CAUSES, ANALYSIS AND STABILISATION OF A COASTAL SLIDE, SOUTH OF SINOP, TURKEY

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ABSTRACT

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The purpose of this study is to determine the causes, residual shear strength parameters of the material involved at the Sinop 33 Evler Landslide, and to establish appropriate stabilisation technique. In order to accomplish these tasks, borehole data, inclinometer measurements, standard penetration test measurements, laboratory test results provided by Yüksel Proje Uluslararası A.Ş. are taken into consideration.

There are two main lithological units involved at the landslide. These are the Saraycık and the Sarıkum formations. Based on the field observations, field and laboratory tests results, Saraycık formation consists of CL, CH, and MH type of soils, which can be described as stiff clay. The clay minerals include considerable amount of smectite. Sarıkum Formation consists of SP, SW, and SM type soils at the upper parts, and it can be described as medium dense sand. At the bottom MH, CL, CH type of soils are dominant, and described as medium stiff clay.

Back analyses are performed along four profiles from the slide investigated. The inclinometer measurements revealed a circular failure surface. The failure surface is mostly located within the Saraycık formation. The result from the back analyses revealed that at the time of failure the residual shear strength parameters were determined as $c_r = 0$ and $\phi_r = 10.3^\circ$.

By considering the size of the landslide, depth of failure surface and the residual shear strength parameters, it was decided that the most appropriate stabilisation technique is rock buttress at the toe of the slide.

Static and pseudostatic stability analysis were performed using Bishop Rigorous Method and it is seen that the factor of safety is greater than 1.2 under possible earthquake loading.

Keywords: Back analysis, inclinometer, residual shear strength parameters, rock buttress, soil slope stability

ÖZ

BİR KIYI HEYELANININ NEDENLERİ, ANALİZİ VE DURAYLILIĞI, SİNOP GÜNEYİ,TÜRKİYE

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Bu çalışmanın amacı, 33 Evler Heyelanının nedenlerini, zeminin heyelanda artık makaslama dayanımı parametrelerini ve en uygun iyileştirme tekniğini belirlemektir. Heyelanın mekanizmasını belirlemek için Yüksel Proje Uluslararası A.Ş. tarafından yapılan sondajlara ait veriler, inklinometre ölçümleri, standart penetrasyon deneyleri ile zemin mekaniği labaratuvar deney sonuçları dikkate alınmıştır.

Heyelan bölgesinde iki jeolojik birim görülmektedir. Bunlar, Saraycık ve Sarıkum formasyonlarıdır. Saha gözlemlerine, saha ve laboratuvar deney sonuçlarına göre Saraycık formasyonu, CL, CH, MH tipi zeminlerden oluşur ve katı kil olarak tanımlanabilir. Formasyon, önemli miktarda simektit tipi kil minerali içerir. Sarıkum Formasyonu ise, üst kısımlarda SP, SW, SM tipi zeminlerden oluşur ve orta sıkılıkta kum olarak tanımlanabilir, alt kısımlarda MH, CL, CH tipi zeminler egemen olup, orta katılıkta kil olarak tanımlanabilir.

Geriye dönük şev duraylılığı analizleri dört adet profil boyunca yapılmıştır. İnklinometre ölçüm sonuçlarına ve saha gözlemlerine göre yenilme modeli dairesel kayma türünde olup, yenilmenin tamamına yakını Saraycık formasyonu içerisindedir. Geriye dönük analiz sonuçlarına göre kayma sırasında artık zemin makaslama dayanımı parametreleri $c_r = 0$, $\phi_r = 10.3^{\circ}$olarak belirlenmiştir.

Heyelanın büyüklüğü, kayma dairesinin konumu ve artık makaslama dayanımı parametreleri göz önüne alındığında, en uygun iyileştirme tekniğinin kaya topuk dolgusu olduğu belirlenmiştir.

Statik ve dinamik yükler dikkate alınarak, duraylılık analizleri, Bishop Karmaşık Yöntemi ile yapılmıştır. Analiz sonuçlarına göre, olası deprem durumunda güvenlik katsayısı 1.2 den büyüktür.

Anahtar kelimeler: Geriye dönük analiz, inklinometre, artık makaslama dayanımı parametreleri, kaya topuk dolgusu, zemin şev stabilitesi.

To My Family

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CHAPTER 1

INTRODUCTION

1.1. Purpose and Scope

The purpose of this study is to investigate the causes and mechanism of a slope failure which occurred at the Sinop 33 Evler locality and to suggest a proper stabilisation technique.

The study has been carried out in three stages. In the first stage an extensive literature survey was performed. This survey included collection of geological and geotechnical data related with the area, and review of slope stability analysis methods and stabilisation techniques.

The second stage of the study included evaluation of the field and laboratory work performed by Yüksel Proje Construction Company. During the field work, seven boreholes were drilled and, through five of them inclinometer measurements were taken by Yüksel Proje Construction Company, to determine the position of the failure surface. On the drill hole samples some soil laboratory tests were also performed.

The third stage of the study included back analysis of the failed slope to determine the residual shear strength parameters mobilising along the failure surface. Based on shear strength parameters determined through back analysis, the effectiveness of the proposed stabilisation technique has been tested under both static and earthquake loadings.

1.2. Location and Accessibility

The study area is located at the 33 Evler region in Sinop and the site is about one kilometer away from Sinop Bus Station (Figure 1.1). The site is accessible throughout the year by the Samsun – Sinop, Kastamonu - Sinop highways.

1.3. Previous Studies

Sinop Peninsula and its vicinity was examined mainly for petroleum occurrences and for nuclear power plant site selection purposes.

Salamon - Calvi (1936), and Blumental (1948) performed geological studies at the Sinop area. Erinç and İnandık (1955) studied the geomorphological properties of the Sinop region. Bangley (1959), determined the stratigraphy of the region from Jurassic to Pliocene for petroleum investigation purpose. Gayle (1959) also examined the region for petroleum occurrences. Ketin and Gümüş (1962) carried out petroleum geological studies in the region and also described Pliocene formations around Sinop Harbour. Akkan (1975) studied the geomorphology of the Sinop Peninsula and he stated that the peninsula is composed of Plio-Quaternary marls and sandstone series. Özsayar (1977) further subdivided the Miocene formations by using mollusc fauna. Çoşkun (1978) measured paleo - current directions of the Neogene deposits. Gül and İplikçi (1979)

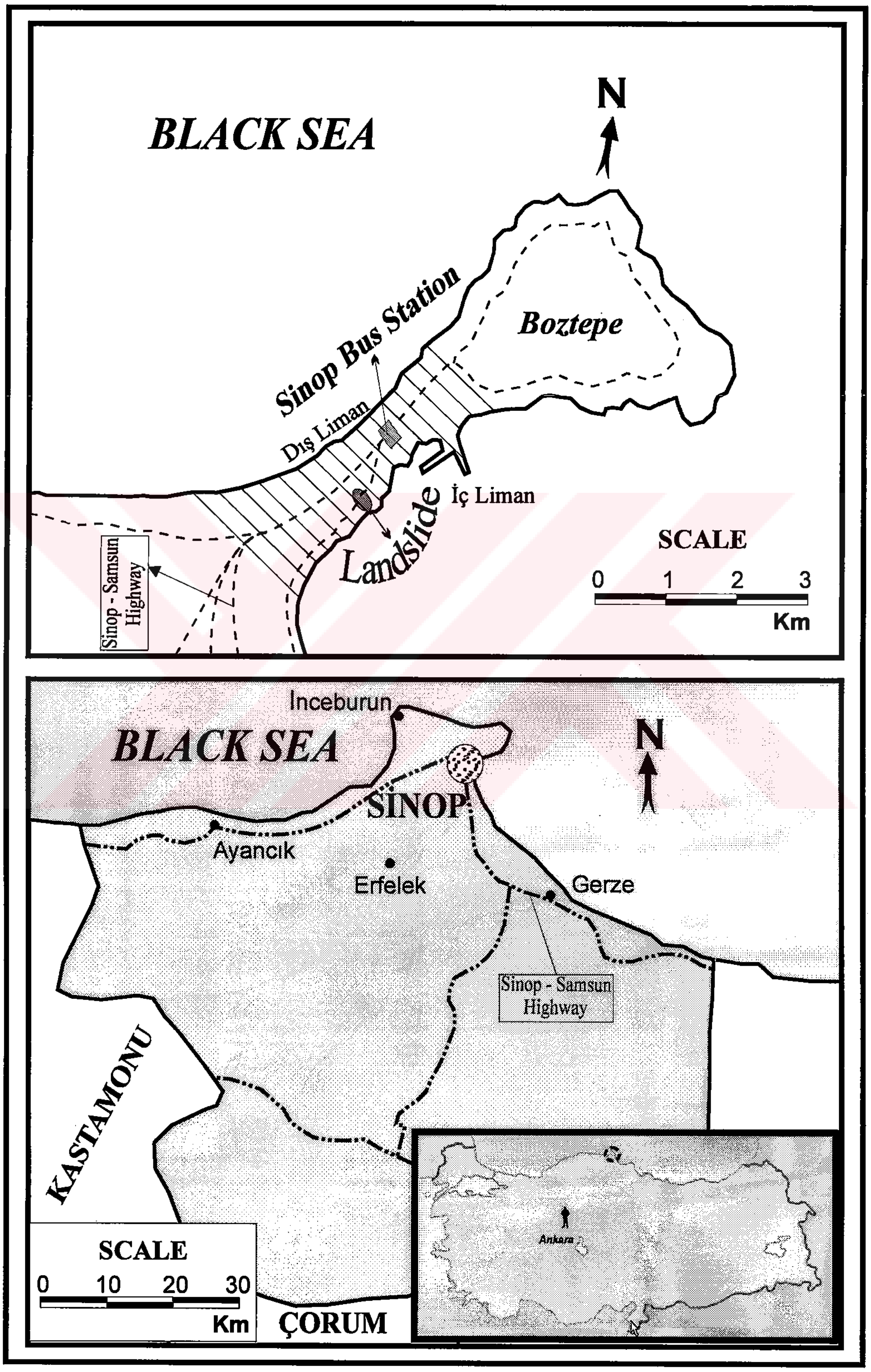


Figure 1.1. Location map of the study area

studied the presence and activity of faults of the Sinop Peninsula. Akarsu and Aydın (1979), Aydın and Serdar (1980) assigned Plio – Quaternary age to the deposits located at the west of the Sinop Peninsula and they further stated that the thickness of these deposits exceeds 700 meters. Doyuran, (1983) complied regional geology of the Sinop peninsula. Doyuran and Tuncer (1983), complied site area investigations for the second Nuclear Power Plant. Doyuran and Erdik (1983) studied the neotectonics and seismicity of the Black Sea for the site selection of the Nuclear Power Plant. Sütçü et al (1982), Barka et al (1983), (1985) described Miocene deposits at the west of the Sinop Peninsula and they studied in detail on the activity of existing faults at the region. These studies were related with Nuclear Power Plant site selection. Arel (1985), examined the landslides around the Sinop region and he prepared a landslide susceptibility map of Sinop and its vicinity. Finally, Yüksel Proje Construction Company (1998) performed geotechnical and borehole inclinometer studies at the 33 Evler Landslide, for movement monitoring.

CHAPTER 2

PHYSIOGRAPHY

2.1. Topography and Drainage

The Sinop region is mostly characterized by a plateau which is rising from west to south-east. Elevations range between 0 to 143 meter at the city center and its close vicinity (Figure 2.1). The Boztepe peninsula (Figure 1.1) is connected to the main land through a tombolo.

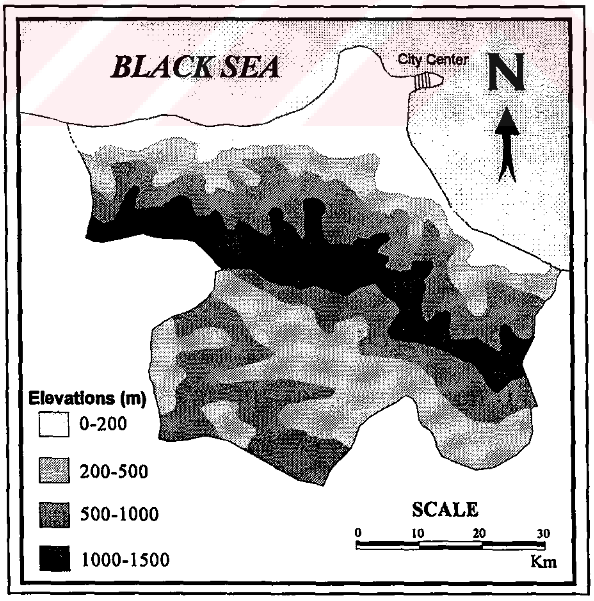


Figure 2.1 Distribution of topographic elevations in the Sinop region.

The drainage pattern is sub parallel and formed by intermittent streams. At some places of the Sinop region, gullies are formed due to weak soil and this phenomenon contributes to the instability of natural slopes.

2.2. Climate

The Black Sea climate is sovereign at the north of the Sinop. However at the south of the Sinop, the effect of Black Sea climate diminishes. All year is rainy, except for a few days in summer. Figure 2.2 and 2.3 show total rainfall quantities and average temperatures respectively for each month of the year (averages of 1978 to 1998) which were obtained from the General Directorate of State Meteorological Works.

