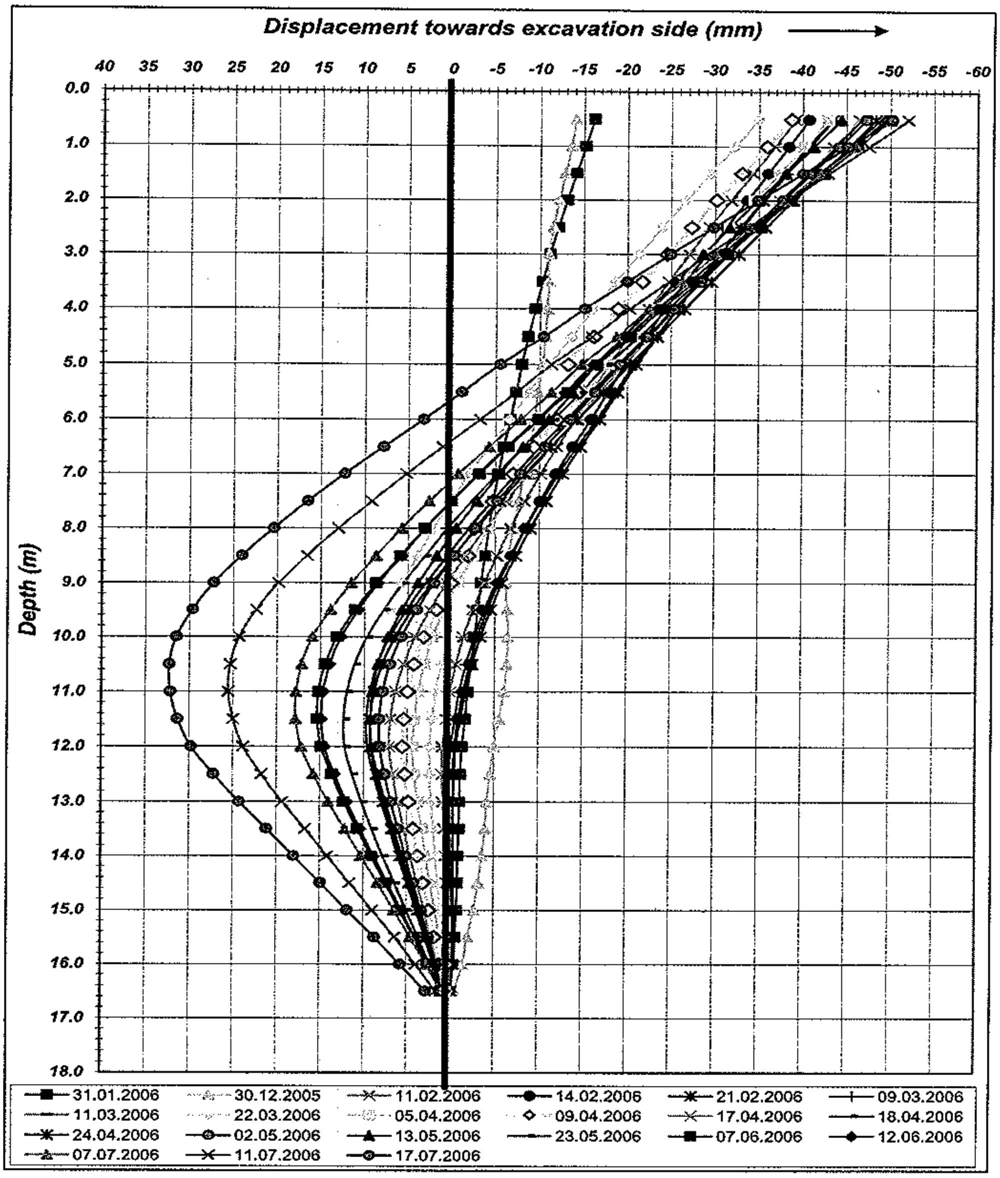

Figure 9. Progress of horizontal displacements at Inclinometer 2.

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