Harmony of Retaining Systems to Various Local Subsoil Conditions – A Case Study

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ABSTRACT

A case study for the utilization of various retaining systems for different subsoil and groundwater conditions encountered within a given site is presented in this paper. The project is known as "BJK Fulya Complex", covering approximately 160,000 m^2 floor area. It is located at a very prestigious district of the city, therefore maximum underground space gain were desired. As a result nearly 20 m of excavation is planned to be performed partly under groundwater. Due to unique topography and geology of the site, subsoil and groundwater conditions at various faces of the excavation differ considerably. Because of the complicated geology, budget constraints of the project and the high seismicity, it was compulsory to employ various retaining structures such as flexible and rigid retaining systems at various locations within the site including permanent and temporary soil nailing, permanent tie-back cast in-situ reinforced concrete wall and temporary tied-back diaphragm wall consist of soldier cast in-situ piles with jet grout columns in between. Performances of various systems are monitored closely by means of inclinometers. Displacement data and experience obtained from this case study serves an excellent source of data and example for future applications in similar conditions within the city.

INTRODUCTION

During the last decade, the city of Istanbul has performed significant growth in economy. Becoming the biggest metropolitan city of the region, the need for high-rise residential and office buildings and shopping malls with multiple basement levels increased noticeably considering the raised value of the land which became a major part of the cost in construction of buildings. In order to build great number of basement levels, especially to obtain parking space and room for shopping and entertainment facilities, deep excavations and construction of retaining structures became compulsory.

An interesting case study is presented in this paper for the utilization of various retaining systems for different subsoil and groundwater conditions encountered within a given site, considering the output of optimization of the cost as well. The project is known as "BJK Fulya Complex" consisting of high-rise residential twin towers, hospital and hotel covering approximately 160,000 m^2 floor area including hypermarket, technomarket, cultural center, entertainment facilities and underground parking area. The project is located at a very prestigious district of the city, therefore maximum underground space gain were desired. As a result nearly 20 m of excavation was performed partly under groundwater. Figure 1 shows the layout plan, where the twin towers are on the right, the hospital is on the left and the hotel block is in between.

The project site has a very rugged topography having about 25 m difference in elevation in perpendicular direction to the covered old creek located at the bottom of the valley along the main street. Due to unique topography and geology, subsoil and groundwater conditions at various faces of the excavation differ considerably. Furthermore, again due to unique topography, at the hill side in addition to 18.5 m of temporary retaining structure, permanent retaining structure of about 15-20 m high had to be constructed over the temporary wall leading to a retaining structure as high as 36 meters.

Fig.1. BJK Fulya Complex

The city of Istanbul is in very seismically active region. Therefore, permanent part of the structures are of prime importance. Marmara Fault System is very close to the city, which is the western end of the North Anatolian Fault, NAF. Figure 2 demonstrates Marmara Fault System, located at the south of Istanbul. Recently August 17, 1999 Kocaeli (M_w=7.4) and November 11, 1999 Düzce (M_w=7.2) earthquakes occurred on NAF within the Marmara Region in approximately 100-150 kilometers east of the city of Istanbul. After these two catastrophic earthquakes further worldwide scientific interest has been given to the structure of North Anatolian Fault System, especially under the Marmara Sea. According to the studies carried out after 1999 Kocaeli and Düzce earthquakes, the probability for the occurrence of a M_w>7.0 earthquake effecting Istanbul within the next 30 years due to the existence of potential seismic gaps is about 65% (Parsons *et al*, 2000).

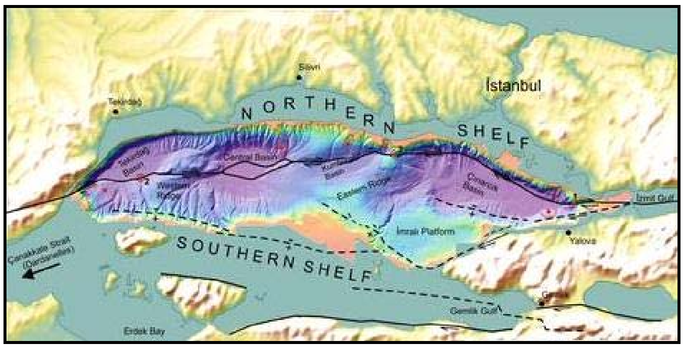

Fig.2. Marmara Fault System

Main lithological unit of the site is soft rock greywacke locally known as Trace Formation, which is lithologically alternating sandstone, siltstone and claystones with various degrees of weathering and fracturing. The extend of weathering and fracturing controls the mechanical properties and in fact geological observations do well agree with the results of measurements reflecting mechanical properties of the formation. The geotechnical modeling of formation, weathered zones, extend of fracturing and compressibility modulus of formation are usually obtained by means of integrated seismic survey and Menard pressuremeter testings performed within the boreholes at various locations and depths. (Durgunoglu and Yilmaz, 2007; Yilmaz and Durgunoglu, 2008)

Due to complicated geology and the high seismicity of this site, it was necessary to employ extensive soil investigations to identify the limits of various lithological units and the ground water conditions. As a result, various types of retaining structures were employed having both flexible and rigid retaining systems at various locations within the site of "BJK Fulya Complex". Various forms of retaining structures that have been utilized at the site include temporary soil nailing, permanent soil nailing, temporary and permanent soil nailing along with the permanent tied-back cast in-situ reinforced concrete caisson wall and temporary tied-back diaphragm wall consist of soldier cast in-situ piles with jet grout columns in between.

The performances of various systems are closely monitored by means of inclinometer recordings taken at certain time intervals in parallel to the staged excavation. Readings from sixteen inclinometers at different locations were recorded throughout the construction.

Displacement data and experience obtained from this case study together with previous experience (Durgunoglu *et al*, 2007) serves an excellent source of data and example for future applications in similar conditions within the city.