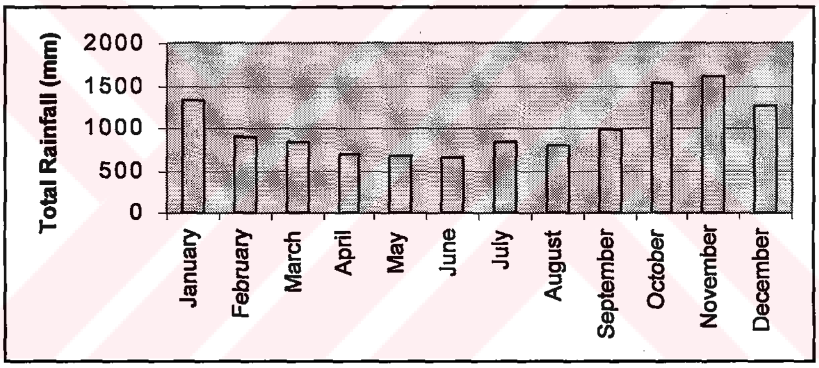


Figure 2.2 Total rainfall quantities of Sinop city center

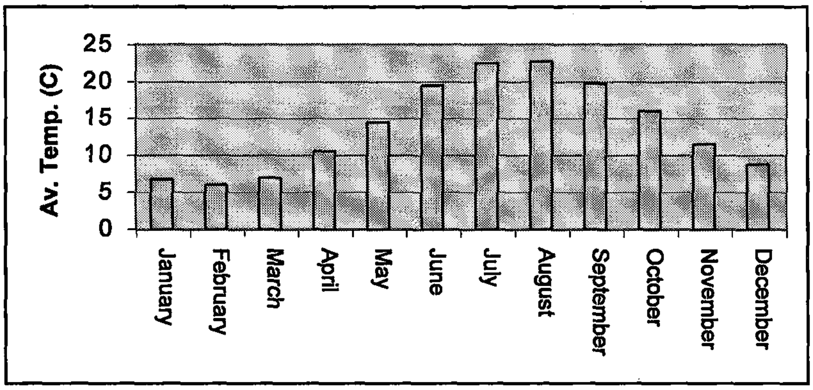


Figure 2.3 Average temperatures of Sinop city center

CHAPTER 3

SITE GEOLOGY

3.1. Stratigraphy

3.1.1. The Hamsaros Formation

The Hamsaros Formation crops out all along the northern coast of the west of Sinop Peninsula, forming in places 5 – 25 meters high cliffs. The formation consists of lava flows, volcanic breccias, agglomerates, volcanic conglomerates, and dykes (Figure 3.1). Where the weathering is high, it is difficult to differentiate pillow lavas and volcanic conglomerates. However, they are typical around the town of Sinop; the flow breccias are only exposed around Boztepe (Figure 3.2), (Barka et al, 1985).

Volcanic conglomerates constitute the higher portion of the Hamsaros Formation, the sizes of individual blocks range between boulder to very large blocks (a few meters in diameter). At some locations grading has been observed, nevertheless irregular stratification is most common. Volcanic conglomerates locally and lateraly pass into siltstone and clay, and upward which indicates an existence of a marine environment which differs

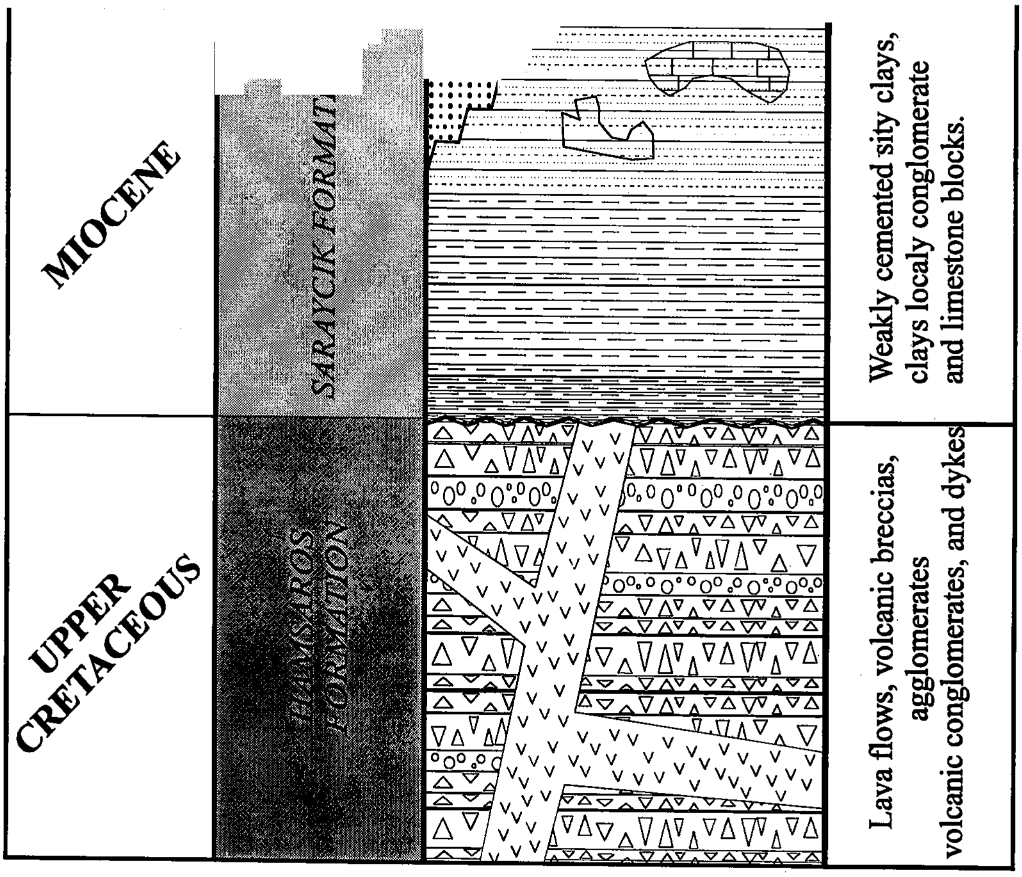


Figure 3.1. General stratigraphic column of the site (not to scale)

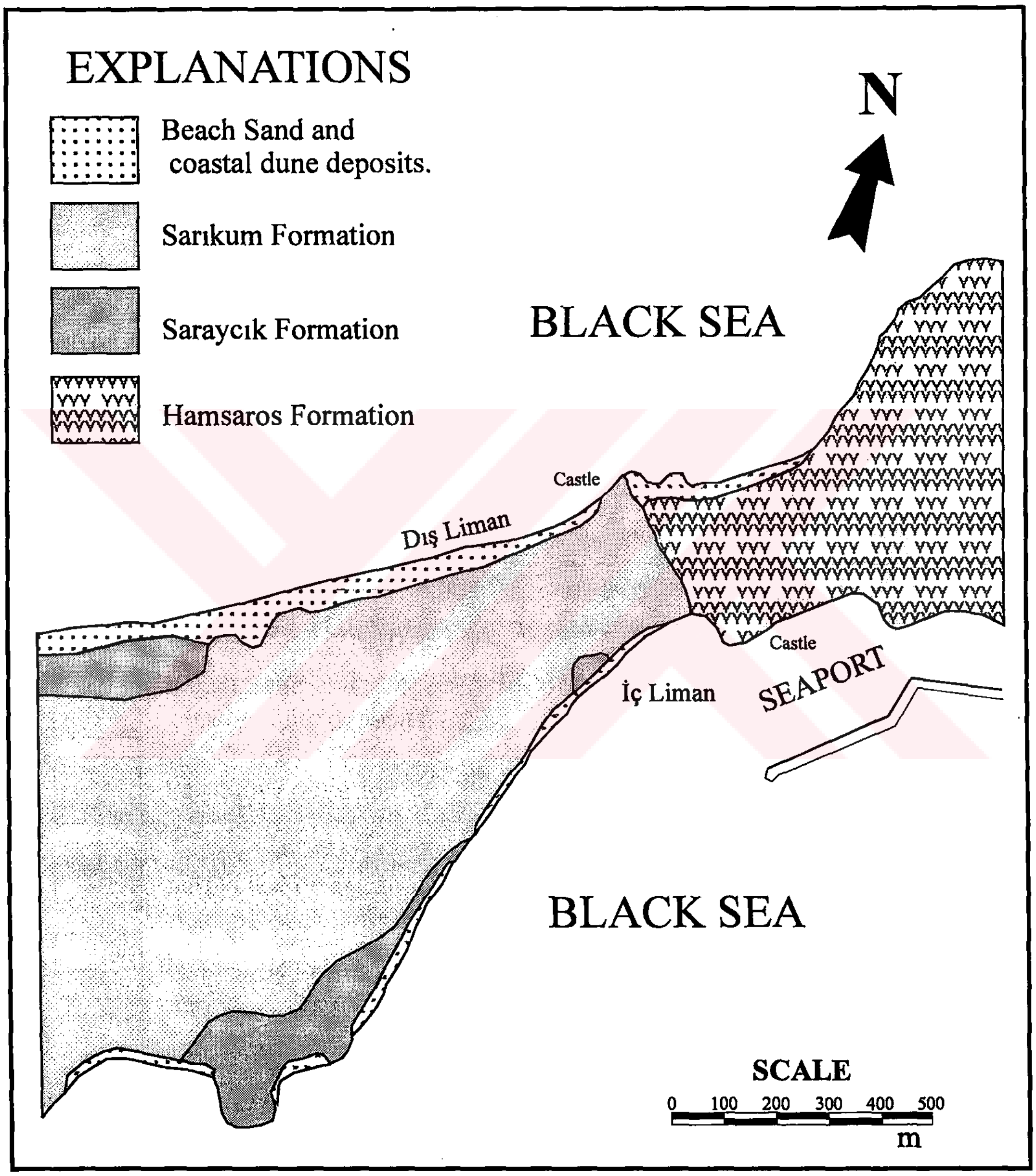


Figure 3.2 Geological map of the Sinop region (modified from Arel, 1985)

volcanic conglomerates from agglomerates (Barka et al, 1985). Explosive volcanism has been evident from black to dark green lapillies within the conglomerates.

Individual blocks of agglomerates are of various sizes, ranging between small pebbles to boulders and blocks, semi - angular in shape which are of basaltic origin including their finer matrix. Agglomerates and breccias are mostly formed by lava blocks, volcanic bombs, tuffs and lapillies. Agglomerates are mostly compact, massive and dark green to black in colour. The dykes occurred after the deposition of agglomerates, breccias and conglomerates.

The age of the Hamsaros Formation is determined by radiometric dating techniques as Late Cretaceous (Barka et al, 1985).

3.1.2. The Saraycık Formation

Miocene formations are widespread at the Sinop Peninsula below the Sarıkum formation. They are generally of marine origin, but some members of the Saraycık formation was deposited in a lagoonal environment.

According to Barka et al, Lower Miocene formations consist of gray, weakly cemented silty claystones and claystones. Locally pelecypoda and gastropoda rich layers are also present, sandy – pebbly lenses could also be encountered. According to Barka et al (1985), these deposits indicate shallow sea environments, because of the fossil groups and sedimentary structures they involved.

Northward, toward Dış Liman, these shallow sea deposits grade into lagoon and lacustrine deposits. These deposits are weakly cemented, grayish to beige coloured siltstones, claysstones, and marls. Locally limestone and conglomerate blocks are also present.

Saraycık formation unconformably overlies The Hamsaros formation and is overlain by the Sarıkum formation (Figure 3.1).

3.1.3. The Sarıkum Formation

The Sarıkum formation covers a wide area in the Sinop Peninsula as well as in the site vicinity which unconformably overlies all other rocks including the Miocene formations. This formation consists of yellowish cross bedded sand at the bottom, uncemented fine sand, silt and conglomerate at the middle, brownish-yellowish quartz sands at the top. There are also clay lenses present at some parts of the formation. Clay lenses indicate local marsh and swamp areas during deposition.

According to Barka et. Al. (1985), the thickness of the formation varies and increases from the north to south. Nevertheless the thickness is entirely controlled by the paleo topography of the Hamsaros formation.

Sarıkum formation can be accepted that it is a Plio – Quaternary age (Barka et. al.,1985).

The Sarıkum Formation is considered that it has been deposited in a flood plain and eolian environment (Barka et. al.,1985).

3.1.4. Recent Deposits

The recent deposits include beach sands and coastal dune deposits along the shore line. These deposits overlie older formations unconformably (Figures 3.1 and 3.2).

CHAPTER 4

ENGINEERING GEOLOGICAL EVALUATION

4.1. Drilling

In order to investigate soil and rock conditions at the landslide area, seven boreholes were drilled by Yüksel Proje Uluslararası A.Ş. (1998). Through the boreholes numbered Hik 1, Hik 2, Hik 3, Hik 4, and Hik 5, inclinometer measurements were taken in order to determine the position of the slip surface and the rate of sliding. The boreholes Hsk1 and Hsk2 were drilled for ground investigation purposes. Table 4.1 shows the type of drilling, diameter of boring, depth to groundwater level, borehole elevations, coordinates and in-situ tests performed. At all boreholes SPT measurements were taken at almost every 1.5 meters and UD (undisturbed) samples were taken at Hik 1, through Hik 5 boreholes. At each boring three UD samples were taken. Figure 4.1 shows the landslide area and the borehole locations. Figure 4.2 shows simplified boring logs.

At the uppermost part of boreholes, a man made fill, which is generally formed from base course (road fill) and construction wastes was seen. Its thickness ranges between 1.1 meter to 3.1 meters (Figure 4.2).

Under the man made fill, there is Sarıkum formation. The landslide scarp is formed within the Sarıkum formation. Below Sarıkum formation Saraycık formation is penetrated (Figure 4.2).

Table 4.1 Summary of borehole data.

Name	Type of Boring	Elevation (m)	Coordinates (x;y)	Borehole Depth (m)	Diameter	Depth to GW (m)	In Situ Tests	Casing Depth (m)	Time of completion (day)
HSK 1	Mobile Drill Auger	15.84	---------	15.03	200 ϕ	1.20	SPT	15 - Auger	1
HSK 2	MDA	18.92	---------	19.95	200 ϕ	3.35	SPT	19.5 Auger	2
HIK 1	D500 Rotary	4.65	4654137.64; 4279822.59	15.45	HW	3.60	SPT-INC	15	1
HIK 2	MDA	8.87	4654158.71; 427965.32	15.45	HW	7.90	SPT-INC	15	1
HIK 3	D500 R	17.59	4654201.1; 427930.58	22.95	HW	11.20	SPT-INC	22.50	4
HIK 4	D500 R	21.41	4654235.97; 427902	15.45	HW	4.0	SPT-INC	15	1
HIK 5	MDA	16.20	4654189.7; 427909.66	21.29	HW	10.00	SPT-INC	21	2

GW: Groundwater

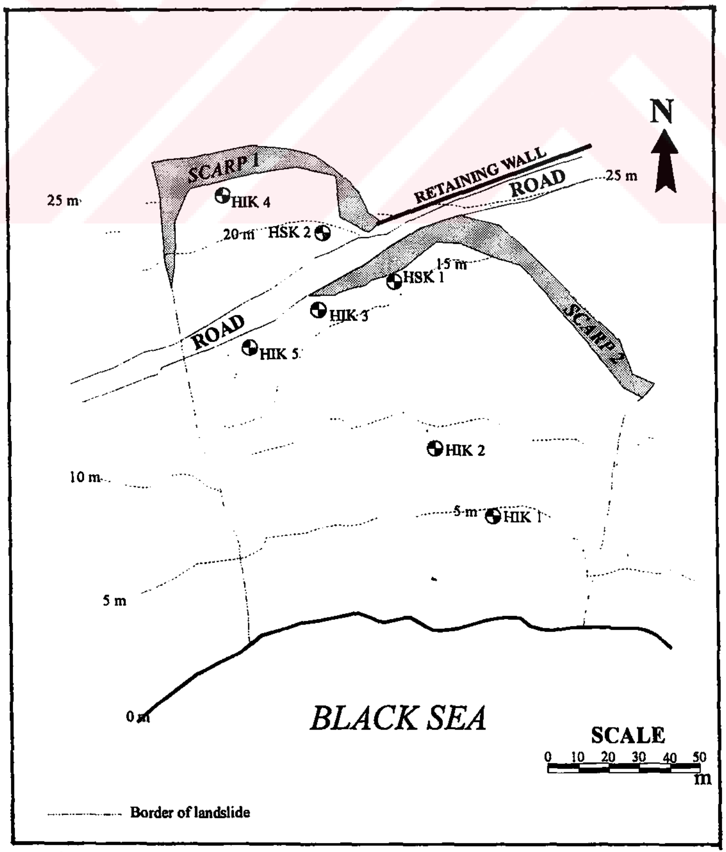


Figure 4.1. Borehole locations.

