A Constructability Problem in a Deep Shoring System

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ABSTRACT: *The construction of deep excavations and retaining structures for the deep basements and infrastructures has frequently been implemented in İstanbul. Primary design concerns for underground construction are the protection of adjacent buildings, properties of the project site, and planned structures; thus, the appropriate selection and design of the retaining wall system are critical for the project's success. This case history presents the design and construction details of the watertight retaining wall for the seven levels of basement which required an excavation depth of up to 30.0m. Considering the presence of high groundwater level in the project area, the temporary retaining system initially consisted of secant bored piles with a diameter of 1.0m and 0.2m overlapping, supported by multilevel prestressed anchors along the entire site perimeter. During the construction of the secant piles, it was seen that the mechanical properties of the encountered rock units of Trakya Formation were better than the expected in design phase, and therefore piles could be constructed with a length of approximately 25.0m which was 10.0m shorter than the project lengths but socketed into the hard rock strata. The design of the bottom part of the retaining wall was revised utilizing shotcrete with wire mesh (t=0.35m) and additional prestressed anchors. The plan view of the system is a rectangular shape, the inclinometer measurements have been taken during the construction phases, and the lateral displacements are compared with determined ones in design. In three sides of the retaining system there were very small displacements; however, in one side the excessive displacements are measured upon completion of the excavation. This case study focuses on the additional design revisions with their results on the inclinometer readings as the construction proceeds.*

KEYWORDS: Deep Excavation, Retaining Wall, Secant Bored Pile, Prestressed Anchors, Shotcrete, Inclinometer Measurements

SITE LOCATION: Geo-Database

INTRODUCTION

In highly urbanized central areas of metropolitan cities, such as downtown İstanbul, deep excavations are a challenging geotechnical engineering problem to solve. The high prices per square meter of rentable areas causes architectural designers to consider the underground part of the structures more efficiently, e.g., by utilizing many basements. Basements are especially attractive for the investors who seek the most economical solutions, which are generally limited by the excavation depth and utilized retaining system.

In the design and construction of deep excavation support systems, high groundwater levels—together with variable soil and rock profiles—represent the main challenges. Increasingly, secant piling techniques become the preferred technical solution for a retaining system in these cases. Recently, improvements in drilling equipment, tooling, and procedures allow economical solutions for constructing deep, overlapped pile systems under problematic ground conditions. This paper presents a case

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study of the design and construction details of the waterproof retaining wall for the seven levels of basement which required an excavation depth of 30.0m.

PROJECT INFORMATION

The subject site is at the Dolapdere District of İstanbul, covering an area of approximately 3397.0m^2 with additional green areas. The project area is located at Dereboyu Street, and is surrounded by Can Eriği, Yeni Bostan, and Keresteci Ali Streets. After the demolition of the old existing building, plans are made for a contemporary gallery with 7 stories and 7 basements to be constructed. The gallery covers approximately 1214.0m^2 area, with 18621.0m^2 total construction area. According to the conveyed information the architectural ±0.00m elevation will be as +22.00m. The foundation depth is 31.2m and base elevation will be -9.15m. The architectural view and cross section of the building is given in Figure 1.

Figure 1. Architectural View and Section of Building.

SOIL INVESTIGATION PHASE

Considering the planned structures and subsurface conditions expected at the subject site, a subsurface investigation program was executed, consisting of rotary borings, Standard Penetration Tests (SPT), in situ packer permeability tests, collection of geotechnical (soil and rock) samples with related laboratory tests, and determination of groundwater levels together with extensive geophysical investigations.

Scope of Site Investigations

The scope of the site investigations consisted of five boreholes having a total depth of 224.0m. In addition, to determine the permeability characteristics of the rock units of the Trakya formation, packer permeability tests were performed within the selected boreholes (BH-04 and BH-5). Site investigation was carried out between the dates of 17.01.2014 and 21.02.2014. The summary table for the boreholes is presented in Table 1.

Table 1. Summary of Boreholes.

Borehole No	Depth (m)	Elevation (m)	Coordinates		Date started	Date completed	Groundwater Table (m)	Groundwater Elevation (m)
			North	East				
BH-01	40.50	21.86	414148.957	4545788.194	17.01.2014	24.01.2014	5.20	16.66
BH-02	40.00	21.89	414150.609	4545812.198	24.01.2014	27.01.2014	4.77	17.12
BH-03	41.00	22.21	414171.511	4545819.957	29.01.2014	05.02.2014	4.20	18.01
BH-04	50.00	22.45	414161.812	4545835.138	6.02.2014	14.02.2014	4.12	18.33
BH-05	52.50	22.08	414188.780	4545805.873	18.02.2014	21.02.2014	4.20	17.88

General Geology of the Project Site

The subject site is in the İstanbul sheet of Turkey Geology Map with 1/500,000 scale. It is located in the İstanbul peninsula. Geological units within the İstanbul peninsula start with Early Paleozoic and continue conformably from Silurian through Lower Carboniferous. This sequence is overlaid by the Triassic sedimentary rocks uncomformably. Paleozoic aged units generally comprise detrital, carbonaceous rocks of Dolayoba, Kartal, Baltalimanı, and Trakya Formations. The general geology map is provided in Figure 2.