PROJECT DESCRIPTION

Besiktas JK, founded in 1903, is one of the famous football clubs in Europe and often participates for the UEFA European competitions and also professionally contributes in many sports branches including basketball, volleyball and handball. BJK is the legal owner of the real property, BJK Fulya Complex, which is located at a very prestigious district of Besiktas, Istanbul. Total area of the land is more than 43,000 m^2, consisting of 29,000 m^2 construction area and the rest is spared for sporting facilities. Total construction area is 160,000 m^2 and more than half of it is constructed underground. The construction cost is more than 100 million US dollars excluding the land purchase cost which is about twice of the construction cost. Figure 3 shows the construction area of "BJK Fulya Complex" just before the beginning of the excavation stage, on September, 2005.

Fig.3. BJK Fulya Complex - before the excavation

"BJK Fulya Complex" is a high standard, modern architecture, multi-functional complex that contains two residential towers, a five-star hotel building, a fully equipped hospital building, a cultural center, a hypermarket, a technomarket, dining and entertainment facilities, and parking lot. There are 240 high-tech residential units in twin towers which are more than 150 m in height. 12,000 m^2 of hospital building is on the north side of the project, having a height of approximately 100 m.

Between the hotel and twin towers there is a hotel building which is 125 m in height and has 15,000 m^2 floor area.

On the ground level dining and entertainment facilities, luxury restaurants and fancy boutiques are located, besides there is a cultural complex that has ateliers, exhibition halls and an amphitheatre. There are also swimming pools, tennis courts and sports areas alongside of the underground facilities.

As in the other densely populated metropolitan cities, the value of the land in high-status areas is a major cost issue in construction. Therefore shopping areas as well as parking lots and the other technical spaces are positioned in underground levels. "BJK Fulya Complex" has also more floor area on underground than upper ground levels. There are 4 to 5 underground levels covering almost 90,000 m^2 floor area. There are 16,000 m^2 of large sized hypermarket, 8,000 m^2 of very large sized technomarket, 3-level parking lot which has the capacity of 2,000 car parking and right under the hospital building almost 8,000 m^2 of side facilities of hospital consisting of various surgery rooms, intensive care units, etc.

The completing date of the construction of "BJK Fulya Complex" is planned to be September, 2008. Figure 4 shows the current construction stage, September 2007, where shell construction of the twin towers, at south, nearly completed and the sixth floor of hospital building, above the ground, at north, is under construction. In between, hotel building has a little of shell construction works left.

Fig.4. BJK Fulya Complex - during the construction, September 2007

SUBSOIL MODELING

The subject site has a very rugged topography having about 25 m difference in elevation in perpendicular direction to the covered old creek located at the bottom of the valley where the main street is positioned. The topographic elevations at site vary from +28.0 m to + 52.0 m LD, elevation above local Istanbul datum. The basement of the complex has the formation level of +9.5 m. Since the main construction axis is alongside the valley, the existing ground should be excavated starting from +28.0 m to the bottom level of +9.5 m. Therefore the average height of the excavation would be nearly 20.0 m.

The scope of the site investigations at the initial stage originally consisted of six boreholes, covering total length of 94.0 m. However considering the planned structures and encountered subsoil conditions, additional second stage ten boreholes with total length of 235.0 m were realized within the scope of soil investigation programme. The locations of the investigation points are shown on the general lay-out plan of the site as given in Fig. 6.

Standard Penetration Testing SPT with regular intervals and representative sampling were performed at the alluvial sand and gravel subsoils located above the lithological bedrock unit according to ASTM D-1586. Energy corrected SPT/N_{60} blow counts are determined. The drilling method was rotary drilling and bentonite slurry was used in all boreholes for circulation in order to minimize the bottom heave of alluvium during drilling and to get higher total core recovery from the main lithological unit of closely fractured greywackes. TCR, SCR and RQD values of rock formations are also determined and their variations with elevation are presented on borehole log charts. Additionally, after the drilling of the boreholes groundwater levels in each borehole are recorded and monitored by means of piezometers.

Since the subject site is in perpendicular to the old creek, the upper levels of the subsoil is formed of alluvium and fill. The depth of the alluvium starts from zero ground level, at the east side, and reaches to eleven meters, at the west side of the subject site. Alluvium and fills lie on the bedrock greywacke, which is classified as Thrace Formation. The fill is uncontrolled manmade to correct the topography. Thrace formation is composed of alternating claystones, shales siltstones and sandstones with various degrees of weathering and fracturing. Figure 5 demonstrates the boundary between greywacke and alluvium zones where greywacke formation of the site has gray in color, while alluvium and the upper fill is brownish.