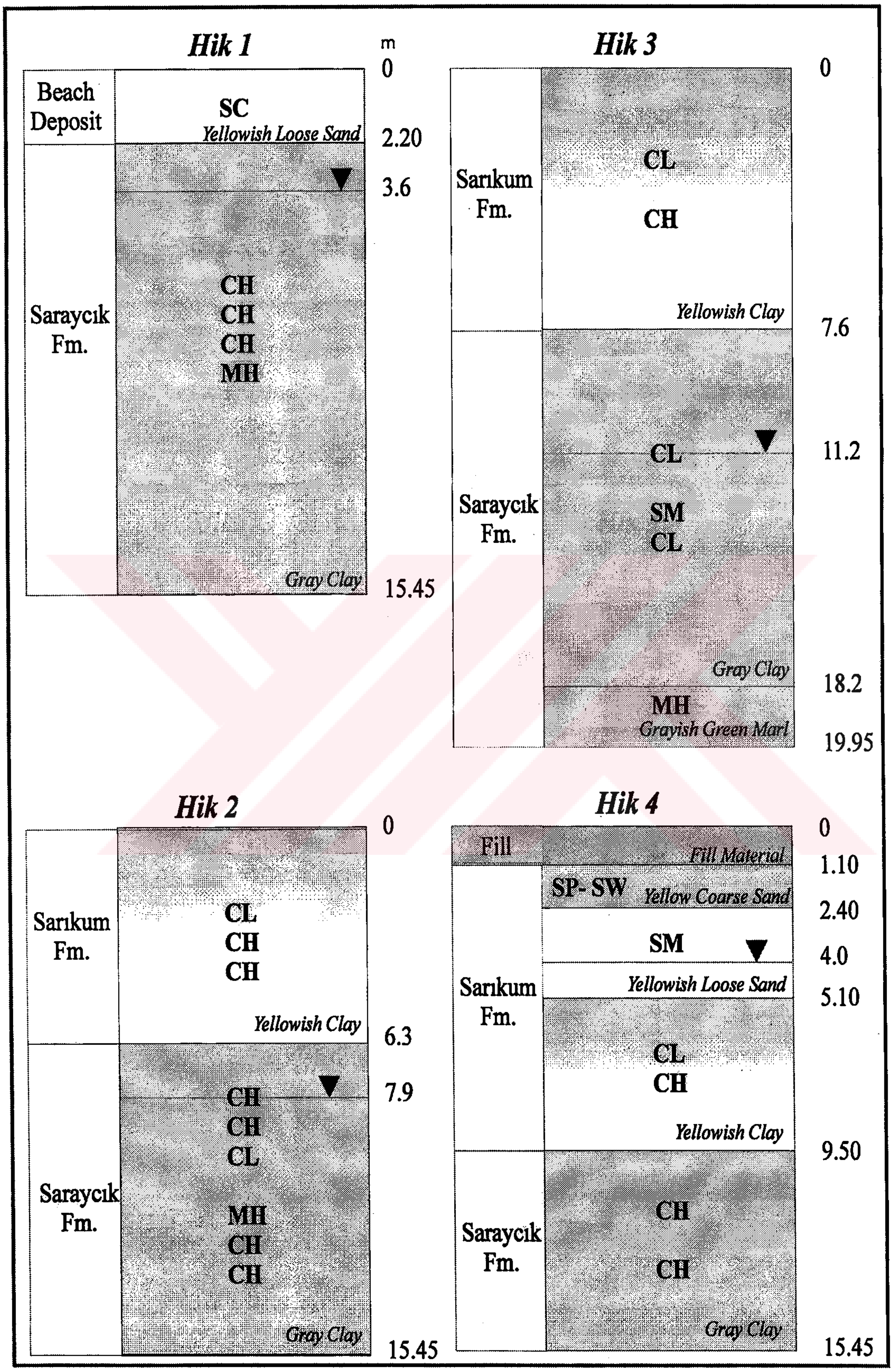


Figure 4.2 Simplified boring logs

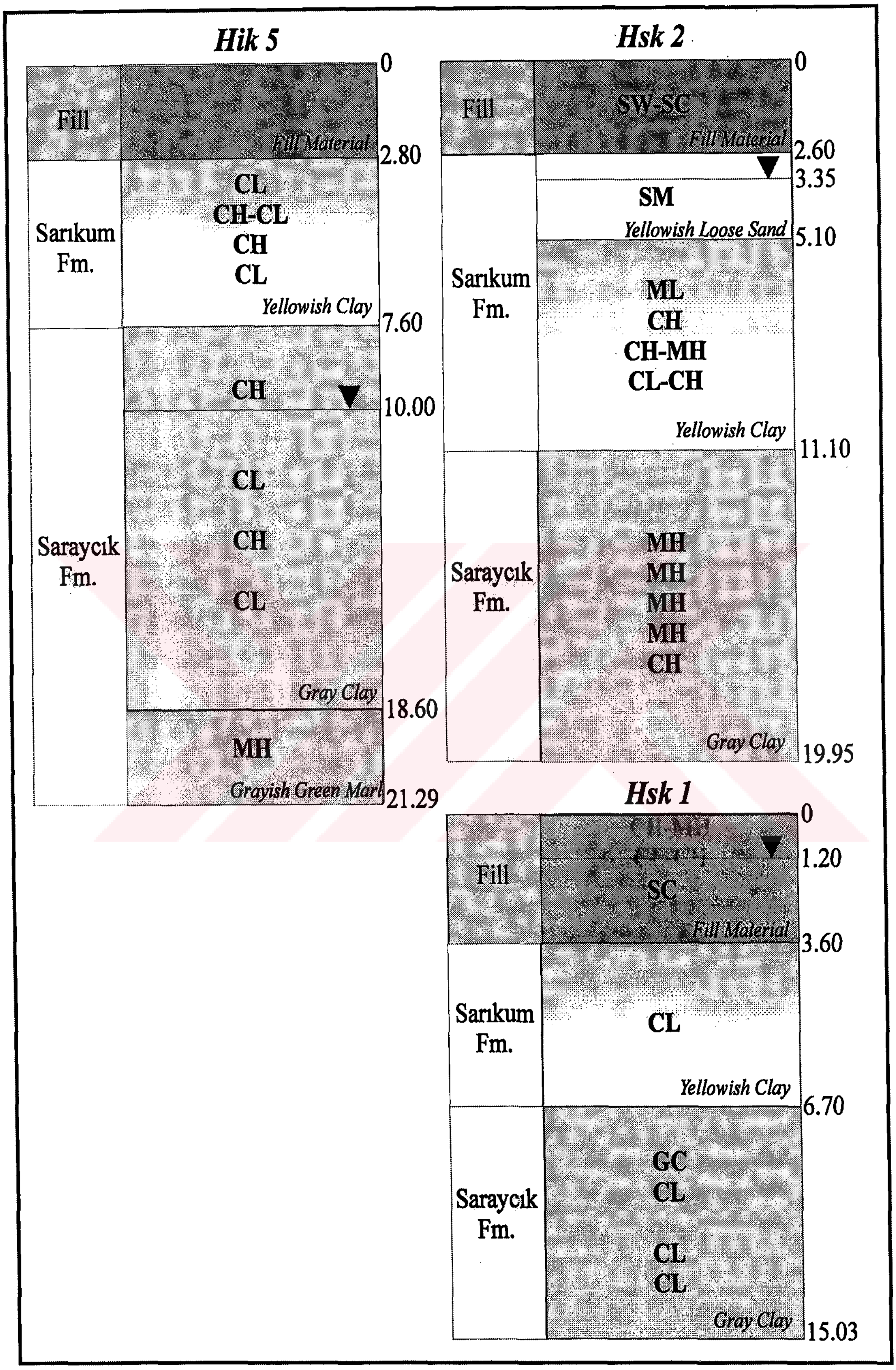


Figure 4.2 (contd.)

4.2 In-Situ Testing

4.2.1 Standard Penetration Test

Standard penetration test (SPT) was performed in every borehole, at every 1.5 meter intervals in order to determine engineering classification of soils present at the site. SPT values range between 4 and 26 at man - made fill, 9 – 15 at Sarıkum Formation, and 29 – 78 at Saraycık Formation. Figure 4.3 shows uncorrected SPT –N plots for the drilled boreholes.

Based on the SPT -N values, the Saraycık formation is stiff to hard clay and Sarıkum formation is medium sand at the top, and medium consinstency clay towards the bottom.

4.2.2 Inclinometer Measurements

Inclinometers are used to monitor lateral earth movements in landslide areas and embankments (Figure 4.4). They are also used to monitor the deflection of retaining walls and piles under load (Slope Indicator Company, 1994). Horizontal inclinometers are used to monitor settlement in foundations and embankments.

Inclinometers are used to measure movements which are perpendicular to borehole axis. By the inclinometer measurements, movement location and movement rate can easily be measured (Figure 4.5). By one inclinometer device, measurements can be taken from many boreholes, at desired time intervals.

At Hik 1, through Hik 5, inclinometer measurements were taken and cumulative displacement and incrimental displacement graphs were drawn. Figure 4.6 shows the inclinometer measurements taken from Hik 1 as an example. From these graphs (Appendix A), the failure surface in each borehole was determined. Table 4.2 shows failure surface depths determined from the inclinometer measurement graphs.

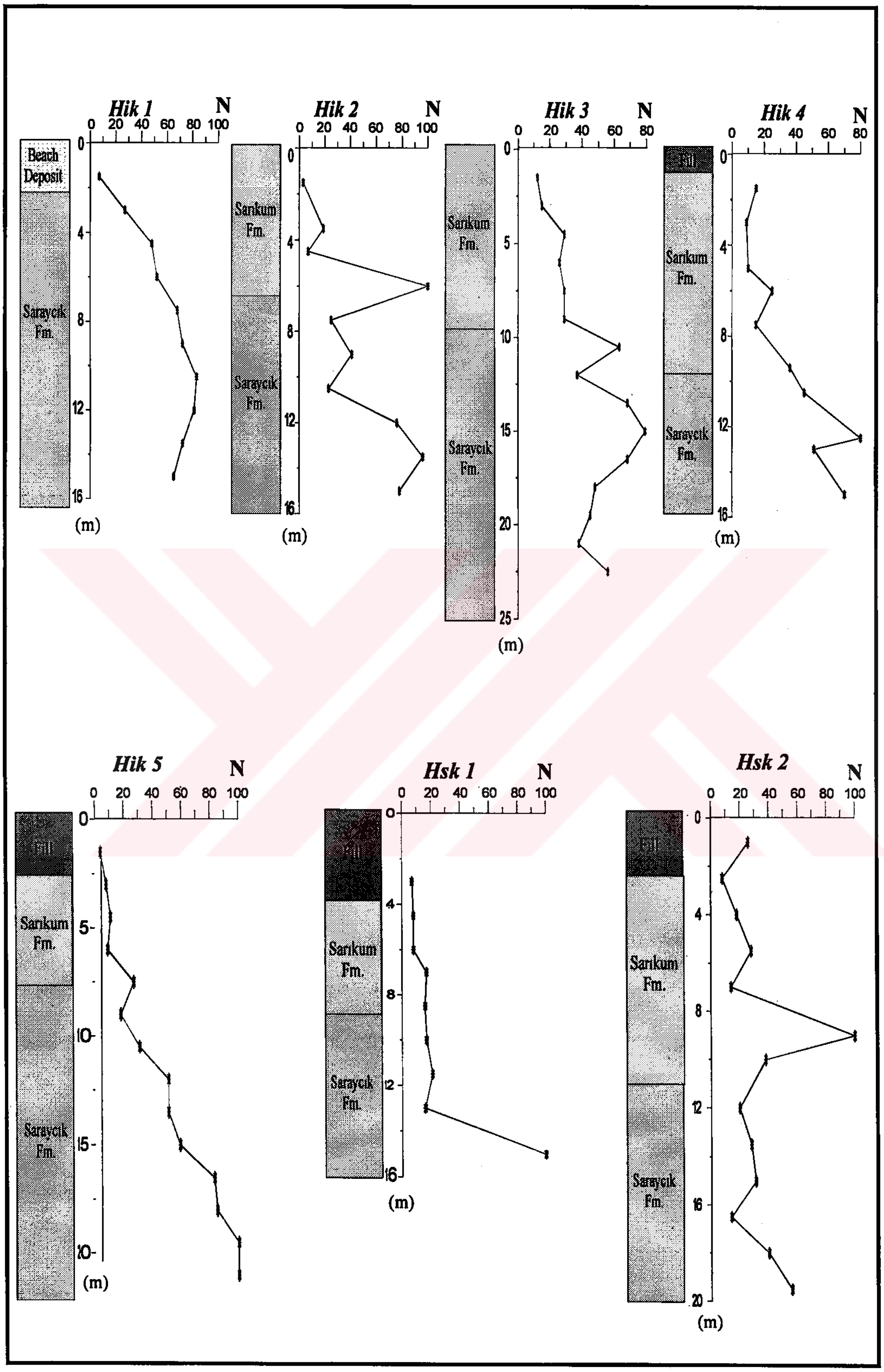


Figure 4.3 Graphical presentation of SPT results.