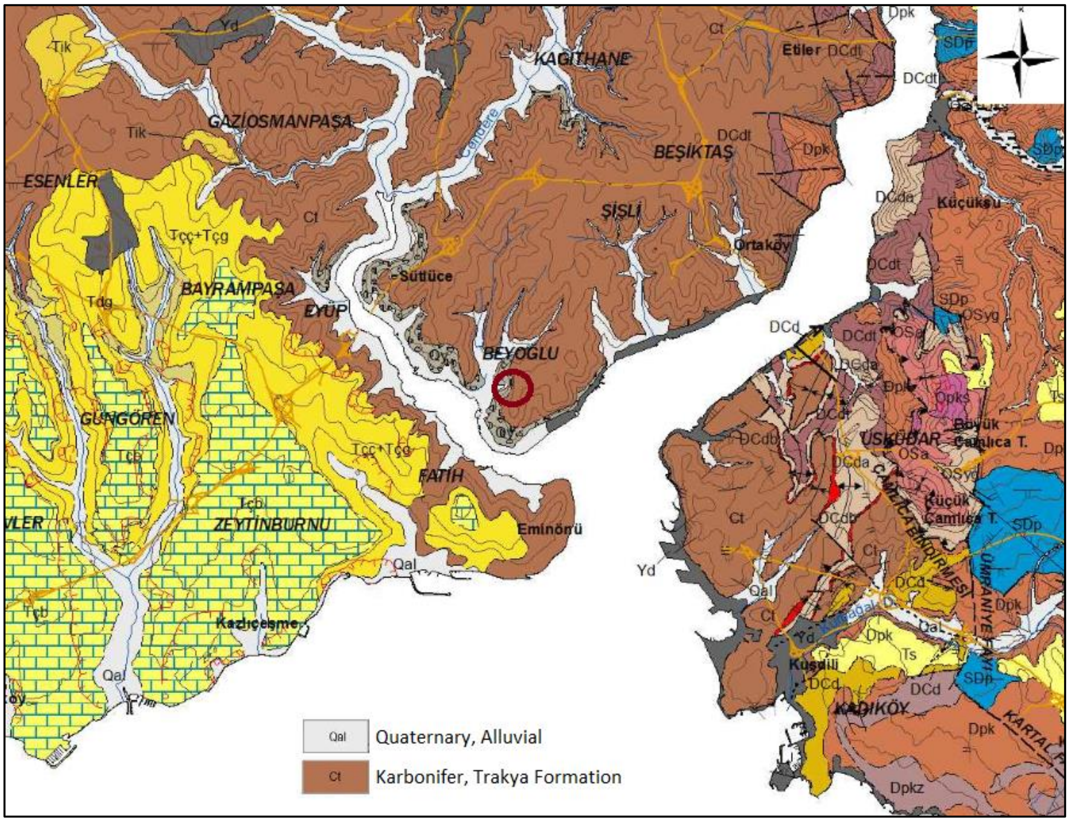

Figure 2. General Geology Map of Project Site (İstanbul Geology Map, İstanbul Metropolitan Municipality, 2011).

Trakya Formation is a succession of shale, siltstone, sandstone subordinate conglomerate, and carbonates, bounded by lydites at the base and limestone at the top. It is very intensely folded, faulted, fractured, and is also weathered, which makes it well developed along discontinuities. Hercynian orogeny of Late Paleozoic resulted in the regional uplift and erosion of Paleozoic rocks.

Local Geology and Evaluation of Soil Profile

Concrete was encountered during drilling operations beneath the existing building slab, 3.5m height basement floor, and approximately 0.6cm basement floor. Beneath the basement floor, generally Quaternary aged alluvium layers consisting of gravel, clay, silt, and sand units were encountered. Carboniferous aged Trakya Formation rock units were observed beneath the alluvium layers.

With the exception of BH-4, encountered alluvium unit thickness varied between 3.7m and 8.9m. Alluvium layers were light brown, moist, fine-medium subangular - subrounded gravelly, with a medium plasticity very stiff-hard clayey, silty and fine-medium grained medium dense-dense sand. FeO and MnO traces were observed on gravel surfaces.

In all boreholes, Carboniferous aged Trakya Formation rock units underlaid the alluvium layers. Claystone-mudstone units were brown-dark gray, slightly-moderately weathered, strong-very strong, very poor-poor rock quality designation (0-38% RQD), and closely-very closely fractured. FeO and MnO coating was observed on joint surfaces. Secondary quartz-calcite veins and pyrite crystallization were observed on joint surfaces. Sandstone units were light gray, slightly-moderately weathered, moderately weak to weak-moderately strong, very poor-poor rock quality designation (0-36% RQD), closely-very closely fractured, and contained secondary calcite veins.

The daily groundwater readings were taken during site investigation and after the completion of site investigation. To monitor the groundwater levels, PVC pipes were installed, within the borings BH-1, BH-2, and BH-3. The groundwater table depth was found to range between 4.1m and 5.2m (January – March 2014).

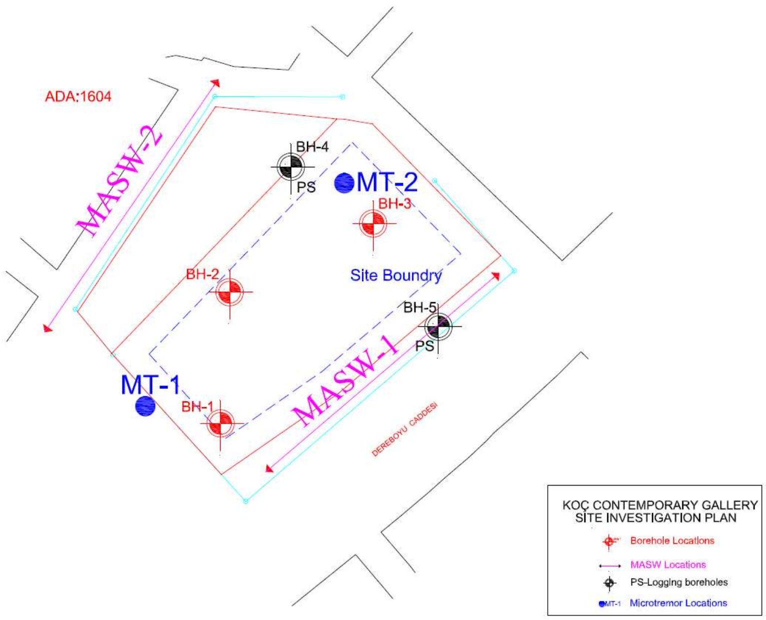

Figure 3a. Soil Investigation Layout.