Fig.5. A picture from early stages of excavation, September 2005

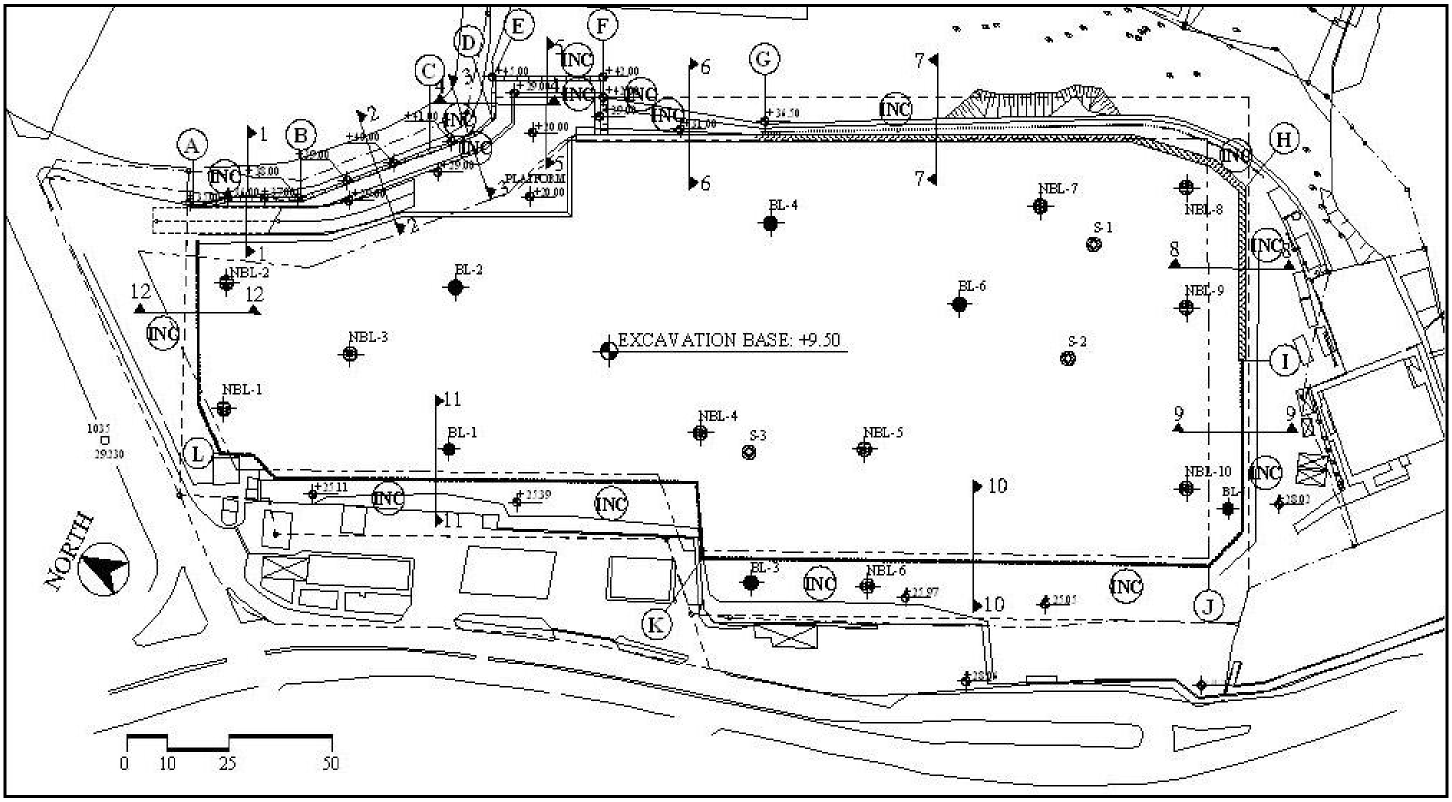

Fig.6. General layout plan of BJK Fulya Complex

Laboratory testing were performed on soil, rock and groundwater samples obtained from boreholes in order to determine geotechnical parameters and aggressiveness of the water for foundation engineering evaluations. The range of index properties obtained from alluvium formations shown below.

Natural Water Content, w_n (%) = 7-24,
Liquid Limit, LL (%) = 22-36,
Plastic Limit, PL (%) = 13-21,
Plasticity Index, PI (%) = 7-18,
Unified Soil Classification, CL, SC, SM, GC, GM.

Over the site of investigation three shallow seismic survey was conducted to derive a 'geodynamical–seismic' model below the ground surface. The dynamic parameters such as the P-wave and S-wave velocities (v_p and v_s), shear modulus, and compressibility modulus of the subsoil were evaluated. Table 1 represents geodynamical properties of the subject site. There were three different seismic zones representing various geological units present. The first zone is composed of loose/soft soils, i.e. alluvium, the second is clay/sand i.e. fill, and the third is fractured greywacke.

In Table 1, "d_i" is thickness of the subsurface layer in meters, "v_s" is the shear wave velocity in m/s, "v_p" is the pressure wave velocity in m/s, "γ" is total unit weight of the subsoil unit in kN/m^3, "G_0" is the dynamic shear modulus in MPa and "E_0" is the dynamic elasticity modulus in MPa. Note that, E_0 and G_0 are modulus values corresponding to very low strain level employed in seismic surveys. The representative values of shear wave velocities for each unit could be taken as 50 to100 m/s for alluvium, 200 to 300 m/s for the fill and 600 to 1100 m/s for the greywacke depending on the extend of weathering and structural discontinuities and depth. The measured v_s values for greywacke are in good agreement with the results of previous seismic surveys (Durgunoglu and Yilmaz, 2007).

Table 1. Geodynamical Parameters of BJK Fulya Complex

Survey Layer		h_i (m)	v_s (m/s)	v_p (m/s)	γ (kN/m^3)	μ	G_0 (MPa)	E_0 (MPa)
S 1	1	0.5-2	56	211	16.4	0.46	5	15
	2	3-3.5	288	737	17.4	0.41	147	415
	3	2.5-5	667	1500	20.0	0.38	907	2500
S 2	1	4.5-5	225	499	16.9	0.37	87	239
	2	3-3.5	600	1500	20.0	0.40	734	2062
	3	3-4.5	1091	2824	22.6	0.41	2742	7745
S 3	1	5-7.5	300	639	17.2	0.36	158	429
	2	3.5-4	667	1625	20.2	0.40	916	2563

Based on the in-situ and laboratory tests carried out for the subject site, together with the previous performance experiences of such retaining structures in similar subsoil conditions following drained condition geotechnical parameters were used as stress based design geotechnical parameters for retaining systems of “BJK Fulya Complex” as given below in Table 2. Although in the short term, temporary conditions, partial drainage conditions were governed, due to complicated geology and the drainage conditions, it was safer to utilize fully drained condition for the design.