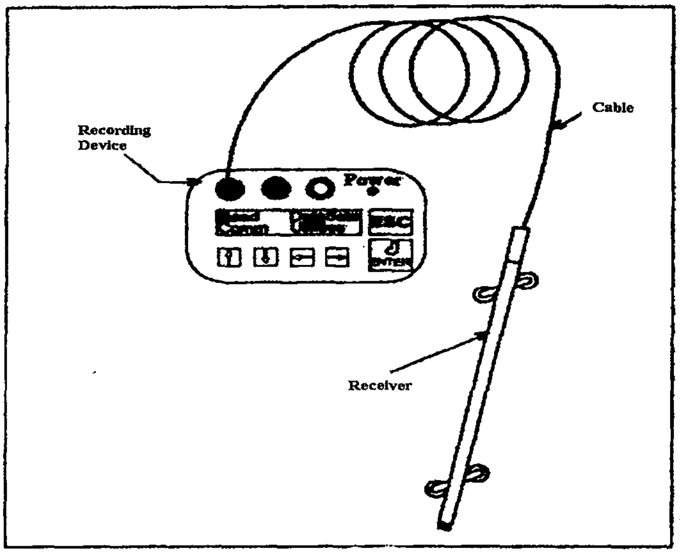


Figure 4.4 Inclinometer (Slope Indicator Company,1994).

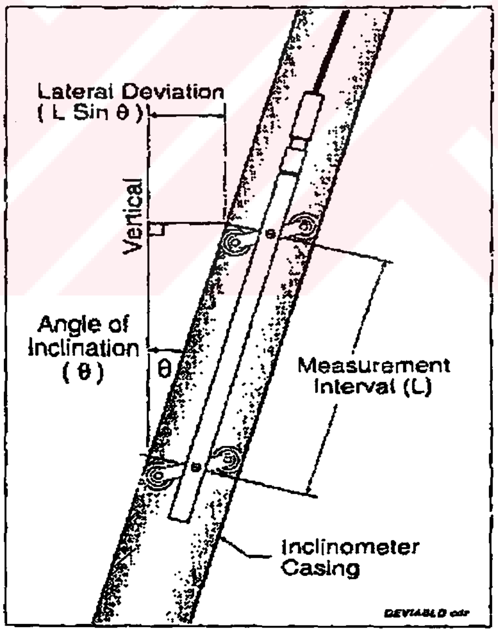


Figure 4.5 Inclinometer Measurement (Slope Indicator Company,1994).

Table 4.2 Failure surface depths

Borehole Name	Depth of Failure Surface (m)
Hik 1	7.6
Hik 2	11.8
Hik 3	21
Hik 4	6.2
Hik 5	21

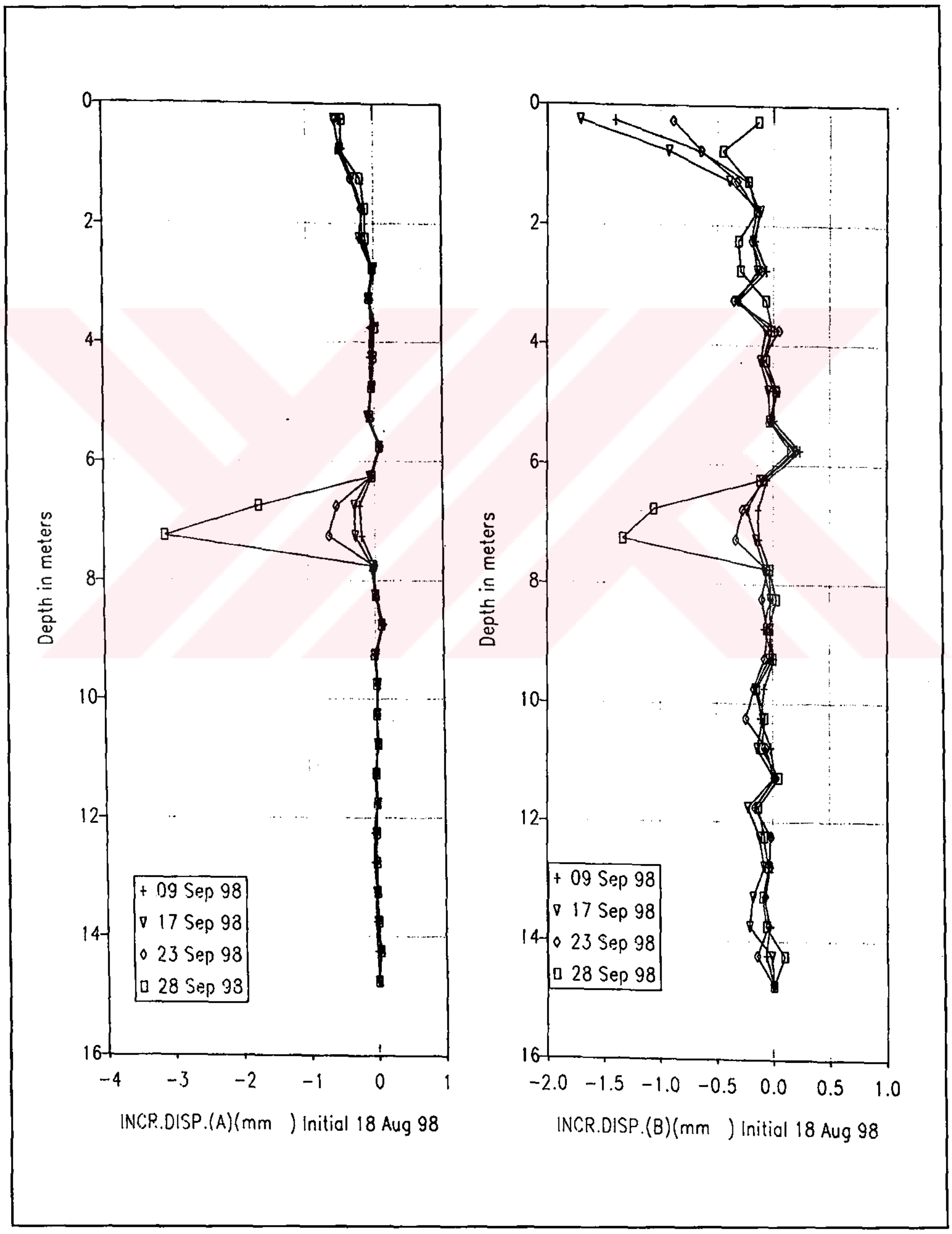


Figure 4.6 Inclinometer measurement graphs of Hik 1.

4.3 Laboratory Tests

Disturbed and undisturbed samples were taken from the boreholes by Yüksel Proje Uluslararası A.Ş. Index tests (water content determination, liquid limit, plastic limit, sieve analysis, specific gravity determination tests) were performed and soil types were determined using the test data. Unconfined compression tests, triaxial tests and consolidation tests were performed on undisturbed samples Yüksel Proje Uluslararası A.Ş. The purpose of the strength tests were to determine shear strength parameters of the formations.

4.4 Geotechnical Evaluation

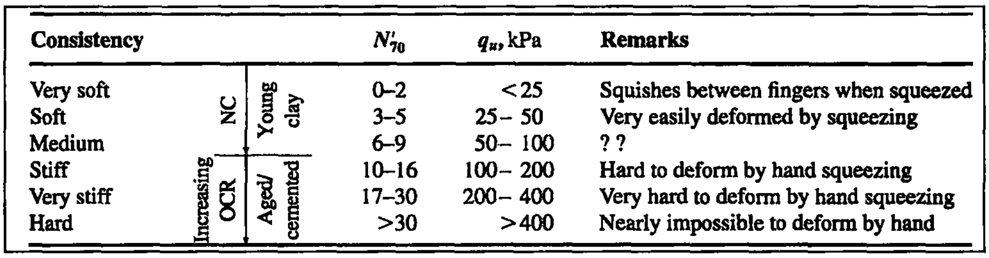
4.4.1 Engineering Geological Description of the Saraycık Formation

Saraycık formation is formed from alternation of claystone, siltstone, and marl. Colour of Saraycık formation is mainly gray and grayish green. Saraycık formation is completely weathered (Grade V) according to BSI, 1981 and turned into clay at the upper parts, the effect of weathering however decreases with depth.

The strength of Saraycık formation can be assessed as stiff clay according to ISRM (1978).

SPT performed within Saraycık formation yield values between 29 and 78 for N_{50} (for N_{70}: 20-55) depending on testing depth, degree of weathering and type of member, in other words whether it is clay, silt or marl. Based on SPT -N values, Saraycık formation may be classified as stiff to hard soil according to the classification proposed by Bowles (1996), (Table 4.3).

Table 4.3 Consistency, qu relationship between SPT values (Bowles, 1996)



Consistency			N'_{70}	q_u, kPa	Remarks
Very soft	NC	Young clay	0–2	<25	Squishes between fingers when squeezed
Soft			3–5	25– 50	Very easily deformed by squeezing
Medium			6–9	50– 100	? ?
Stiff	Increasing OCR	Aged/ cemented	10–16	100– 200	Hard to deform by hand squeezing
Very stiff			17–30	200– 400	Very hard to deform by hand squeezing
Hard			>30	>400	Nearly impossible to deform by hand

Laboratory tests were performed according to ASTM standards. 53 index tests were performed in order to determine water content (Wn), liquid limit (LL), plastic limit (PL), plasticity index (PI), and size distribution of the samples. 7 tests were performed in order to determine void ratio (e), specific gravity (Gs), and natural unit weight (γ_n). 7 tests were performed in order to determine unconfined compressive strength. 7 unconsolidated-undrained (UU) tests were performed in order to determine and shear strength parameters of the samples.

Table 4.4 summarises the laboratory test results of the Saraycık Formation.

Table 4.4 Summary of laboratory test results performed on the Saraycık formation samples.

				Sieve Analysis				
W_n (%)	LL (%)	PL (%)	PI (%)	+ 4	- 200	q_u (kPa)	c (kPa) Triaxial	ϕ (deg) UU test
26 - 33	33 –70 (60)	22 –36 (30)	12 –40 (30)	0	78 –99 (95)	55.9- 440.3 (240)	117,7- 269,7 (255)	1-5 (2)
The values in the parentheses indicate the mean value.								

Average value of the natural and saturated unit weights are γ_n = 18.9 kN/m^3 and γ_s = 20 kN/m^3 respectively for Saraycık formation.

Unconfined Compressive Strength (qu) determined by SPT –N values (Table 4.5), are in good agreement with the laboratory test results.

According to The Unified Soil Classification System, the Saraycık formation may be classified as CL, CH and MH (CL: low plasticity silts inorganic clay, CH: high plasticity inorganic clay, MH: inorganic silts and silty soils).

According to XRD analysis of the samples taken from the Saraycık Formation dominant minerals are clay minerals, quartz, plagioclase, and calcite. Clay minerals are mainly smectite, illite, kaolinite, and some chlorite (Arel, 1985). The most abundant clay mineral is smectite.

4.4.2 Engineering Geological Description of the Sarıkum Formation

The Sarıkum formation is formed from, yellowish loose pebbly, silty sands, and clayes silts. SPT -N values taken from the Sarıkum Formation range between 9 and 15 depending on testing depth.

The Sarıkum formation is medium dense sand at the top, and medium consistency clay towards the bottom, where formation consist of clayey silts, clays.

According to the Unified Soil Classification System, the Sarıkum formation may be classified as SP, SW, SM (SP: poorly graded sands, little or no fines, SM: silty sands, sand and silt mixtures, SW: well graded sands, gravelly sands, little or no fines) at its upper parts, and MH, CL, CH towards the bottom. Table 4.5 shows the summary of the laboratory test results of the samples taken from the Sarıkum formation.

Table 4.5 Summary of laboratory test results performed on Sarıkum formation samples.

Wn (%)	LL (%)	PL (%)	PI (%)	Sieve Analysis		q_u (kPa)	c (kPa) Triaxial	ϕ (deg) UU test
				+ 4	- 200			
11 - 37	32 –65 (36)	15 –28 (30)	12 –40 (20)	8	3 – 40 for SP, SW, 86 – 95 at CL	58.9-142.2	No Data	No Data

The values in the parentheses indicate the mean value.

Average value of the natural and saturated unit weights are γ_n = 19 kN/m^3 and $\gamma_s = 20$ kN/m^3 respectively for Sarıkum formation.

According to XRD analyses performed by Arel (1985), dominant minerals are clay minerals, quartz, plagioclase, and calcite; but amounts of quartz, and plagioclase are more than that in the Saraycık formation. The amounts of clay mineral and calcite are less than those of the Saraycık formation. Clay minerals include smectite, illite, and kaolinite (Arel, 1985), smectite is the most common clay mineral.

CHAPTER 5

SEISMICITY OF THE SİNOP REGION

The local geological studies and the potential for surface faulting in the near vicinity of the Sinop have been covered in Doyuran, (1983), Doyuran and Erdik, (1983), Doyuran and Tuncer (1983), and Barka et al. (1985).

The Sinop region is at the fourth degree earthquake zone according to Turkish Earthquake Zoning Map. The seismicity of the Sinop region is mainly controlled by the North Anatolian Fault Zone (N.A.F.Z), located approximately 100 kilometer to the south of Sinop (Figure 5.1). It has been known that, large earthquakes occurring along the North Anatolian Fault Zone, also affect the Sinop region (Arel, 1985).

Canıtez and Büyükaşıkoğlu (1984), reported the followings:

(i) The Alaaddin Mosque built in Sinop by Selçuks was damaged by earthquakes and a timber niche was reconstructed by Sultan Abdülmecit.

(ii) The December 26, 1939 Erzincan Earthquake was felt strongly in Sinop and vicinity. Some poorly built chimneys and garden walls collapsed.

(iii) During the November 26, 1943 Ladik Earthquake, Sinop experienced slight damage. The 4 meters high garden walls and the peripheral walls of the niche of the Alaaddin Mosque experienced cracks. These walls were of stone masonry and the cracks were about 1 cm wide. Some of the reinforced brick masonry public buildings received cracks. The sea was withdrawn by about 10 meters. Few roofs slid on their base and several chimneys and mud plaster garden walls collapsed. A landslide of dimensions about 1 km square was triggered at Yaykıl Village.

None of the earthquakes affecting the Sinop were felt with intensities greater than MMI VI (Erdik et al, 1990).

It has been concluded that, there is no evidence indicating existence of active faults and there does not exist any potential for surface faulting at the Sinop region (Erdik et. al, 1990).

According to Gülkan et al (1993), peak horizontal ground accelerations for city center (approximately latitude 42°, longitude 35°) are 0.093 g for 100 year and, 0.119 g for 225 year return periods.