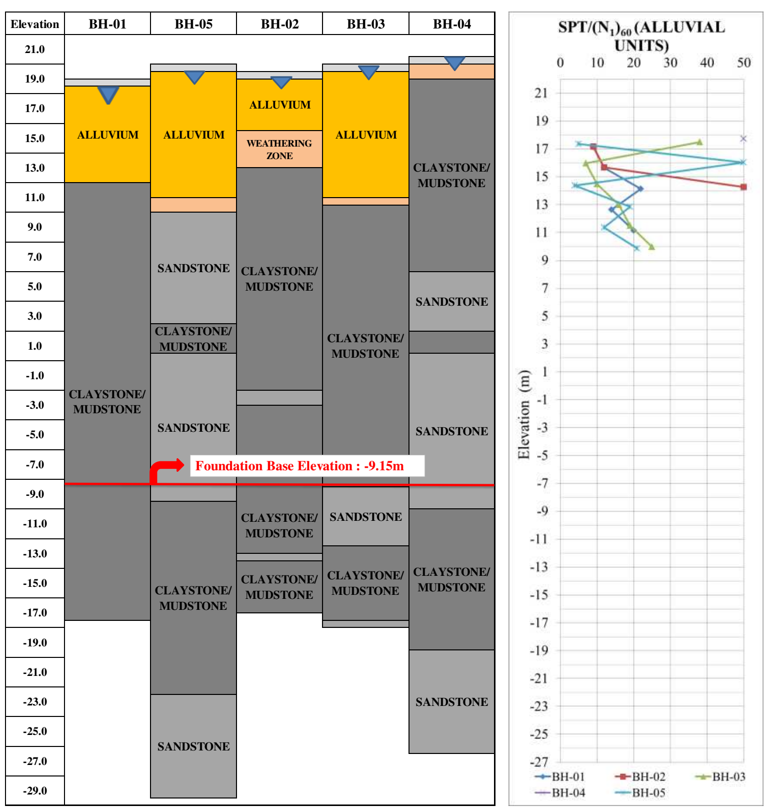

Figure 3b. Soil Model, Groundwater, and Corrected SPT/$(N_1)_{60}$ values of Alluvial Units.

To determine the permeability characteristics of the Trakya formation rock units, the packer permeability tests were conducted within the suitable rock conditions in BH-4 and BH-5 boreholes. The summary of obtained results and the variation of the Lugeon coefficient versus elevation are given in Figure 4. The Lugeon values are L_v=10 to 50, indicating permeable to highly permeable formation.

Borehole No	Depth (m)	Lugeon Coefficient	Permeability
BH-4	16.5	50	Highly Permeable
BH-4	19.5	49	Highly Permeable
BH-4	25.5	11	Permeable
BH-4	33.0	11	Permeable
BH-4	46.5	39	Highly Permeable
BH-5	30.0	34	Highly Permeable
BH-5	39.0	17	Permeable

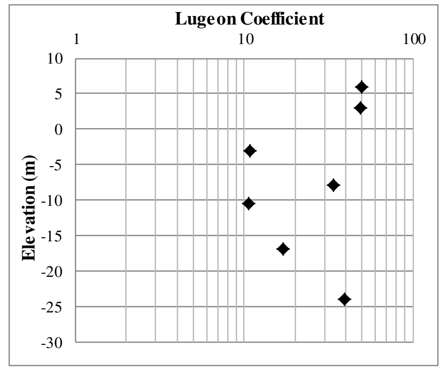

Figure 4. Packer Permeability Test Results.

Laboratory Tests

The following soil laboratory tests were performed: sieve analysis, hydrometer tests, specific gravity, Atterberg limits, natural water content, consolidation, and undrained unconsolidated triaxial tests. Point load and uniaxial compression tests (with deformation modulus) were performed on obtained rock core samples. The results are summarized in Table 2 and Figure 5.

Table 2. Soil Mechanics Laboratory Test Results.

Borehole No	Specimen No	Depth	Water content Wn (%)	Atterberg's limits			Sieve Analysis		Hydrometer	Triaxial Compression Test	Consolidation Test			Specific Gravity	Classification
		(m-m)		LL	PL	PI	+No.10 (%)	-No.200 (%)	-2 µm (%)	cu (kPa)	σp (kPa)	Cc	Cr	Gs	USCS
BH-1	SPT-2	6.00-6.45	21	26	16	10	18	45	-	-	-	-	-	-	SC
BH-1	SPT-4	9.00-9.45	21	37	16	21	4	74	33	-	-	-	-	-	CI
BH-1	SPT-5	10.50-10.95	23	43	18	25	3	75	-	-	-	-	-	-	CI
BH-2	SPT-1	4.50-4.95	16	24	18	6	42	23	-	-	-	-	-	-	GM
BH-2	SPT-2	6.00-6.45	18	29	17	12	26	34	-	-	-	-	-	-	SC
BH-3	SPT-1	4.50-4.95	22	NP	NP	NP	55	13	-	-	-	-	-	-	GM
BH-3	UD-1	6.50-7.00	11	27	16	11	43	38	-	-	270	0.1171	0.0074	2.69	GC
BH-3	UD-2	9.70-10.10	14	35	18	17	29	37	17	25	-	-	-	2.71	SC
BH-3	SPT-6	12.00-12.45	20	34	17	17	24	43	-	-	-	-	-	-	SC
BH-5	SPT-1	4.50-4.95	17	35	15	20	51	28	-	-	-	-	-	-	GC
BH-5	UD-1	7.00-7.50	16	30	15	15	25	46	-	50	-	-	-	-	SC
BH-5	SPT-4	9.00-9.45	19	30	16	14	31	31	-	-	-	-	-	-	SC
BH-5	UD-3	11.50-12.00	16	30	16	14	49	21	-	-	-	-	-	-	GC

-No.200(%) : percent by weight passed on the #200 sieve
LL(%) : Liquid limit
PL(%) : Plastic limit
NP : Non-Plastic specimen
PI(%) : Plasticity index
USCS : Unified Soil Classification System