Table 2. Geotechnical Parameters Used for Design

Subsoil Layers	Parameters	Symbol	Value
Alluvium and Fill	Internal friction angle	ϕ'	27.0°
	Cohesion	c'	0 kPa
	Total Unit weight	γ	18 kN/m^3
	Active earth pressure	K_a	0.376
Sandstone, Siltstone, Claystone (Greywacke)	Internal friction angle	ϕ'	33.0°
	Cohesion	c'	0 kPa
	Total Unit weight	γ	20 kN/m^3
	Active earth pressure	K_a	0.295
	Earth pressure at rest	K_0	0.500

VARIOUS TYPES OF RETANING STRUCTURES

Various locations along the perimeter basically five different types of retaining systems are utilized and shown in Fig. 6. Type 1 is permanent pre-stressed anchored reinforced concrete caisson utilized on temporary soil nailing, from point A to B in total length of 27 m refer to Fig. 7a and Fig. 7b. Type 2 is permanent pre-stressed anchored reinforced concrete caisson utilized on soil nailing that upper part is permanent and the lower part is temporary, from point B to F in total length of 87 m refer to Fig. 8. In Fig. 9, back side faces demonstrate the view of the both systems, Type 1 and Type 2 together. Type 3 is permanent soil nailing utilized on temporary soil nailing, from point F to G in total length of 52 m refer to Fig. 10a, Fig. 10b and 10c. Type 4 is temporary soil nailing, from point G to I in total length of 161 m refer to Fig. 11a and Fig. 11b. Type 5 is temporary pre-stressed anchored diaphragm wall, from point I to A in total length of 361 m refer to Fig. 13a, Fig. 13b and Fig. 13c.

<u>Retaining Structure – Type 1</u>

From A to B, permanent pre-stressed anchored reinforced concrete caisson walls are utilized on the upper side of the ramp, sloping down from elevation +28.0m to +20.0m and 7m in width. In lower elevations of the ramp beneath the caisson walls, temporary soil nailed walls are constructed. The height of the soil nailed walls is between 10.5 m–18.5 m. Horizontal and vertical spacings of nails are S_h=S_v=1.5 m and length of the nails are ranging from 4 m to 12 m. Cast in-situ reinforced concrete caisson walls are 70cm in thickness and 10.3 m to 20.5 m in height. The thickness of the footing of the caisson wall is 1.5 m. Caisson walls are manually constructed due to very limited space available and anchored by permanent pre-stressed anchors, with bond length of 8 m and total length ranging from 19 m to 22 m. Horizontal and vertical spacings of anchors are S_h=1.5 m and S_v=2.5 m–3.0 m respectively. The lock-off load of tieback anchors is 350 kN while the test load is 450 kN. Typical detailed cross-section of Type 1 is presented in Fig 7a and a picture from caisson wall is given in Fig. 7b.

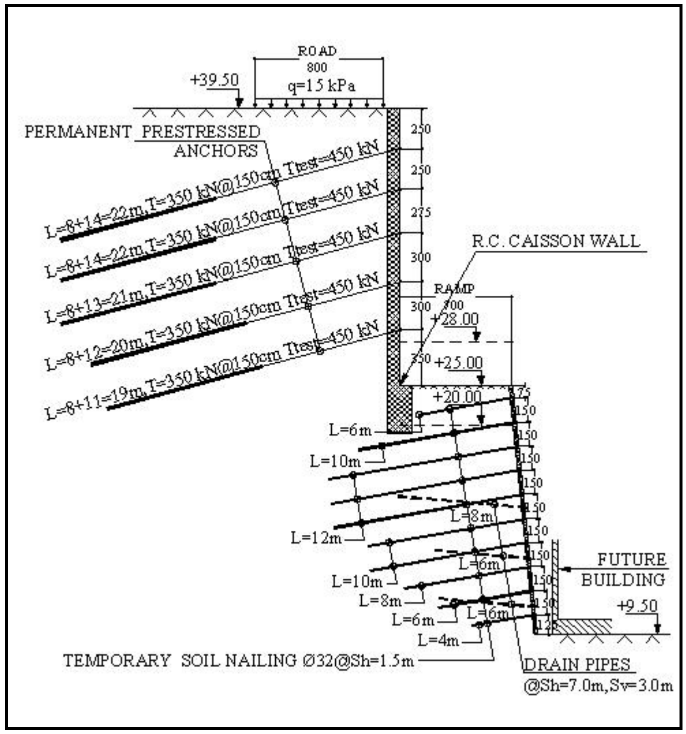

Fig. 7a. Detailed cross-section of Type 1

Fig. 7b. A picture from caisson wall of Type 1

Retaining Structure – Type 2

After point B along the perimeter due to increase in elevation of existing topography, permanent soil nailing is utilized vertically between permanent pre-stressed anchored caisson wall and temporary soil nailing reaching to point F. The maximum height of the caisson wall is 12.5 m. Caisson walls are tied back permanently with anchors as in Type 1, 21 to 22 m in length. Permanent soil nailed walls are 15 m in height. One row of pre-stressed anchors are utilized on the soil nailed wall above the permanent nails to prevent excessive lateral displacement of the nailed walls, as shown in Fig. 8a and Fig. 8b. At this section, the height of the permanent retaining system reaches to 25 m and the subsoil gets stronger 10 m below the road. Therefore, permanent soil nailing system between caisson wall and temporary soil nailing is preferred.