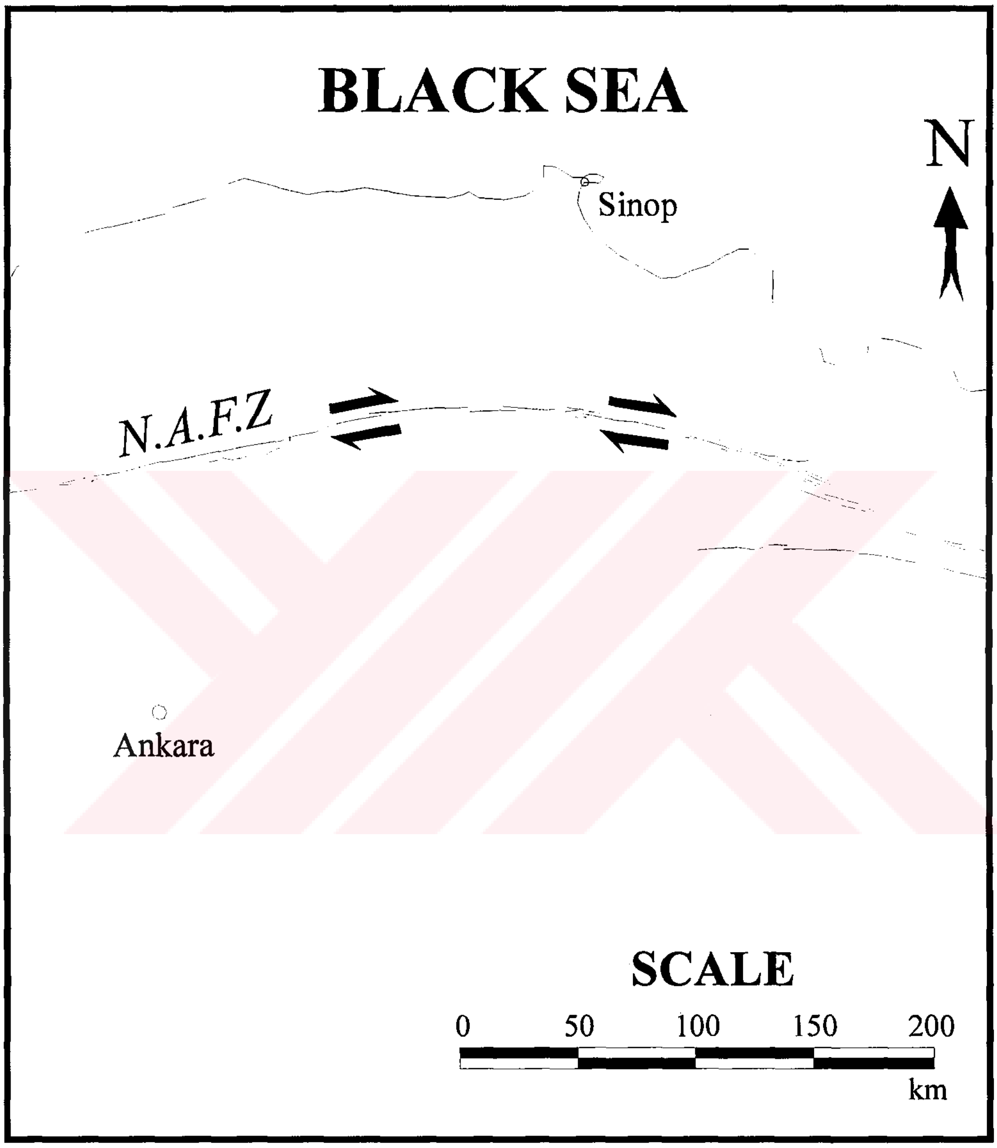


Figure 5.1 Location of the N.A.F.Z

CHAPTER 6

SLOPE STABILITY

Type of the 33 Evler landslide is rotational slide according to Varnes (1978) classification system. Because inclinometer measurements revealed a circular failure surface.

6.1 Methods of Soil Slope Stability Analysis.

The study of soil stability problems has traditionally been based on the concept of limit equilibrium. A mass of soil is identified and regarded as a free body with known external boundaries and an internal boundary along a real or assumed discontinuity generally known as the slip surface (Chowdhury, 1987). This surface may be curved or may consist of one or more planes. The forces acting on the free body may be distinguished into two categories, disturbing forces and resisting forces. Limit analysis is concerned with the balance between resisting and disturbing forces as if a condition of incipient failure had been reached.

For many slope failures, the observation that the surface along which sliding took place was not planar but curved, led to the idea of using curved failure surfaces for the analysis of slope stability. Although the actual surface of rupture is in most cases bowl shaped, the representation of the failure surface as a single curve greatly simplifies the analysis (Mostyn and Small, 1987). Initially, methods of analysis based on circular surfaces were developed; however, methods for non – circular surfaces were soon established.

All limit equilibrium methods for slope stability analysis divide a slide mass into (n) smaller slices (Figure 6.1). Each slice is affected by a general system of forces (Figure 6.2). The thrust line indicated in the figure connects the points of application of the interslice forces (Zi).

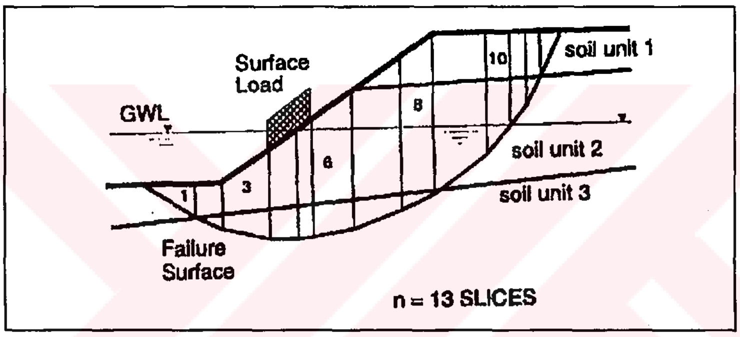
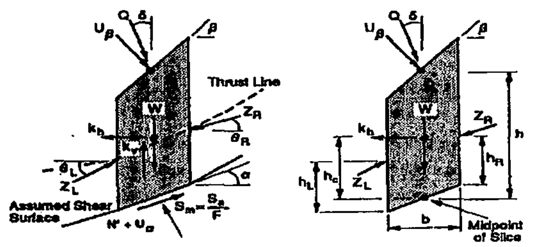


Figure 6.1 Dividing the potential sliding mass into slices.



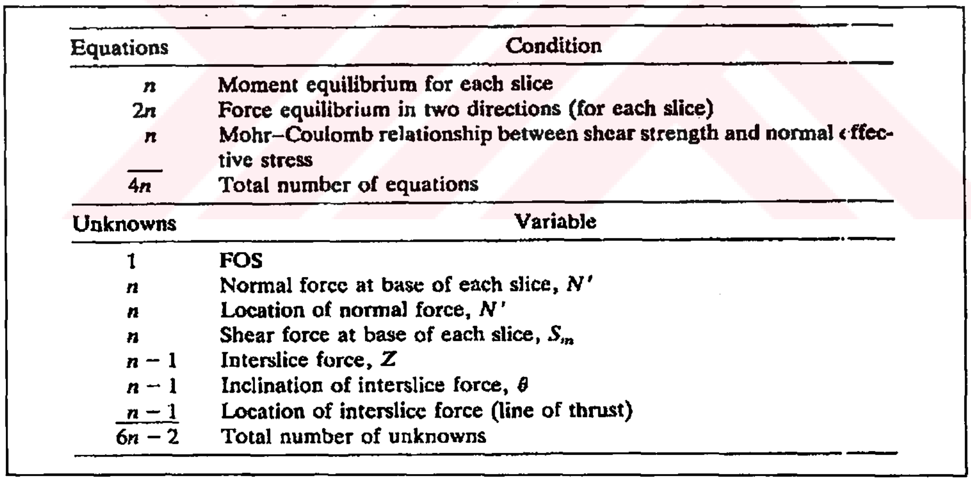
F = factor of safety
S_a = available strength
= C + N' tanϕ
S_m = mobilized strength
U_α = pore water force
U_β = surface water force
W = weight of slice
N' = effective normal force
Q = external surcharge
k_v = vertical seismic coefficient
k_h = horizontal seismic coefficient

Z_L = left interslice force
Z_R = right interslice force
θ_L = left interslice force angle
θ_R = right interslice force angle
h_L = height to force Z_L
h_R = height to force Z_R
α = inclination of slice base
β = inclination of slice top
b = width of slice
h = average height of slice
h_c = height to centroid of slice

Figure 6.2 Forces acting on a typical slice

For the system shown in Figure 6.2, there are (6n-2) unknowns (Table 6.1). Also, since only four equations can be written for the limit equilibrium for the system, the solution statically indeterminate. However, a solution is possible providing the number of unknowns can be reduced by making some simplifying assumptions. One of the common assumptions is that the normal force acts at the midpoint, thus reducing the number of unknowns to 5n-2. This then requires an additional (n-2) assumption to make problem determinate (Abramson, et al, 1996). It is these assumptions that generally categorise the available methods of analysis (Sharma and Lowell, 1983).

Table 6.1 Equations and unknowns associated with the method of slices.

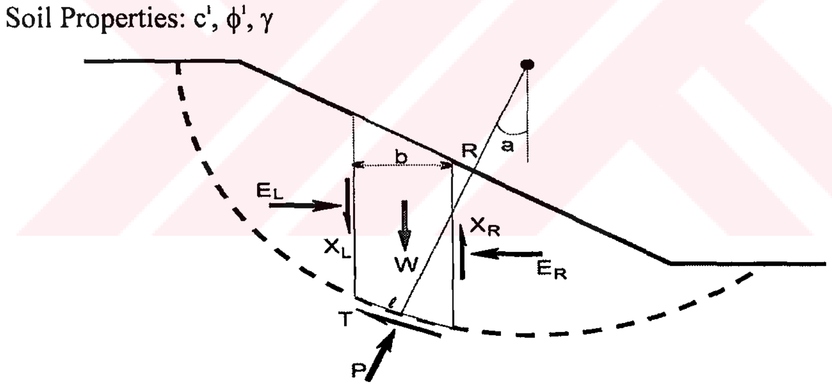


Equations	Condition
n	Moment equilibrium for each slice
$2n$	Force equilibrium in two directions (for each slice)
n	Mohr–Coulomb relationship between shear strength and normal effective stress
$4n$	Total number of equations
Unknowns	**Variable**
1	FOS
n	Normal force at base of each slice, N'
n	Location of normal force, N'
n	Shear force at base of each slice, S_m
$n - 1$	Interslice force, Z
$n - 1$	Inclination of interslice force, θ
$n - 1$	Location of interslice force (line of thrust)
$6n - 2$	Total number of unknowns

6.1.1 Bishop's Simplified Method

Bishop (1955) originally presented his method for analysis of circular slip surfaces, but it can be applied to non circular slip surfaces by adopting a fictional center of rotation. In this method it is assumed that the interslice shear forces may be neglected. The total normal force is assumed to act at the center of the base of each slice, and is determined by resolving the forces on each slice vertically (Figure 6.3). This method of analysis involves a total of 2n-1 assumptions. Thus the problem is overspecifed, and in general overall horizontal equilibrium is not satisfied for a slice.

Whitman and Bailey (1967) have looked closely at the performance of the Bishop simplified method and have concluded that the error involved in using this type of analysis against the more rigorous formulation is usually only 2% or less. These authors also point out that the method may be inaccurate in the case where the angle α of the base of the slice is negative (which may happen at the toe of the circle); they suggest that the value of the term m_α ($m_\alpha = \cos\alpha\ (1 + (\tan\alpha * \tan\ \phi^1)/F)$) be checked and if $m_\alpha \leq 0.2$, the results should be regarded as unreliable.



For slice shown: at base – normal stress σ, shear stress τ, pore pressure u.

Mohr – Coulomb failure criterion: $S = c^1 + (\sigma\text{-}u).\tan\phi^1$

Mobilised shear strength $\tau = s/F$ where the F is the factor of safety

Now $P = \sigma.l$, $T = \tau l$ so $T = (1/F).(\ c^1 l + (P\text{-}ul).\tan\phi^1\ )$

Resolve vertically: $P.\cos\alpha + T.\sin\alpha = W\text{-}(X_R - X_L)$

Assuming $X_R = X_L = 0$ (interslice forces are horisontal)

$P = [W\text{-}(1/F)(\ c^1 l.\sin\alpha - ul.\ \tan\phi^1.\sin\alpha)]/m_o$

Where $m_o = \cos\alpha(1 + \tan\alpha(\tan\phi^1/F))$

Overall MOMENT equilibrium (about O): $\Sigma\ W.R.\sin\alpha = \Sigma\ T.R$

Rearranging and substituting for T gives

$F_m = \Sigma\ (c^1.l + (p\text{-}ul).\tan\phi^l) / \Sigma\ W.\sin\alpha$

As this equation contains F on both sides it has to be solved iteratively.

Figure 6.3 Bishop simplified method

Although the complete method proposed by Bishop satisfies all of the conditions of equilibrium with respect to forces and moments, and takes into considerations all of the components of the interslice forces, the routine method does not. Why the routine method could therefore be so accurate was eventually explained by a further extension to the theory by Spencer (1967). An important finding of this work was the factor of safety given by the moment equation varied only a little with increasing θ values (θ is the angle with respect to the horizontal of the resultant of all the interslice force components on a single slice Z), θ=0 case is identical to the Bishop simplified method. In contrast, the force equilibrium derived equation was very sensitive to θ. This then is the key to the relative accuracy of Bishop's Routine method; it is soundly based on the equilibrium of moments (Broomhead, 1992). These are much larger in magnitude than the forces, and so satisfying moment equilibrium brings about near satisfaction of force equilibrium; the reverse, however is not true (Broomhead, 1992). As a result a factor of safety determined using a force equilibrium procedure is much more sensitive to the assumption about interslice forces than the factor of safety determined by satisfying moment equilibrium (Nash, 1992).

6.1.2 Bishop's Rigorous Method

Bishop (1955) assumes (n-1) parallel inclined but not necessarly horisontal interslice shear forces to calculate a factor of safety. Since this assumption leaves (4n-1) unknowns, moment equilibrium cannot be directly satisfied for all slices. However, Bishop introduced an additional unknown by suggesting there exist a unique distribution of the interslice resultant force, out of a possible infinite number, that will rigorously satisfy the equilibrium equations (Abramson, et al, 1996).

6.2 Methods of Seismic Analysis

Earthquake ground motions are capable of inducing large destabilizing inertial forces of a cyclic nature, in slopes and embankments. Also, the shear strength of the soil may be reduced due to transient loads or due to the generation of excess pore water pressures. The combined effect of the seismic loads and the changes in shear strength will result in an overall decrease in the stability of the affected slope (Abramson, et al, 1996).