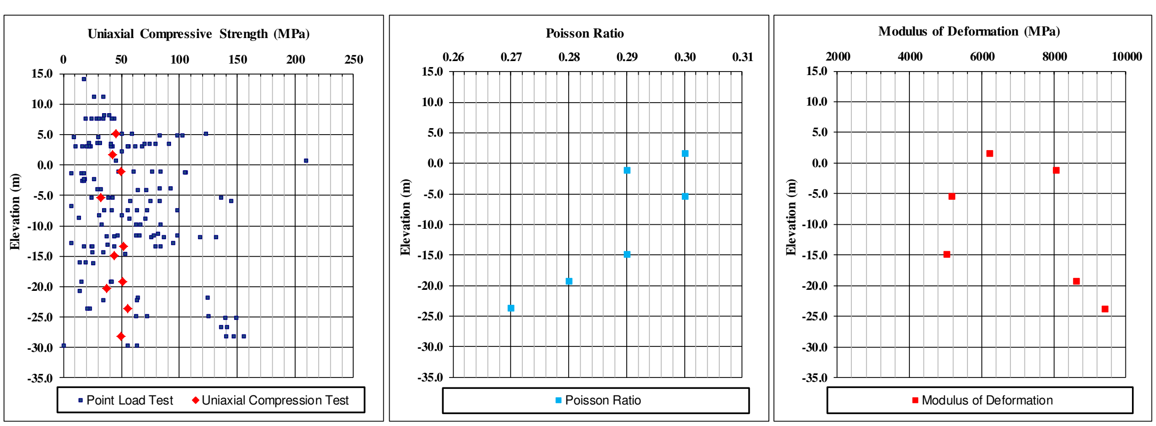

Figure 5. Rock Mechanics Laboratory Test Results.

RETAINING WALL DESIGN PHASE

Considering the depth of excavation (~31.0m), encountered soil and groundwater conditions, and the existing structures, the secant bored piles supported by multi-level prestressed anchors were selected as retaining system along the entire site perimeter. The underlying reason for this selection is to form a structurally safe retaining wall and impermeable barrier against water.

The retaining system is designed by analyzing three different sections as per the location and surcharge loads.

Table 3. Summary of the Critical Sections.

Section	*Top Elevation*	*Excavation Base Elevation*	*Max. Excavation Depth (m)*	*Surcharge (kPa)*
Section 1	*+20.40*	*-9.30*	*29.70*	*80*
Section 2	*+20.40*	*-9.30*	*29.70*	*60*
Section 3	*+20.40*	*-9.30*	*29.70*	*15*

In all sections, the diameters of the secant piles are 1.0m with 0.2m overlapping. The lengths of the piles are 34.7m. However, due to the lack of sufficient construction area, the diameter of the secant piles was reduced to 80cm with 0.1m-0.2m overlapping where the transformer building exists. The plan view of the retaining wall is presented in Figure 6.

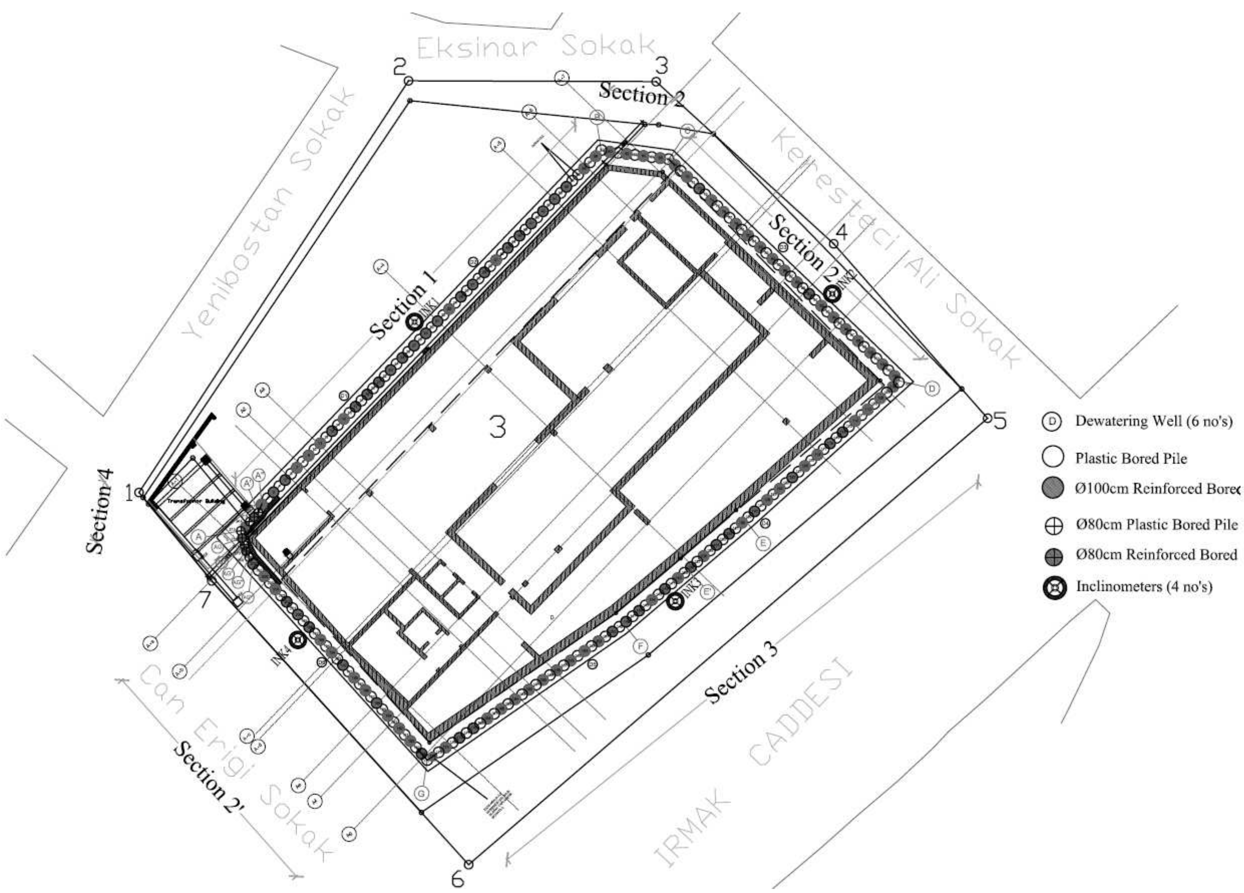

Figure 6. The Plan View of the Retaining System.