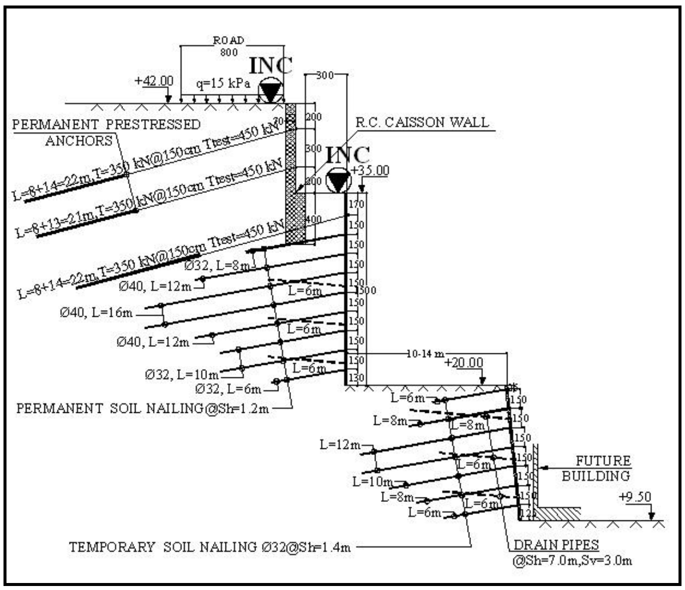

Fig. 8a. Detailed cross-section of Type 2

Fig. 8b. Pre-stressed anchors on permanent soil nailing to prevent excessive lateral displacement

Horizontal and vertical spacings of the permanent soil nails are S_h=1.2 m and S_v=1.5 m respectively, while those of temporary soil nailed walls are S_h=1.4 m and S_v=1.5 m. An overview from retaining structures Type 1 and Type 2 is given in Fig 9.

Fig. 9. Overview from Type 1 and 2

Retaining Structure – Type 3

Between points F and G on the perimeter, permanent soil nailing is constructed for the permanent part of the retaining system. Below the permanent part, temporary soil nailing is again placed. The height of the permanent soil nailing is in the range of 7.5 m to 14 m. The slope of the permanent soil nailing wall is 1H/3V. The maximum nail length is 16 m. The height of the temporary soil nailed wall is constant and 18.5 m. Horizontal and vertical spacings of the permanent soil nails are S_h=1.4 m and S_v=1.5 m, and S_h=1.5 m and S_v=1.5 m for the temporary soil nailing, respectively. A picture from construction of temporary soil nailing is given in Fig. 10a.

Fig. 10a. A picture from Type 3

Figure 10b presents the typical detailed cross-section of retaining Type 3.

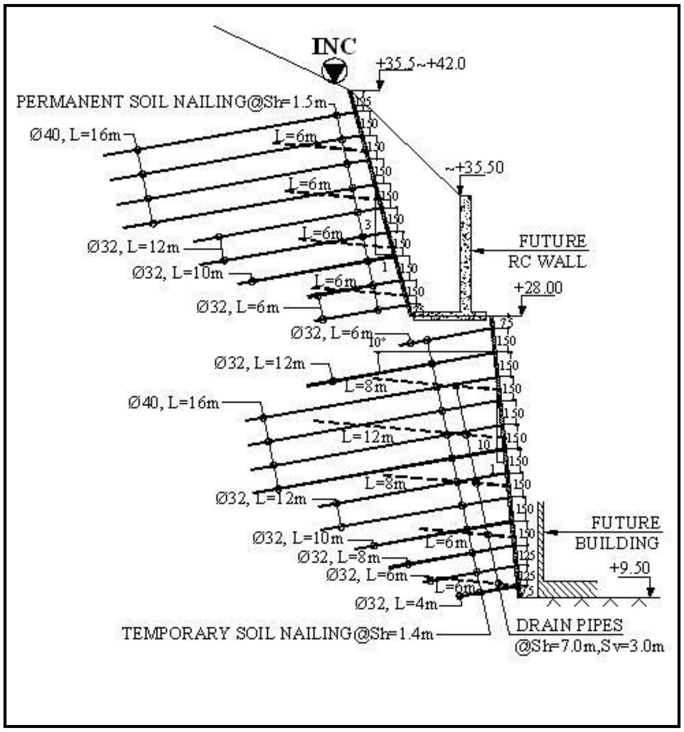

Fig. 10b. Detailed cross-section of Type 3

Excessive lateral displacement is monitored by means of one inclinometer within this part of the retaining structure because of unforeseen potential slip plane due to adverse bedding of the greywacke formation during excavation. Additional long pre-stressed anchors are constructed at these location reaching behind the instable wedge to provide further stability of excavated slope. Figure 10c shows two rows of additional pre-stressed anchors in temporary soil nailed part of the retaining system.

Fig. 10c. Additional rows of pre-stressed anchors

<u>Retaining Structure – Type 4</u>

From point G to I, only temporary soil nailing retaining system is constructed. The typical nail diameter of D=105mm, nail orientation of ω=10˚ with the horizontal and slope angle of β=85˚ (1H/10V) are utilized for all temporary soil nailed walls constructed within the site. Two different nail bars with Ø32mm and Ø40mm in diameter are used. The length of the nails is ranging from 4m to 16m horizontal, spacings of the nails are S_h=1.4 m–1.8 m while vertical spacings are S_v=1.5 m. Typical cross-section and a photograph from temporary soil nailing are given in Fig. 11a and Fig 11b, respectively.