Typically, cyclic loads will generate excess pore water pressures in loose, saturated cohesionless materials (gravels, sands, and nonplastic silts), which may liquefy with a considerable loss of pre-earthquake strength. However, cohesive soils and dry cohesionless materials are not generally affected by cyclic loads to the same extent. If the cohesive soil is not sensitive, in most cases it appears that at least 80 percent of the static shear strength will be retained during and after the cyclic loading (Makdisi and Seed, 1978). In general, four methods of analysis have been proposed for the evaluation of the stability of slopes during earthquakes. In increasing order of complexity and expense, these are:

(1) *Pseudostatic Method:* The earthquake inertial forces are simulated by the inclusion of a static horizontal and vertical force in a limit equilibrium analysis.

(2) *Newmark's Displacement Method:* This method is based on the concept that the actual slope accelerations may exceed the static yield acceleration at the expense of generating permanent displacements.

(3) *Post-Earthquake Stability:* This is calculated using laboratory undrained strengths, determined on representative soil samples that have

been subjected to the cyclic loads comparable to the anticipated earthquake.

(4) *Dynamic Finite Element Analysts:* A coupled two- (or three-) dimensional analysis using an appropriate constitutive soil model will provide details concerning stresses, strains, and permanent displacements.

From the above list, the first two methods have become well established in general geotechnical engineering practice, mainly due to their ease of implementation, familiarity, and financial economics. The post-earthquake stability method is simple to implement, but requires extensive dynamic laboratory testing to determine the shear strength of the soils along some of the preselected potential failure surfaces in the slope. The finite element analysis is also expensive, as it requires extensive laboratory testing to identify the parameters for the constitutive model and considerable computational resources (Abramson, et al, 1996).

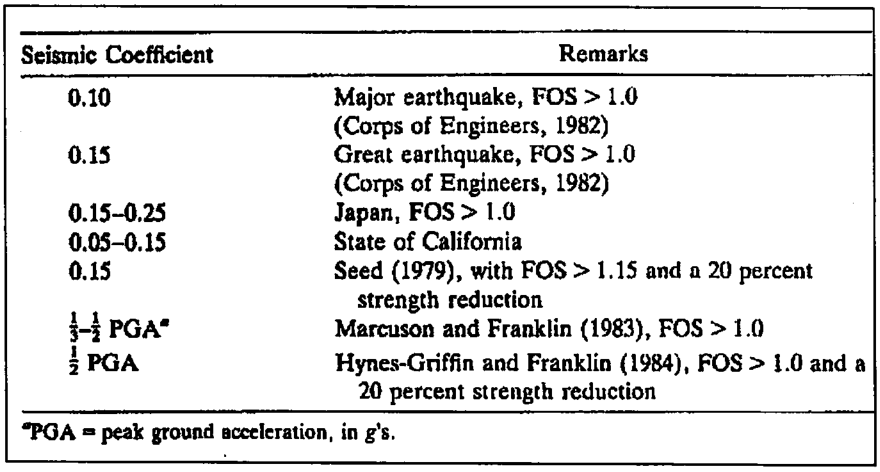
6.2.1 Pseudostatic Method

The pseudostatic method offers the simplest approach for evaluating the stability of a slope in an earthquake region. In its implementation, the limit equilibrium method is modified to include horizontal and vertical static seismic forces that are used to simulate the potential inertial forces due to ground accelerations in an earthquake. These seismic forces are assumed to be proportional to the weight of the potential sliding mass times a seismic coefficients, *kh* and *kv,* expressed in terms of the acceleration of the underlying earth (in units of g). It is recommended that only the most critical surface, as identified by a static analysis, should be reanalyzed using pseudostatic seismic coefficients, as it will be the most stressed region within the slope.

Typically, the seismic force is presumed to act in a horizontal direction only, that is, *kv* = 0, inducing an inertial force, *kh.W,* in the slope, where W is the weight of the potential sliding mass (Abramson, et al, 1996) A factor of safety is then calculated using conventional methods. The greatest difficulty with this procedure involves the selection of an appropriate seismic coefficient and the value of an acceptable factor of safety.

The magnitude of the seismic coefficient should effectively simulate the nature of the expected earthquake forces, which will depend on, earthquake intensity, for example, peak ground acceleration (PGA), duration of shaking, and frequency content (Abramson, et al, 1996). Of course as a very conservative assumption, one can select a seismic coefficient that is equal to the peak ground acceleration expected at the slope. However, this conservatism will lead to a very uneconomic evaluation. The selection of such coefficients, therefore, must be rationalized if slopes are to be designed economically. Some typical seismic coefficients that have been used for evaluating the seismic stability of slopes are given in Table 6.2.

Table 6.2 Typical seismic coefficients and factor of safeties in practices.



Seismic Coefficient	Remarks
0.10	Major earthquake, FOS > 1.0 (Corps of Engineers, 1982)
0.15	Great earthquake, FOS > 1.0 (Corps of Engineers, 1982)
0.15–0.25	Japan, FOS > 1.0
0.05–0.15	State of California
0.15	Seed (1979), with FOS > 1.15 and a 20 percent strength reduction
$\frac{1}{3}$–$\frac{1}{2}$ PGA*	Marcuson and Franklin (1983), FOS > 1.0
$\frac{1}{2}$ PGA	Hynes-Griffin and Franklin (1984), FOS > 1.0 and a 20 percent strength reduction

*PGA = peak ground acceleration, in *g*'s.

6.2.2 Newmark's Displacement Method

The procedure proposed by Newmark (1965) extends the simple pseudostatic approach by directly considering the acceleration time history (accelerogram) of the slide mass within the slope. This accelerogram, selected to represent a realistic model of the ground motions expected at the site, is then compared with the yield acceleration to determine permanent displacements. Newmark's method assumes: (1) existence of a well-defined slip surface, (2) a rigid, perfectly plastic slide material, (3) negligible loss of shear strength during shaking, and (4) permanent strains occur only if the dynamic stress exceeds the shear resistance. Also, the slope is only presumed to deform in the downslope direction, thus implying infinite dynamic shear resistance in the upslope direction. The procedure requires that the value of a yield acceleration or critical seismic coefficient, ky, be determined for the potential failure surface using conventional limit equilibrium methods. The main difficulty associated with this method is related to the selection of an appropriate accelerogram that simulates the motions of the slide mass. However, once this has been selected, the permanent displacements are calculated by double integration of the portions of the accelerogram that exceed the yield acceleration for the critical failure surface.

The calculated permanent displacements represent the motion of the center of gravity of the slide mass. For a planar slip-surface, the direction of this permanent displacement will be parallel to the slip surface. For the typical nonplanar failure surface, the direction of the permanent displacements is not immediately obvious. In such cases, the initial direction of the block's motion may be determined by considering the free-body forces that exist along the boundary of the slide mass. This direction may be calculated by first determining the resultant of all the shear forces and all the normal forces along the failure surface boundary. This essentially amounts to a vectorial summation of the shear and normal forces at the base of all slices, as determined in a limit equilibrium analysis. The permanent displacements are then assumed to act along the direction of the resultant of the cumulative shear and normal forces (Bromhead, 1992).

A typical ground response analysis consists of selecting an accelerogram to represent expected motions on bedrock, which should effectively simulate the intensity, duration and frequency content of the shaking motions. Then by using a numerical model, these bedrock motions are propagated through the overlying soil layers. Results from such an analysis can provide acceleration, stress and strain time histories within the geometric model of the slope.

Only a full, nonlinear numerical analysis can provide details about the actual permanent deformations that a slope or embankment may experience during an earthquake. The magnitude of the displacements computed by Newmark's approach are a qualitative reflection of the impact that the seismicity of the site will have on the stability of the slope. The tolerable levels of displacement that have been used to differentiate between safe and unsafe behaviour for example, Keefer and Wilson (1989) used 10 centimeters for coherent slides in Southern California.

6.3 Comparison and Selection of Analysis Methods.

There are three main methods of soil slope stability analysis. These are, limit equilibrium method, theory of plasticity solution and finite element method.

In the view of uncertainity and a lack of familarity with the finite element method, the complex approach is not used for the design and analysis of typical highway slopes and embankments (Abramson, et al, 1996).

Theory of Plasticity solutions can be applied if the soil is homogeneous and the shape of the slip surface is limited to those permitted for an upper bound solution (plane or logaritmic spiral) the limit equilibrium solution may be allowable as an upper bound (Nash, 1992).

As a result, although the limit equilibrium method of analysis gives neither a true upper nor lower bound value of the collapse loading, experience has shown that when used with care a good estimate of the collapse loading may be obtained. It has numerous advantages over the more rigorous plasticity methods, as account can readily be taken of non homogeneity of soil, seepage and surface loadings. Because of this, the method is widely used in engineering practice (Nash, 1992).

The accuracy of the analysis of a particular slope depends on the accuracy with which the geometry of the slope, the groundwater conditions, the soil properties, and the stability model of the slope. A number of studies have been made in which several methods are used to analyse the same problem. Fredlund and Krahn (1977) compared the results of a number of methods of analysis when applied to an example slope stability problem (Figure 6.4). Table 6.3 shows the results of Fredlund and Krahn (1977) study.

Duncan and Wright (1980) carried out a parametric study of homogeneous slopes, they found that Bishop's method and the methods satisfying all conditions of equilibrium, all give values of factor of safety which are within 5% of the value obtained by using a logaritmic spiral analysis, which is an upper bound value.

As a result one may conclude that; the methods which satisfy all conditions of equilibrium (Janbu's rigorous, Spencer's, Morgenstern and Price methods) all give accurate results (+-%5) for the analysis of slopes, Bishop's method which only satisfies moment equilibrium gives similarly accurate results except where the slip surface is steeply inclined at the toe; other methods which do not satisfy all conditions of equilibrium (ordinary method, force equilibrium methods) may be highly inaccurate (Nash, 1992).

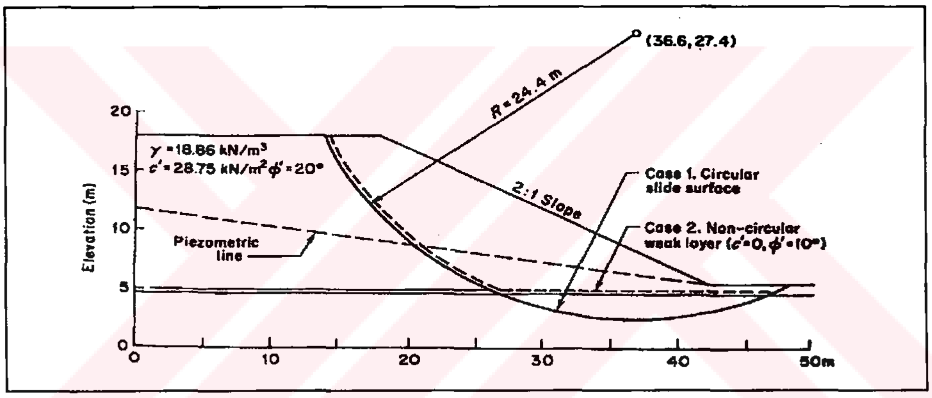


Figure 6.4 Problem used by Fredlund and Krahn.

Table 6.3 Results of Fredlund and Krahn study.

Case no.	Example problem*	Ordinary method	Simplified Bishop method	Spencer's method			Janbu's simplified method	Janbu's rigorous method†	Morgenstern–Price method $f(x)$=constant	
				F	θ	λ			F	λ
1	Simple 2:1 slope, 12 m high, $\phi'=20°$, $c'=28.75$ kPa	1.928	2.080	2.073	14.81	0.237	2.041	2.008	2.076	0.254
2	Same as 1 with a thin, weak layer with $\phi'=10°$, $c'=0$	1.288	1.377	1.373	10.49	0.185	1.448	1.432	1.378	0.159
3	Same as 1 except with $r_u=0.25$	1.607	1.766	1.761	14.33	0.255	1.735	1.708	1.765	0.244
4	Same as 2 except with $r_u=0.25$ for both materials	1.029	1.124	1.118	7.93	0.139	1.191	1.162	1.124	0.116
5	Same as 1 except with a piezometric line	1.693	1.834	1.830	13.87	0.247	1.827	1.776	1.833	0.234
6	Same as 2 except with a piezometric line for both materials	1.171	1.248	1.245	6.88	0.121	1.333	1.298	1.250	0.097

6.4 Back Analysis

Slope failures provide a valuable opportunity to estimate the strengths of materials, such as soils or geosynthetics, involved in the failure (Gilbert, et al, 1998).

Because of the difficulties inherent in the classical design approaches to slopes, one cannot expect to obtain, neither in the laboratory nor through field tests alone reliable strength parameters for rock discontinuities, soils and rock masses, back analysis of a slope failure often provides valuable information for future design purposes. This can only be meaningful, however, in the circumstances where the majority of factors that contributed to the failure can be evaluated. The most reliable way to obtain a statistically mean value of shear strength parameters of a slope forming material in a slope is back calculation (Ulusay, 1996).

Back analysis approach is based on the following assumptions:

- The geometry of the slope before and after failure must be known.
- A condition of static equilibrium at the point of failure exists at the time of failure (FOS=1) .
- The mechanism of the instability is known.
- Homogeneity and isotropy are not necessary conditions.
- The shear strength obtained from the back analyses is the weighted average shear strength mobilised along sliding surface.

Back analysis procedure proposed by Filz et al. (1992) would include four steps.

1- Laboratory test results and/or correlations with index properties are used to establish trial values of shear strength along the failure surface. Often to simplify the anaysis c^1 is assumed equal to zero.

2- A stability analysis is performed using slope geometry, groundwater levels, and external loading conditions at the time of failure. The analysis yields a factor of safety, that corresponds to the trial strengths from step 1.

3- The trial strengths from step 1 are adjusted using the safety factor computed in step 2.

4- The final back calculated strengths that produce a safety factor equal to unity are appropriate for the existing sliding surface, where the shear strength has been reduced to residual.

The confidence in back analysis is increased when the same set of strengths are in reasonable agreement with laboratory tests.

6.4.1 Residual Shear Strength.

The residual strength is the shear strength along a well defined failure surface at large displacements, it is independent of stress history and original structure (Mitchell, 1976). With continuing shear displacement the shear strength continues to decrease, below the critical state value, and eventually reaches to a residual value at a relatively large displacement

(Craig, 1992). Shear tends to orient platy or elongate particles parallel to the plane of movement due to drag or flow. The relative resistance to such reorientation varies with the fabric of the clay, the strength of the interparticle bonds, and with the orientation of the shear plane (Gillott, 1987).

If the clay minerals have a random or floccuated arrangement considerable energy will be required to reorient the particles regardless of the orientation of the shear plane. If the clay minerals already have a parallel arrangement less effort will be needed to effect shearing provided the orientation of then shear plane lies close to the direction of parallelism of the clay minerals. Hence the fabric of clay soils and orientation of potential shear planes with respect to a plane of preferred orientation of the clay minerals require consideration in estimating the shearing resistance (Gillott, 1987).