The retaining system was designed using PLAXIS 2D, an industry standard finite element software which can assess the displacements and settlements of soil and rock. Recommended soil parameters given in the Soil Investigation Report (April 2014) were reviewed, and the geotechnical parameters relevant to the calculations used for the design of the retaining system are summarized in Table 4. The alluvial unit is modelled with Hardening Soil Model since it is the appropriate model for the unload-reload problems of soil units. The Trakya Formation rock unit is modelled with Mohr Coulomb. The shear strength parameters of the MC model are defined considering the corresponding Hoek-Brown shear strength for representative UCS, GSI, and mi values.

Table 4. Soil Parameters.

Parameters	***Alluvial Unit***	***Trakya Formation***
Soil Model	*HS*	*MC*
Drainage Type	*Drained*	*Drained*
Unit Weight, γ_n / γ_{sat} (kN/m^3)	*18 / 19*	*22 / 23*
Initial void ratio, e_{ini}	*0.5*	*0.5*
Permeability Coef., k (m/day)	*0.12*	*0.25*
R_{inter}	*0.9*	*0.9*
E_{50}^{ref} *(kPa)*	*15,000*	-
E_{oed}^{ref} *(kPa)*	*15,000*	-
E_{ur}^{ref} *(kPa)*	*45,000*	-
Modulus of Elasticity, E (kPa)	-	*250,000*
Poisson ratio, ν	-	*0,3*
Cohesion, c (kPa)	*1*	*20*
ϕ' (°)	*28*	*35*

Three types of anchors were used for the calculations. Type 1 has 4.0m-20.0m free length, 8.0m grout length, at a 15º angle from the horizontal placed with 3.20m spacing in alluvial units. The other two are located in the rock unit. The first has 20.0m free length, 8.0m grout length, at a 15º angle from the horizontal (Type 2) placed with 1.6m spacing. The second has 17.0m-22.0m free length, 10.0m grout length, at a 15º angle from the horizontal (Type 3) placed with 1.6m spacing. Ten (10) levels of prestressed anchor rows are considered in the analysis.

Table 5. Detail of the Anchors.

Anchor No	***Anchor Type***	***Anchor Elevation (m)***	***Anchor Length (m)***	***Number of Rope***	***Prestressed Load (t)***
1	*Type 1*	*16.4*	*12-28*	*3x0.6"*	*45*
2	*Type 2*	*13.4*	*28*	*4x0.6"*	*60*
3	*Type 2*	*10.4*	*28*	*4x0.6"*	*60*
4	*Type 3*	*7.4*	*32*	*5x0.6"*	*75*
5	*Type 3*	*4.9*	*32*	*5x0.6"*	*75*
6	*Type 3*	*2.4*	*28*	*5x0.6"*	*75*
7	*Type 3*	*-0.1*	*28*	*6x0.6"*	*90*
8	*Type 3*	*-2.6*	*23*	*6x0.6"*	*90*
9	*Type 3*	*-5.1*	*27-29*	*6x0.6"*	*90*
10	*Type 3*	*-7.6*	*24*	*6x0.6"*	*90*

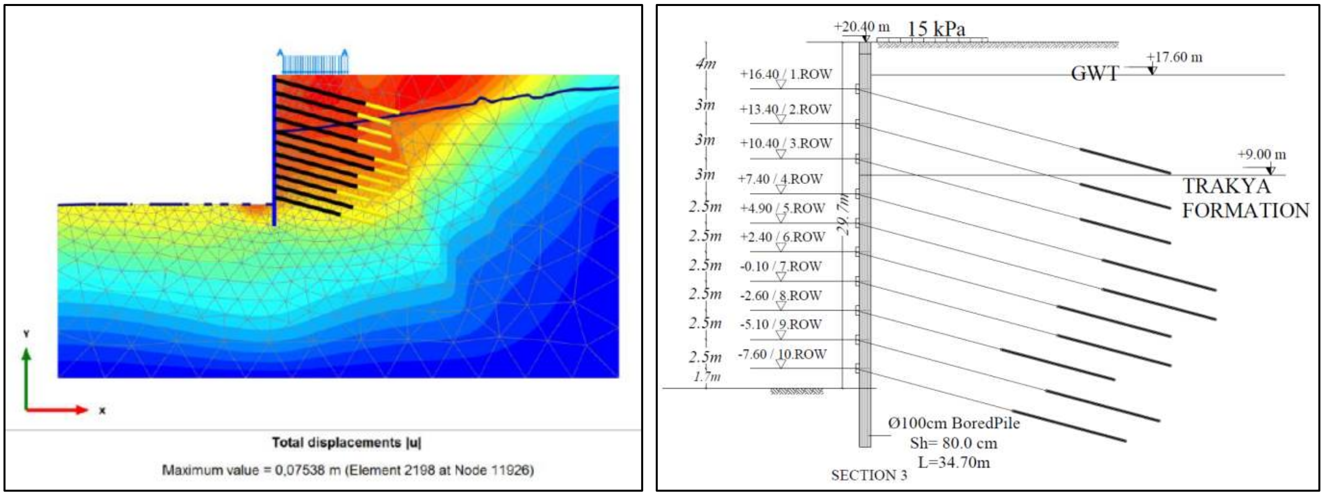

Figure 7. Section View of the Retaining System.