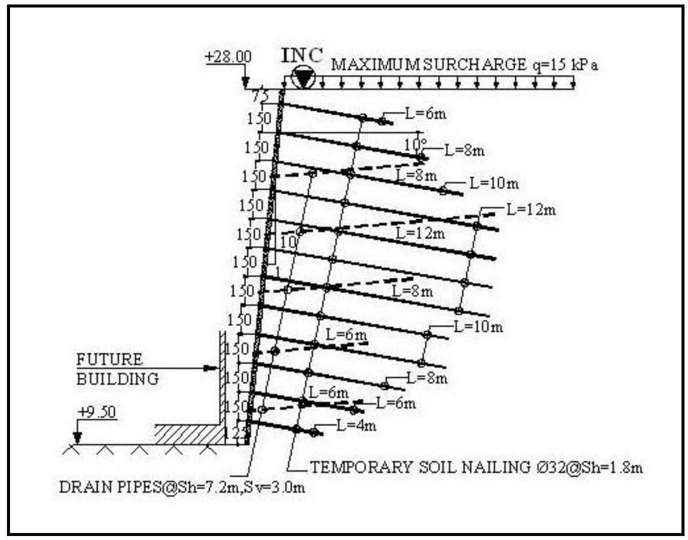

Fig. 11a. Detailed cross-section of Type 4

Fig. 11b. A picture from Type 4

A minor variation in the temporary soil nailing section is described below. There exists an old 7 m high retaining wall between point G and H. The retaining wall is to be kept in place by improving its stability with mini piles of diameter Ø225mm in front of the toe. After construction of mini piles, temporary soil nailed wall is utilized below the existing retaining wall. Detailed cross-section of this section is presented in Fig. 12a. Figure 12b, shows a view of the initial stage of the excavation where mini piles had been constructed in front of the toe of the old retaining wall and the first row of soil nailing has just been completed.

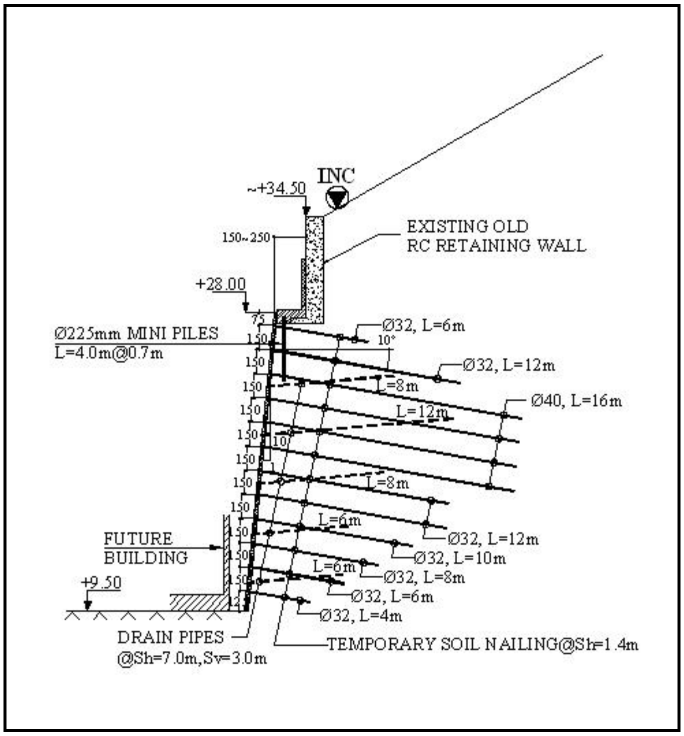

Fig. 12a. Detailed cross-section from temporary soil nailing under the existing old retaining wall

Fig. 12b. Completion of the first row of soil nailing under the existing old retaining wall

Retaining Structures – Type 5

Temporary pre-stressed anchored diaphragm wall from point I to A is constructed. The wall consists of bored piles of diameter Ø65cm, spaced at 90 cm intervals from center to center. To prevent ground water intrusion, jet grout columns in 60 cm diameter are constructed between piles. The height of the diaphragm wall is 15.5 m–18.0 m. Five rows of pre-stressed anchors, 18 to 20 m in length, are constructed to overcome both earth and hydrostatic water pressures on the diaphragm wall. The horizontal spacing of the anchors are 0.9m for upper rows and 1.25 to 2.70 m for bottom rows. The lock-off load for the anchors located in alluvium and fill is 300kN and for the anchors drilled in greywacke is 350-450kN. The concrete pile cap is 60x70cm in section and 100x35cm reinforced concrete beams are placed continuously for each row of anchors. Figure 12a presents the typical detailed cross-section of Type 5. Two photographs taken after the completion of the work are presented in Fig. 12b and Fig. 12c.

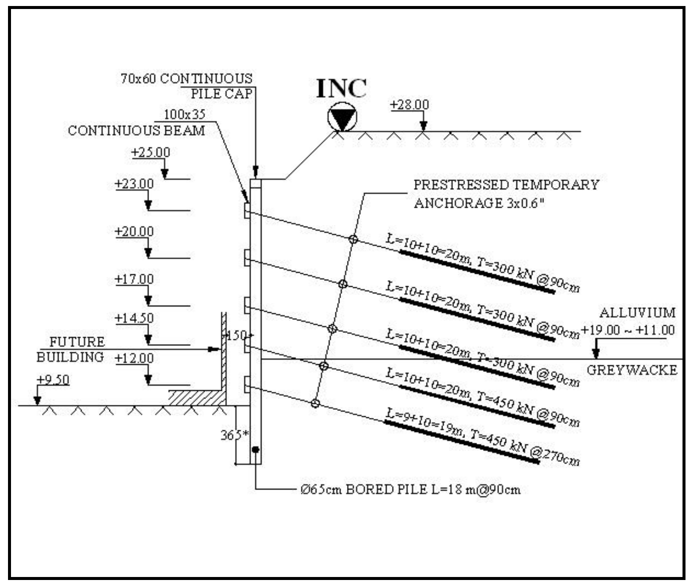

Fig. 12a. Detailed cross-section of Type 5

Fig. 12b. A picture from Type 5

Fig. 12c. Another picture from Type 5

A minor variation of compulsory at certain length along the subject periphery. At some sections between point L and A, bored piles could not be drilled for the last 3 to 6m due to very strong silicified sandstone formation. This formation is a different variation of main lithological unit of greywacke, having much greater strength, hardness and compressibility modulus. At these sections, cast in-situ reinforced concrete walls in segments are constructed below the piles after partial excavation as shown in Fig. 12d. Lateral displacements were monitored carefully during partial excavations and it was seen that there were no appreciable displacement increase as a result of segmental construction procedure followed.