The proportions of platy to round particles and the coefficient of interparticle friction of the platy particles were found to control the type of residual strength mechanism (Lupini et al., 1981). Three modes of residual shear were recognised and termed turbulent, sliding and translational. In the turbulent mode the proportion of round particles is high, or the platy particles lack preferred orientation. A shear zone has a different porosity and brittleness results from dilatant behaviour. In the sliding mode, shear is dominated by low friction platy particles, initial brittle behaviour, results from development of strong preferred orientation of particles on a plane which offers low shear resistance. The translational mode occurs when there is no dominant particle shape so both türbulent and sliding modes occur together in different parts of a shear zone (Lupini et al., 1981).

Residual strength is given by $\tau_r = c_r^I + \sigma_n^1.\tan\phi_r^I$, where c_r^I and ϕ_r^I are the residual strength parameters in terms of effective stresses, for many soils the value of c_r^I is relatively low and can be taken to be zero (Craig, 1992). In general the value of ϕ_r^I decreases with increasing clay content.

Das (1983) also stated that residual shear strength can be given by $\tau_r=\sigma_n^1.\tan\phi_r^I$, in other words cohesion component is zero.

Thin bands of clay particularly those rich in smectite minerals are often associated with landslides (Gillott, 1987). Clay minerals have a high capacity for water uptake and when they are oriented the shearing resistance is low and such a band offers a favourable surface on which sliding may take place (Gillott, 1987).

CHAPTER 7

ASSESSMENT OF THE SLOPE INSTABILITY

7.1 Mechanism of 33 Evler Landslide

According to inclinometer measurements, the slip surface is circular, and located within the Saraycık formation which is a stiff clay. Figure 7.1 shows geological cross section and the failure surface based on the inclinometer data. According to Varnes (1978) classification system, 33 Evler landslide is a rotational slide (earth slump). Failure surface terminates at the shoreline, slightly towards sea. Local people have stated that large blocks at the shore line were tilted and overturned. 33 Evler landslide has occurred ten years ago following a heavy rainstorm and it is continuing to slide with an extremely slow rate according to inclinometer data. Before the landslide, there were about, one to three storey twenty buildings at the slipped area, based on the personal communication with local people. Causes of instability were probably due to heavy rainfall, removal of toe support by sand extraction from the sea and/or coastal erosion and excess surcharge loads (building loads).

33 Evler landslide can be subdivided into two sectors; one is the area below scarp 1 (Figure 7.2), and the other one is the area below scarp 2

(Figure 7.3) as also shown in Figure 4.1. The deepest part of slip surface is about 22 m, at sector 1 and 12 m at sector 2. Figure 7.4 shows the topographic model of the landslide. Retaining wall just above the road, probably prevented the advance of Scarp 1, because retaining wall is highly deformed (Figure 7.5) so Scarp 2 was formed below the retaining wall. Figure 7.6 shows the deformed road along the landslide due to continuous movement.

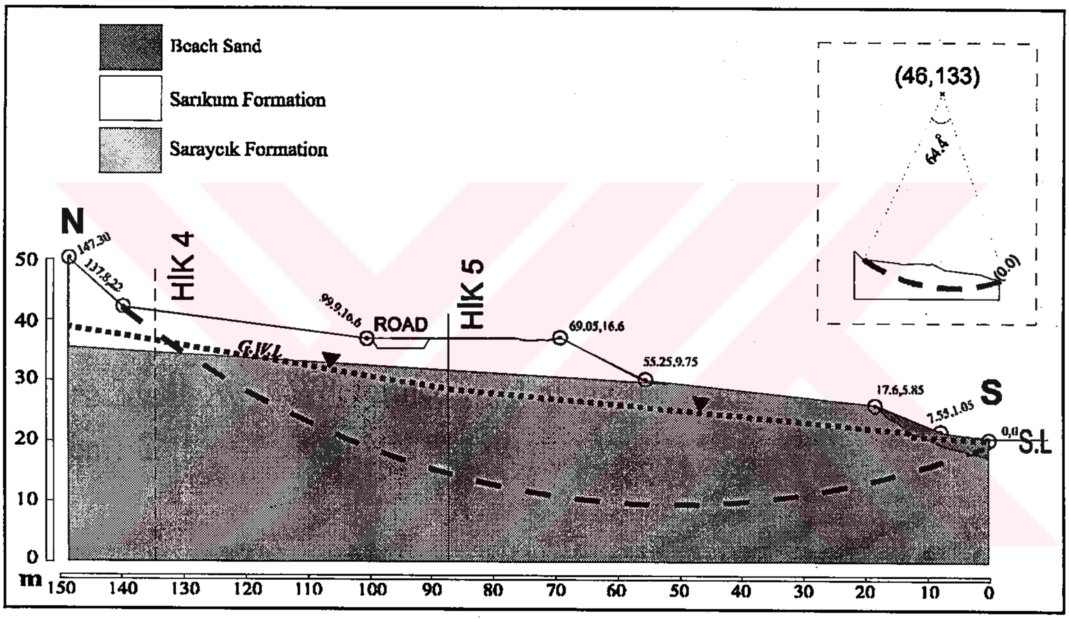


Figure 7.1 Geological cross section of the landslide investigated (cross section 1)

Figure 7.2 Panoramic view of scarp 1

Figure 7.3 Panoramic view of scarp 2

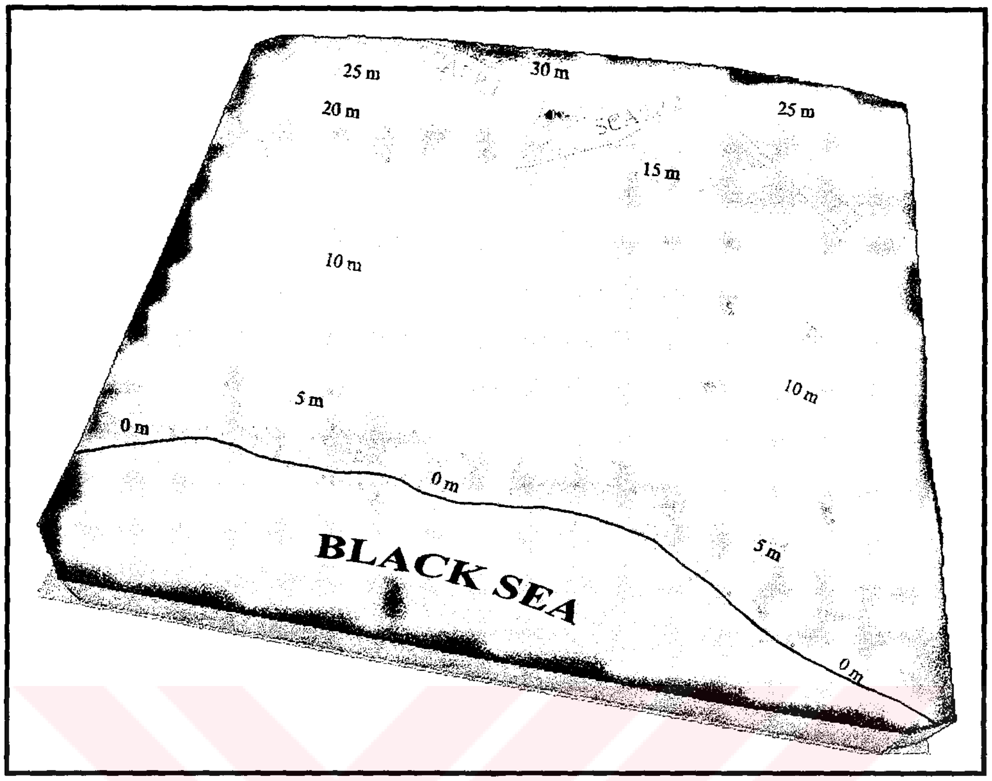


Figure 7.4 Topographic model of the area investigated

Figure 7.5 Deformed retaining wall at the east of scarp 1 (see Figure 4.1)

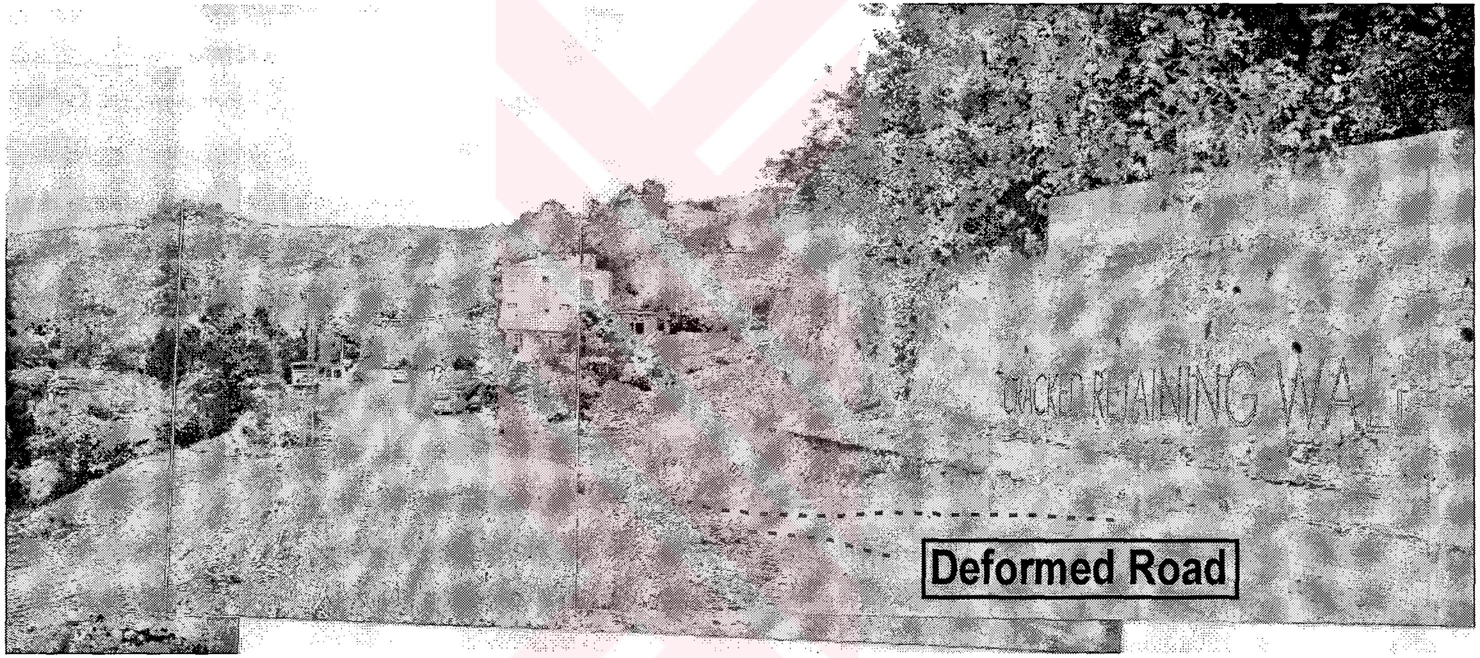


Figure 7.6 View of deformed road

7.2 Determination of the Shear Strength Parameters of the Failure Surface.

According to the laboratory and the field test results, the Saraycık formation can be described as stiff clay. However 33 Evler landslide is sliding with a slope angle of approximately 9 degrees.

There are many landslides around Sinop region within the Saraycık and Sarıkum formations. Both Saraycık and Sarıkum formations include considerable amount of smectite type of clay mineral (Arel, 1985). Smectite minerals have high water sorption capacity, and volume increase, thus their shear strength parameters, with a little displacement, can easily drop down to residual values.

In order to determine residual shear strength parameters associated with 33 Evler landslide, back analysis method was used for four cross-sections. Figure 7.7 shows the direction of the cross sections employed in the analyses.

During the back analysis one soil layer is assumed because, unit weights of the Saraycık and Sarıkum formations are almost the same, failure surfaces pass almost completely through Saraycık formation, and two soil layers introduces an additional difficulty in residual shear strength parameter determination in other words, two layers mean four unknown parameters to be solved. Figures 7.8, to 7.11 shows the cross-sections, sliding surfaces, groundwater levels, and center of rotations determined from borehole and inclinometer data.

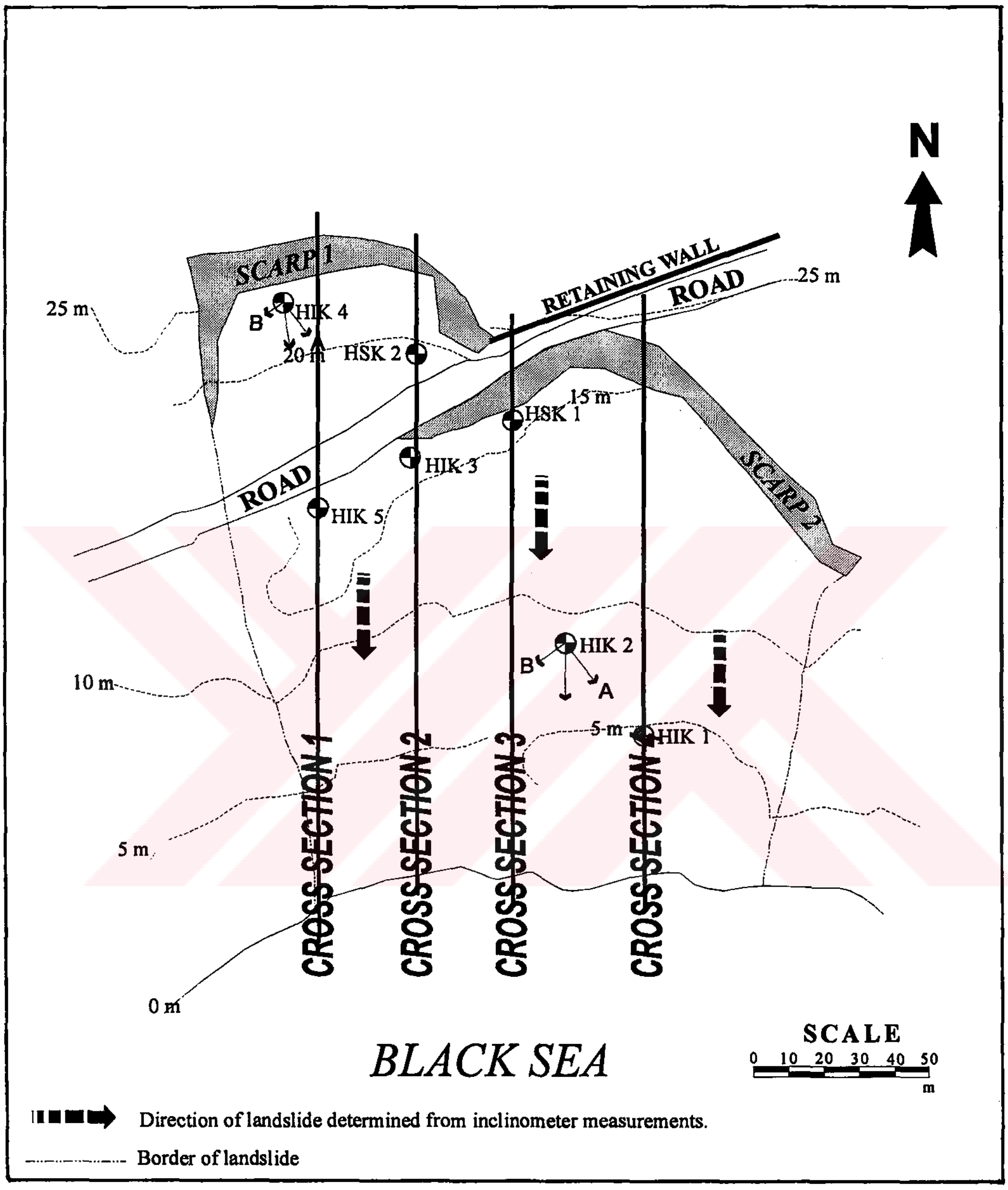


Figure 7.7 Cross section directions employed in the back analyses.