To reduce the groundwater elevation behind the secant bored piles, six (6) dewatering wells were installed outside of the excavation area at a distance of about 1.0m from the face of the secant piles.

As can be seen from Figure 6, four inclinometer boreholes were drilled around the project site and used to measure lateral displacements of the soil/rock mass behind the secant pile wall. Measurements for inclinometers were taken for each construction phase and at frequent intervals during the construction period.

CONSTRUCTION PHASE AND ENCOUNTERED PROBLEMS

During the pile installation, it was observed that the strength of the Trakya Formation rock units fared better than expected, especially below the 25.0m depth (-4.6m elevation). The progress rate of the drilling unit was controlled regularly for each pile; it is shown that the progress rate was less than 1.0m/hour after 25.0m depth. Additionally, magmatic origin diabase blocks were encountered after a certain depth.

Diabase, also called Dolerite, is a fine- to medium-grained, dark gray to black intrusive igneous rock. It is extremely hard and tough and compositionally equivalent to gabbro and basalt but is texturally between them. It occurs mostly in shallow intrusions (dikes and sills) of basaltic composition. It grades to basalt when it solidifies rapidly and to gabbro when more time is provided for the crystals to grow.

Figure 8. Encountered Diabase Units.

Considering the aforementioned reasons, due to the challenges encountered in such hard units, the design of the retaining system was revised. The length of secant bored piles was reduced to 25.0m and the remaining part of the excavation was designed to be supported by shotcrete with wire mesh having thickness, t, equal to 35cm.

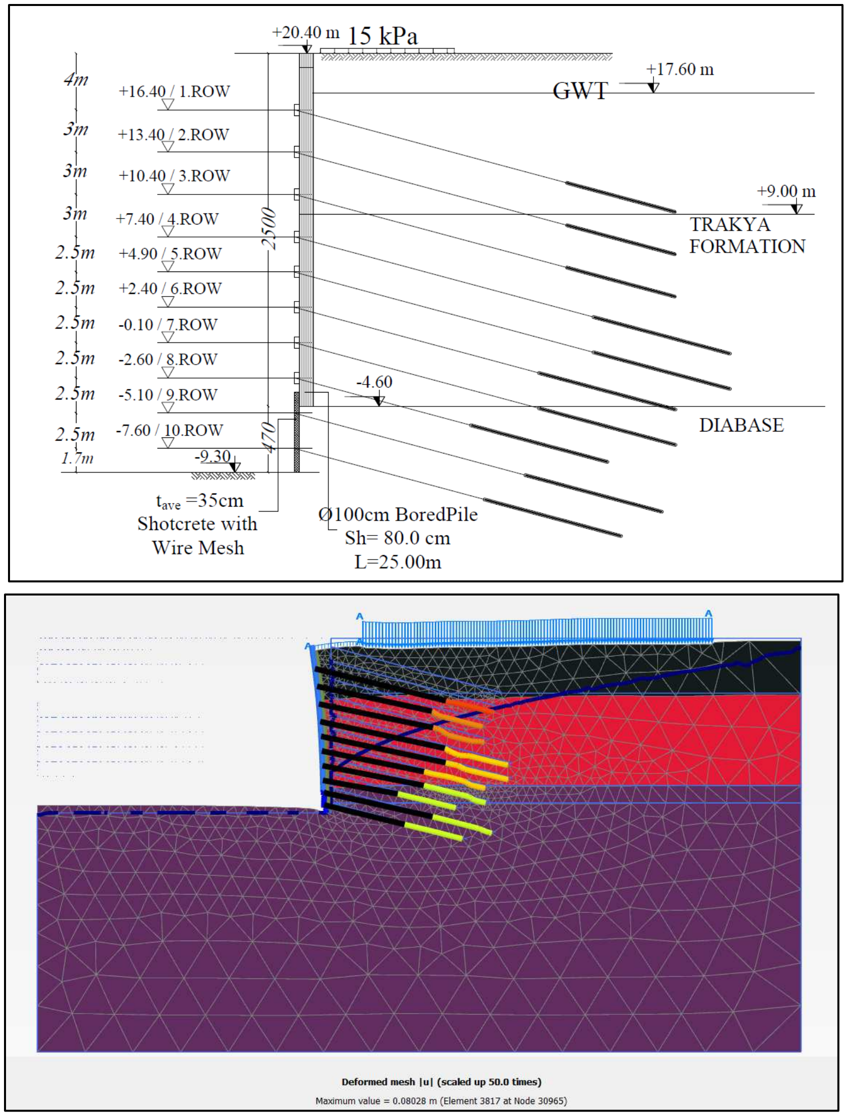

Figure 9. Section View of the Revised Retaining System.

Inclinometer Readings

With the commencement of the construction activities at the project site, inclinometer measurements were also collected at weekly intervals. During the design phase, the limit values for the lateral displacements were defined as:

- Critical Limit: D_f x 0.0015 (D_f: Excavation Depth)
  Considering the excavation depth as 30.0m, the upper limit displacement is 45mm. If the displacements exceed this limit, reading frequency should be increased, mechanism of the movement should be observed, and the critical slide surface should be determined. If the displacement increase is not constant between each excavation level, mitigation measures (such as temporary toe filling or additional anchors) should be discussed.

- Red Limit: D_f x 0.003 (D_f: Excavation Depth)
  Considering the excavation depth as 30.0m, the red limit displacement is 90mm. If the displacements exceed this limit or displacement increase is not constant between each excavation level, the implementation of temporary toe filling or additional anchors should be considered.

In the excavation phase of Section 3, the lateral displacement exceeded the critical limit (45mm) in the middle of August 2016. Following immediate discussion by the client, contractor, and project engineers, mitigation measures were determined.