Fig. 12d. Reinforced concrete walls under the bored piles

Two photographs showing completed retaining walls facing both north and south side of the subject site are presented in Fig 13a and Fig. 13b.

Fig.13a. Retaining system facing north side

Fig.13b. Retaining system facing south side

INCLINOMETER READINGS AND LATERAL DISPLACEMENTS

The performances of deep excavations and retaining structures are monitored by means of observed lateral displacements. It is well known that, there are complicated numerical models and software programmes are available to predict the lateral displacements during design stage prior to construction. However, where complicated subsoil geology is prevailing, such as in this case study, the strict performance evaluations should be made based on the measured displacement data rather than the predicted values, especially where the design is based on both stability (i.e. certain factor of safety for each retaining member) as well as occurred displacements of the retaining structures at different phases of the excavations. Although, the procedure followed in design guarantees the safety against lateral earth pressures and the hydrostatic water pressures, design requirements further implement that developed lateral displacements at various stages and various retaining systems should be kept below the acceptable limiting values. Further, even in simpler geological conditions, i.e. only in presence of greywacke formation, results of previous case studies in the city have shown that, the prediction of displacement even employing sophisticated software programmes such as PLAXIS and/or FLAC is mainly governed by the deformation modulus formulation of the subsoil unit. It is also known that, the modulus of soils such as encountered in this case study alluvium, manmade uncontrolled fills and even greywacke are dependent on many factors, including the inhomogeneity of the unit and even more important to excavation induced displacement, strain. Previous experience, Durgunoglu et al (2007) have demonstrated that, the correct predictions of displacements in such conditions as in this case study is almost impossible, therefore strict displacement monitoring during various stages of the construction is compulsory.

Total of sixteen inclinometers were installed prior to any earthworks at various locations along the periphery covering considering the presence of various types of retaining structures that are planned to be constructed, as shown in Figure 6. The inclinometer boreholes are located just outside the retaining wall, in order to guarantee that the measured displacements are not influenced by the relative rigidity of the various retaining wall systems that are constructed. Inclinometers are recorded daily throughout the construction, covering all phases of the excavation steps. Typical inclinometer recording for each retaining systems except from Type 1 are presented in Figures 14 through 17.

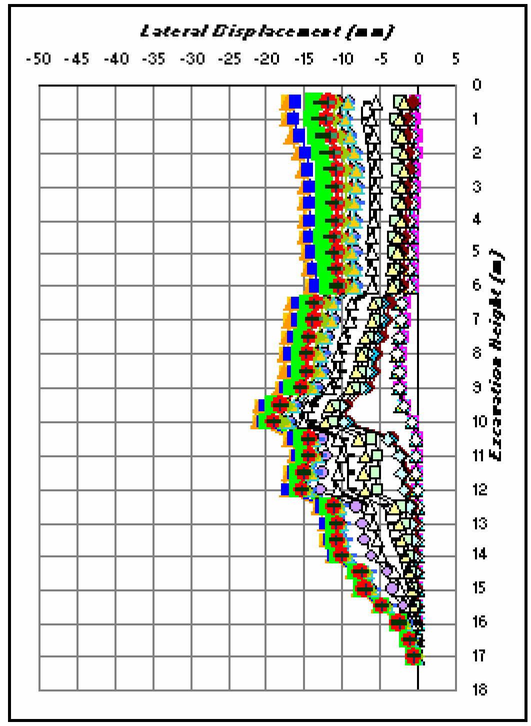

Fig.14. Typical inclinometer readings from Type 2

Table 3. Typical Lateral Displacements of Retaining Types

Retaining Types	Excavation Height, H (m)	Max. Lateral Displacement, δ_{hm} (mm)	Performance Ratio, P_r (δ_{hm}/H, 10^{-3})
Type 2	17.0	21.4	1.26
Type 3	32.5	50.6	1.56
Type 4	18.5	14.0	0.76
Type 5	17.0	20.7	1.22

It is seen that lateral displacement vs. depth relations for retaining walls of Type 2, 3 and 4 are about the same form i.e. maximum displacements have occurred at the surface leading to spandrel type curve. On the other hand, the maximum lateral displacement has occurred at certain depth for retaining wall Type 5 leading to concave type curve. The observed shape of lateral displacement vs. depth curves are in agreement with the previous displacement curves obtained for that specific retaining wall system. The maximum lateral displacement values, δ_{hm} with the corresponding height of excavation, H, together with performance ratios, $P_r = \delta_{hm}/H$ are summarized in Table 3. The measured δ_{hm} values for soil nailed systems described in retaining walls, Types 2, 3 and 4, are between δ_{hm}=0.1 to 0.2%H, depending on the nature of greywacke formation, which is in good agreement with the results reported for similar conditions by Durgunoglu et al (2007). On the other hand δ_{hm} is about 0.12%H for the diaphragm wall which is given as Type 5. Considering the subsoil alluvium layers were sand and gravel, this value is in good agreement with the value obtained by Ou (2006) for excavations performed in sandy soils at Taipei area.

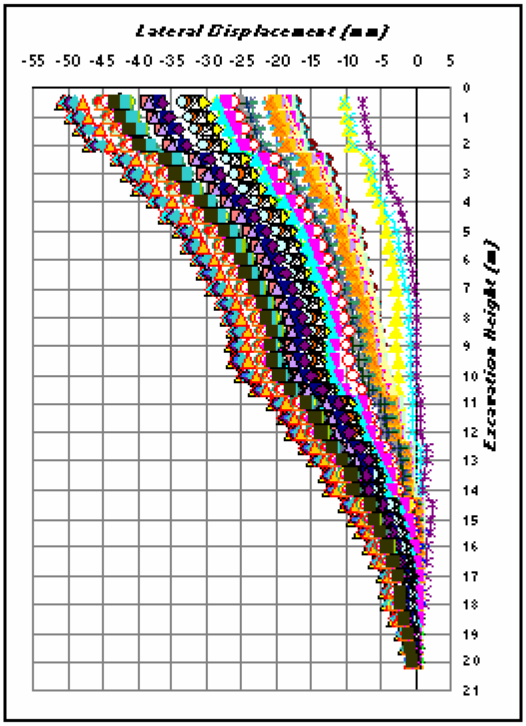

Fig.15. Typical inclinometer readings from Type 3

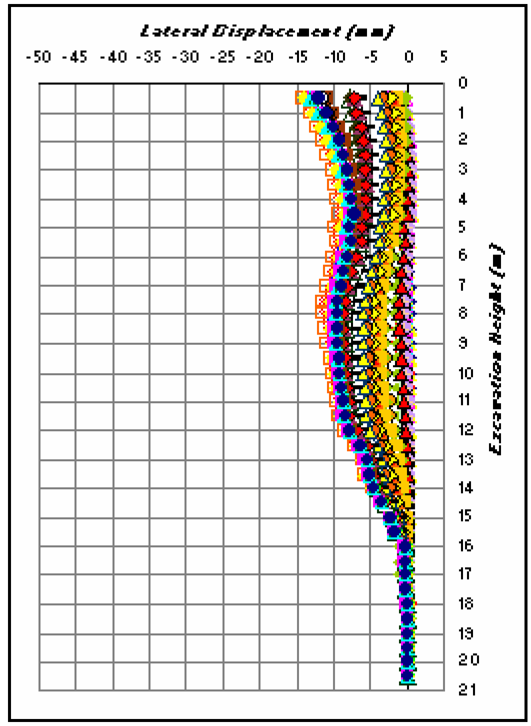

Fig.16. Typical inclinometer readings from Type 4

CONCLUDING REMARKS

An interesting case study is presented for the implications of various retaining wall systems at a specific project site based on the observed complex geological and groundwater conditions. It is shown that, site subsoil conditions, ground water regime, the topography and the architectural elevations and locations imposed by the project have dictated the tailor made retaining wall design and construction for this specific case.

Except some minor variations employed as described in the paper total of five different retaining wall system have been employed having temporary and permanent parts. Due to the high seismicity of the site the permanent walls are preferred to be flexible type, i.e. soil nailing, except from the top part of the Type 1 retaining system which had to employed manually constructed very rigid caissons due to limitations of the space for construction equipment and more strict lateral displacement limitations towards nearby infrastructures. Both flexible and rigid type of retaining wall systems on the other hand have been employed for the temporary structures.

The performance criteria for the walls were based on the observed lateral displacement during excavation. The careful monitoring of the various systems by means of inclinometers have provided the opportunity to implement further measures as in one instance described in the paper. Further, the observed form of lateral displacement vs. excavation depth relationships are in good agreement with the previous findings. In addition, the performance ratios defined as the

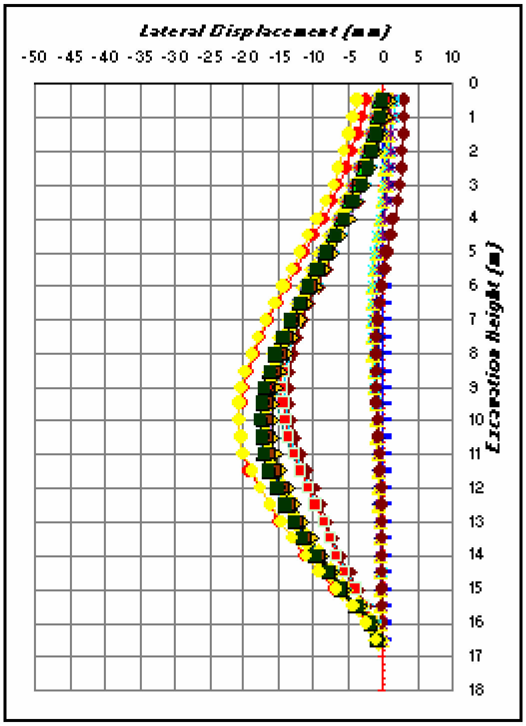

Fig.17. Typical inclinometer readings from Type 5

ratio of maximum lateral displacement to excavation height were within the range of 0.1 to 0.2 % which were below the critical value of 0.3 % imposed in the contract documents.

It is further concluded that with the tailor made approach it was possible to complete the project within budget on time. This engineering approach has deserved to implement the title to the paper "Harmony of Retaining Structures to Various Local Subsoil Conditions" and has proved to be a successful one.

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REFERENCES

Durgunoglu, H.T., Keskin, H.B., Kulaç H.F., İkiz S. and Karadayılar T. [2007]. "*Performance of Soil Nailed Walls Based on Case Studies*", XIV European Conference on Soil Mechanics and Geotechnical Engineering, Paper no. 257, September 24-27, Madrid, Spain.

Durgunoglu, H.T. and Yilmaz, O. [2007]. "*An Integrated Approach for Estimation of Modulus Degradation in Soft Rocks*", Proceedings of 4th ICEGE, Paper no. 1174, June 25-28, Thessalonica, Greece.

Ou, C.Y. [2006]. "*Deep Excavation Theory and Practice*". Taylor & Francis Group, London, UK.

Parsons, T., Toda, S., Stein, K.S., Barka, A. and Dietrich, J.H. [2000]. "*Heightened Odds of Large Earthquake Near Istanbul: An Interaction Based Probability Calculation*", Science, No. 288, pp. 661-665.

Yilmaz, O. and Durgunoglu, H.T. [2008]. "*An Integrated Shallow Seismic Survey for Geotechnical Modeling – A Case Study*", Proceedings of 3rd International Conference on Site Characterization, April 1-4, Taipei, Taiwan.