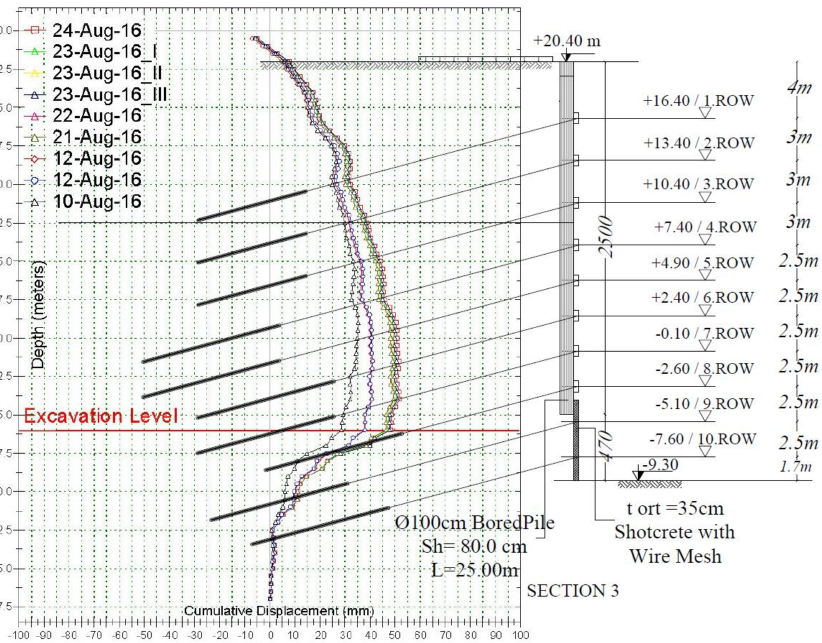

Figure 10. Encountered Excessive Displacements on Section 3.

In the second half of August 2016, toe filling was carried out up to the middle of 6th and 7th row prestressed anchor levels (19.25m depth, +1.15m elevation), and six additional prestressed anchors (each 32.0m in length) were installed.

At the beginning of September 2016, excessive lateral displacements were observed again in the inclinometer readings. An additional ten prestressed anchors, each 32.0m in length, were also implemented between the 7th and 8th row anchor levels. A frontal view of Section 3 is shown in Figure 11.

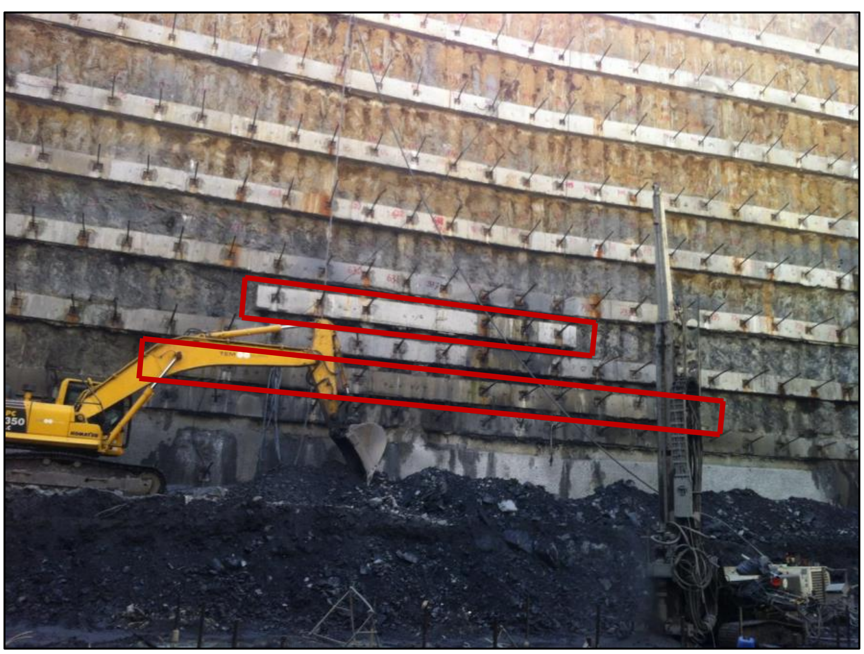

Figure 11. Additional Anchors on Section 3.

During the prestressing of the additional anchors on September 17th, 2016, the lateral displacement was measured as 58.0mm. Due to the stage excavation of 9th row anchors, the lateral displacement was increased to 62.0mm on September 26th, 2016. From the completion of the stage excavation till October 4th, 2016, inclinometer readings were regularly collected and it was observed that the increase of the lateral displacement stopped. The graph of the measured lateral displacements between the -22.5m and -25m elevations of Section 3 is presented in Figure 12.

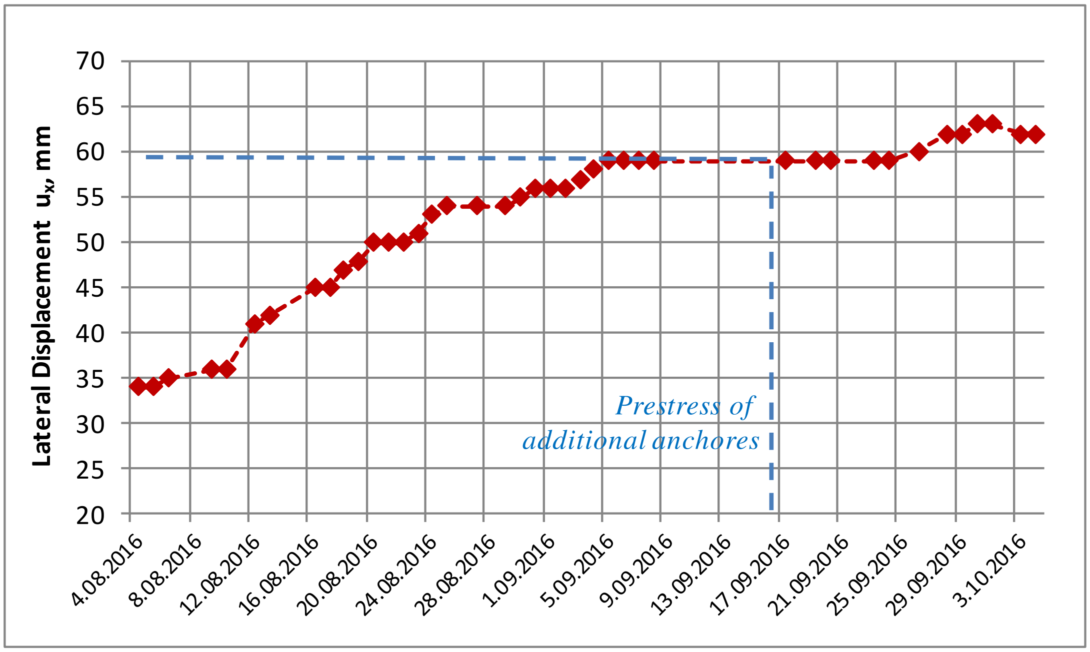

Figure 12. Section 3 Lateral Displacements.

In general, inclinometer readings were regularly collected during the entire construction phase and it was observed that the obtained lateral displacements were less than the design values.

Table 6. Lateral Displacements of Design and Construction Phases.

	Section 1 Inclinometer 1	Section 2 Inclinometer 2	Section 3 Inclinometer 3	Section 2 Inclinometer 4
Calculated Lateral Displacement from Finite Element Analysis (mm)	90	80	**65**	80
Lateral Displacement Obtained from Inclinometer Measurements (mm)	15	8	**62**	20

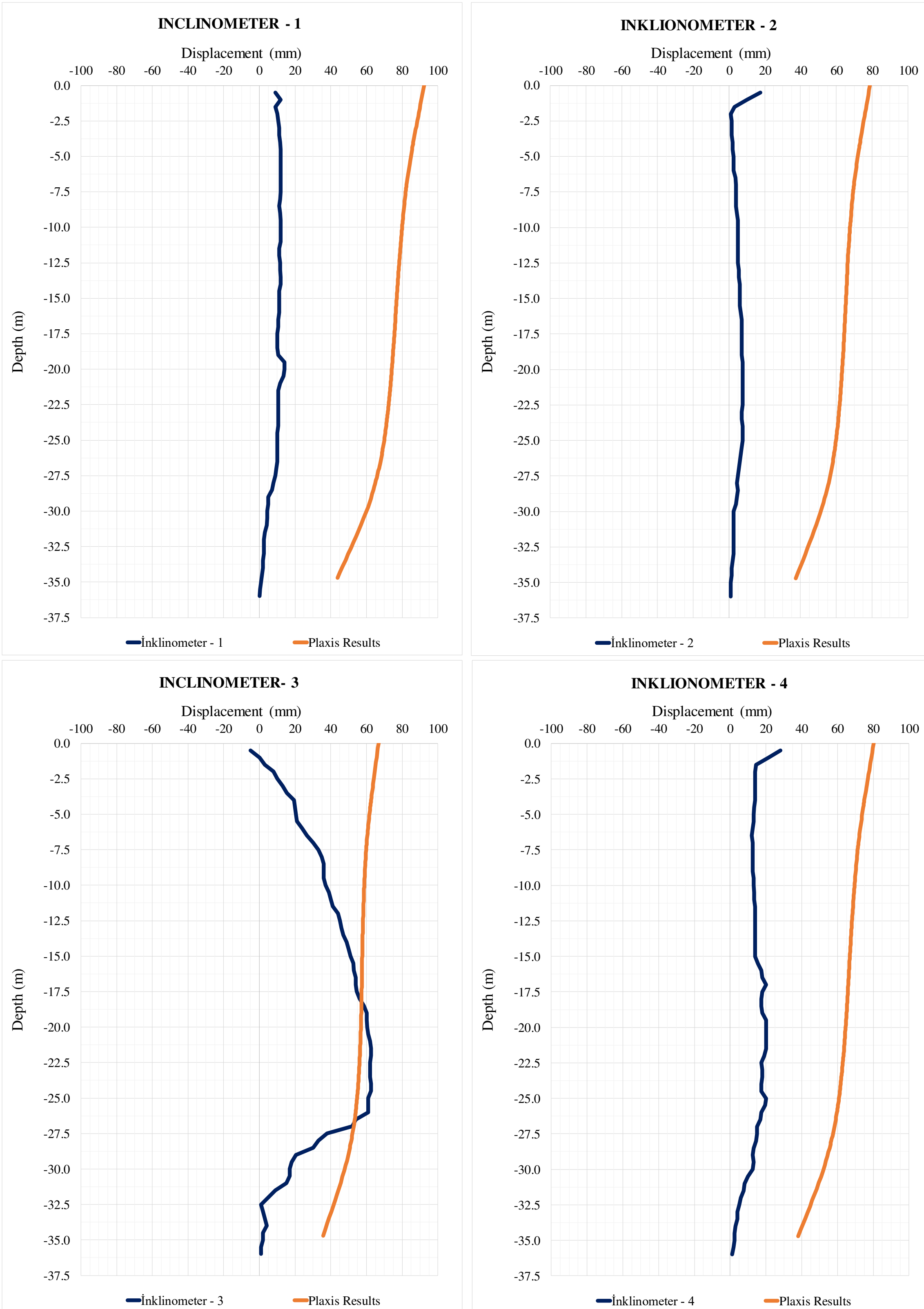

Figure 13. Inclinometer Readings.

Figure 14. An Aerial View of the Project Site.

CONCLUSION

This paper describes the secant pile retaining wall system from the soil investigation phase to the design and construction phases. From the information presented, the following conclusions can be drawn:

- The measured lateral displacements obtained from inclinometer readings during the construction phase are less than the values calculated during the design phase.
- The soil and rock parameters were analyzed in detail during the soil investigation phase and were determined to meet the most critical conditions.
- Considering the mechanical properties and dip direction of the rock units, the most critical section is Section 3 where the encountered lateral displacements are almost the same as those of the design values.
- The measurement of lateral displacements provided some comfort with respect to the system behavior. Additional prestressed anchors were constructed to limit a further increase in lateral displacements.

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