There is no surcharge load acting on the slope, because effect of buildings behind the scarp 1 is negligible. Figure 7.12 shows pressure bulb exerted by building behind scarp 1 (Drawings are scaled, elastic pressure distribution for a circular footing from Whitlow, (1983)).

The Bishop method was used in the analysis, because when the toe of the slope is not steeply inclined, error involved in using Bishop method against the more rigorous formulation is usually only 2% or less (Mostyn and Small, 1987). Results obtained from the multiple point c-ϕ method (Ulusay, 1996) back analyses using the computer program SGSLP (Sönmez and Gökçeoğlu, 1995) are presented in Table 7.1 and in Figure 7.13.

Table 7.1 Back analyses results showing the c-ϕ pairs of limiting equilibrium condition

C. Section 1 (c,ϕ) for FS = 1.0	C. Section 2 (c,ϕ) for FS = 1.0	C. Section 3 (c,ϕ) for FS = 1.0	C. Section 4 (c,ϕ) for FS = 1.0
29 kPa, 2 deg	19.6 kPa, 2 deg	13 kPa, 2 deg	16.4 kPa, 2 deg
20.2 kPa, 5 deg	12.6 kPa, 5 deg	8.4 kPa, 5 deg	11.5 kPa, 5 deg
11.1 kPa, 8 deg	5.4 kPa, 8 deg	4 kPa, 8 deg	6.5 kPa, 8 deg
0 kPa, 11.7 deg	0 kPa, 10.3 deg	0 kPa, 10.45 deg	0 kPa, 11.7 deg

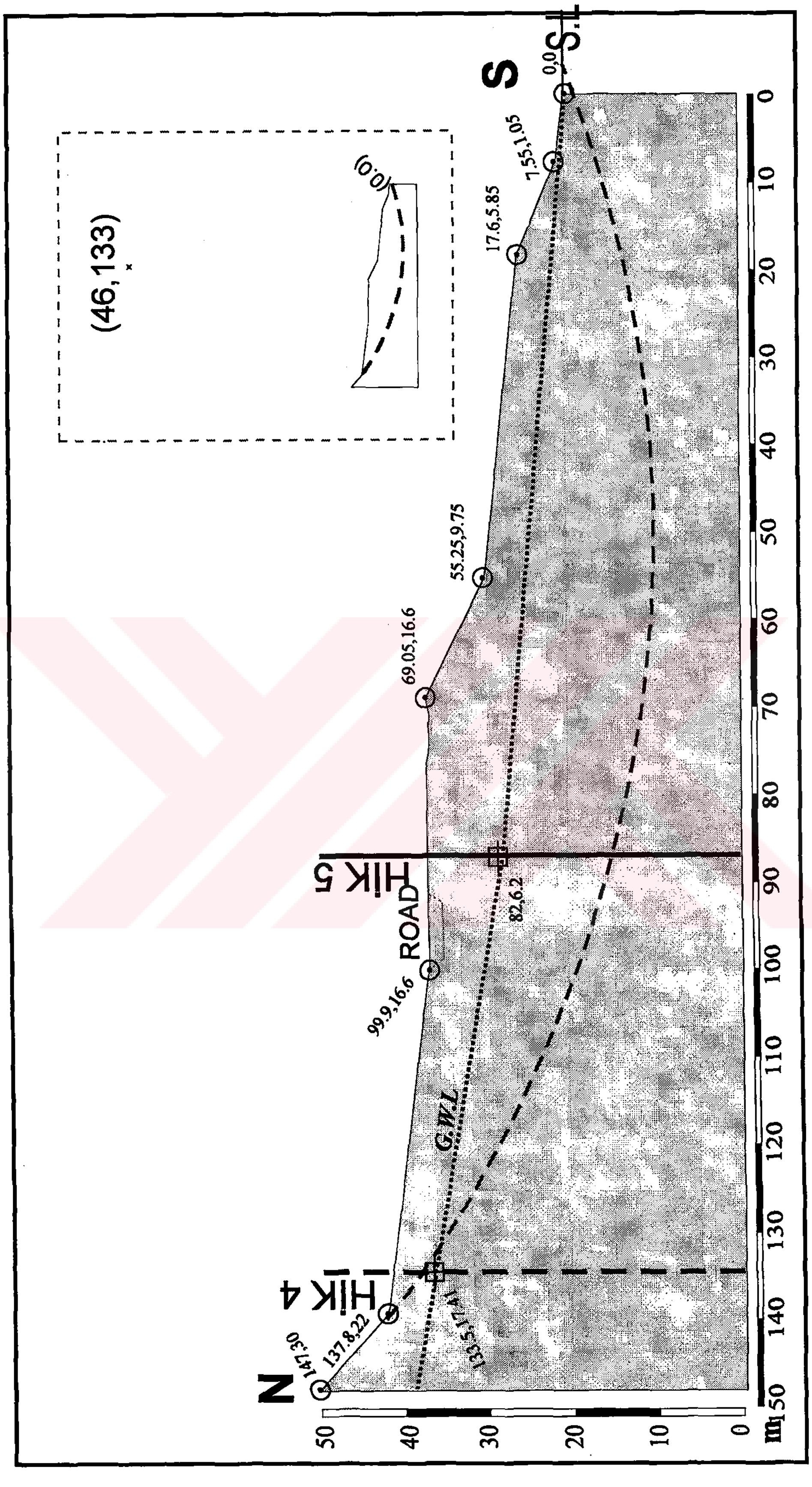


Figure 7.8 Cross - section 1

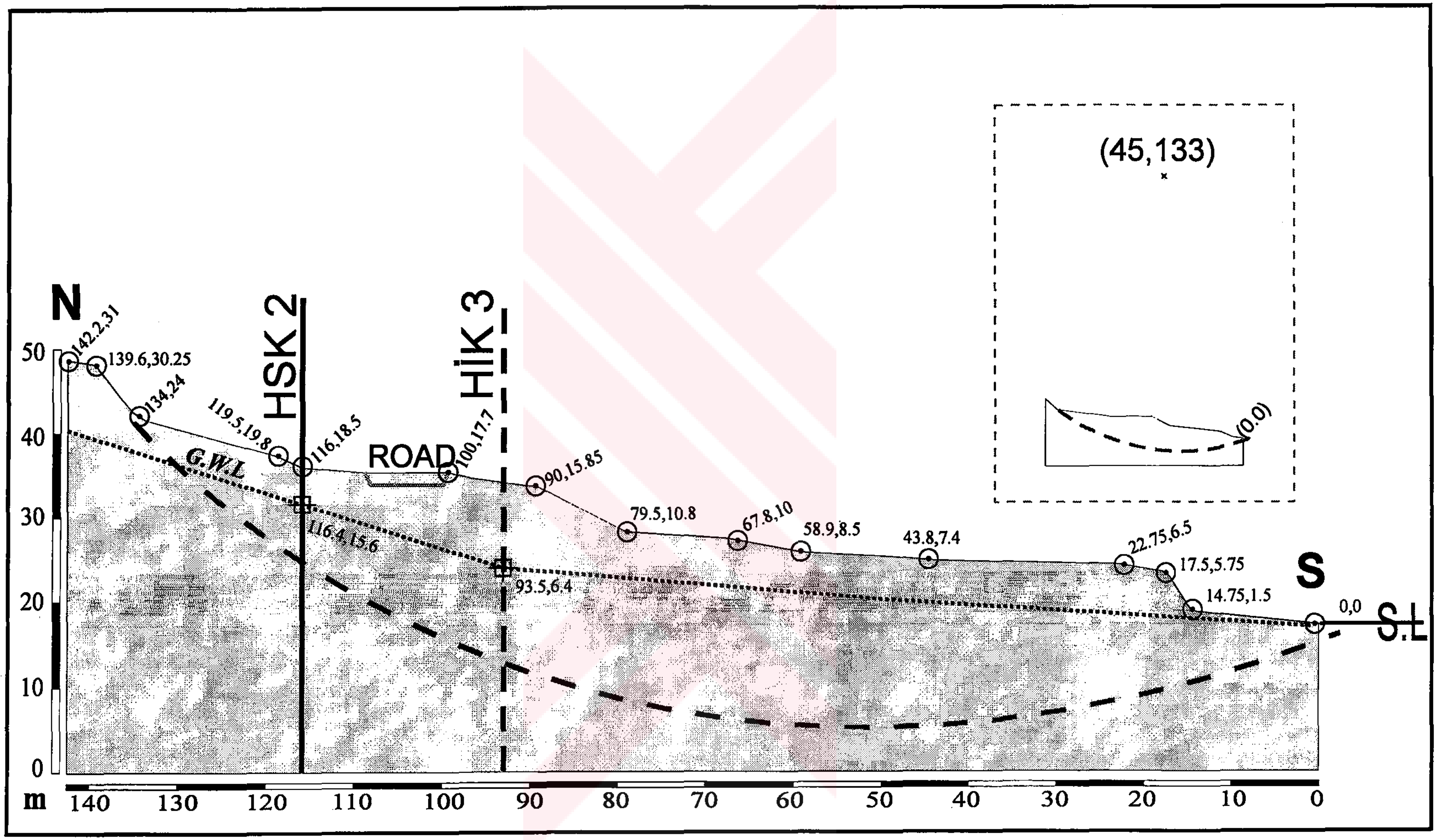


Figure 7.9 Cross - section 2

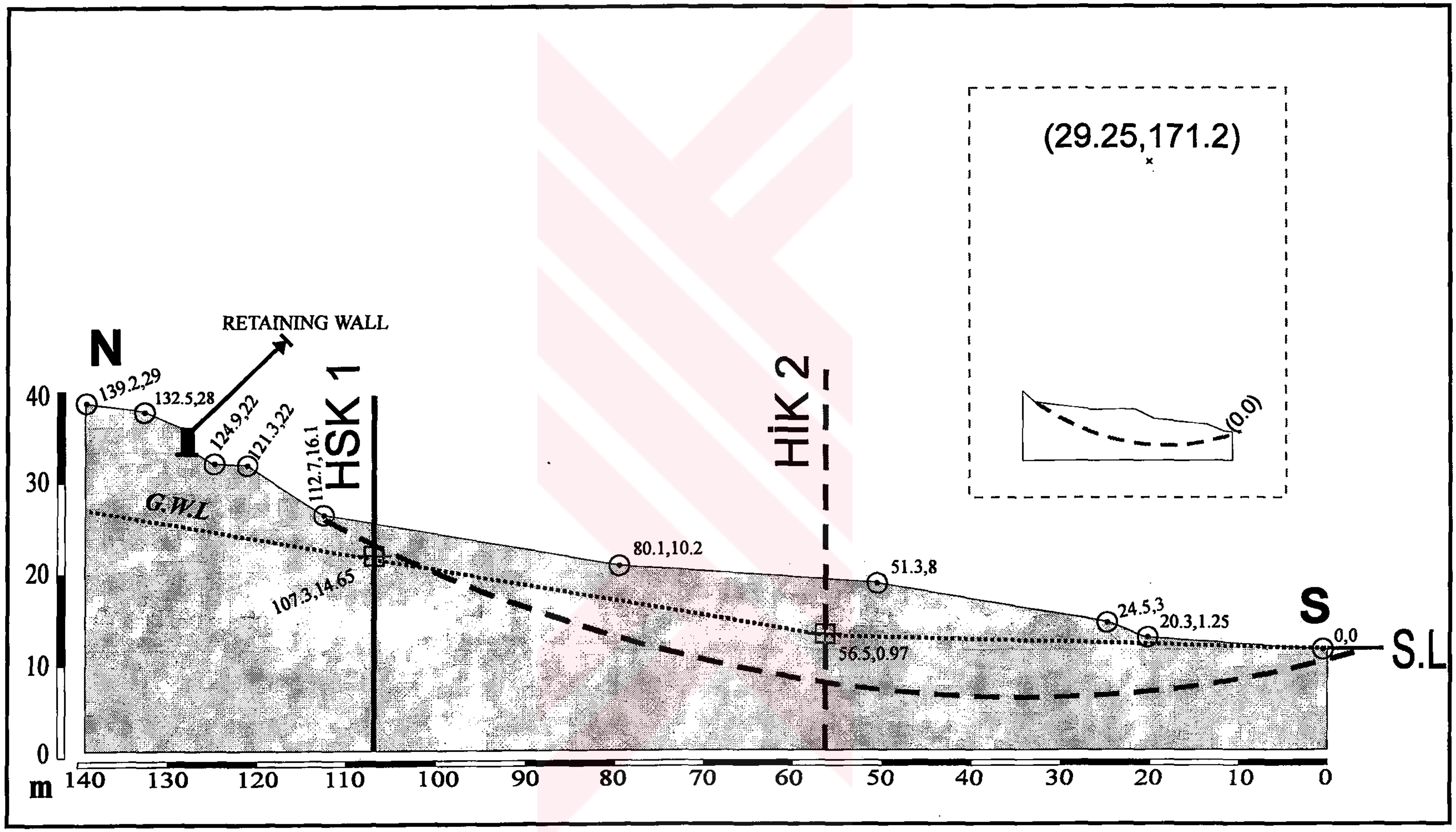


Figure 7.10 Cross - section 3

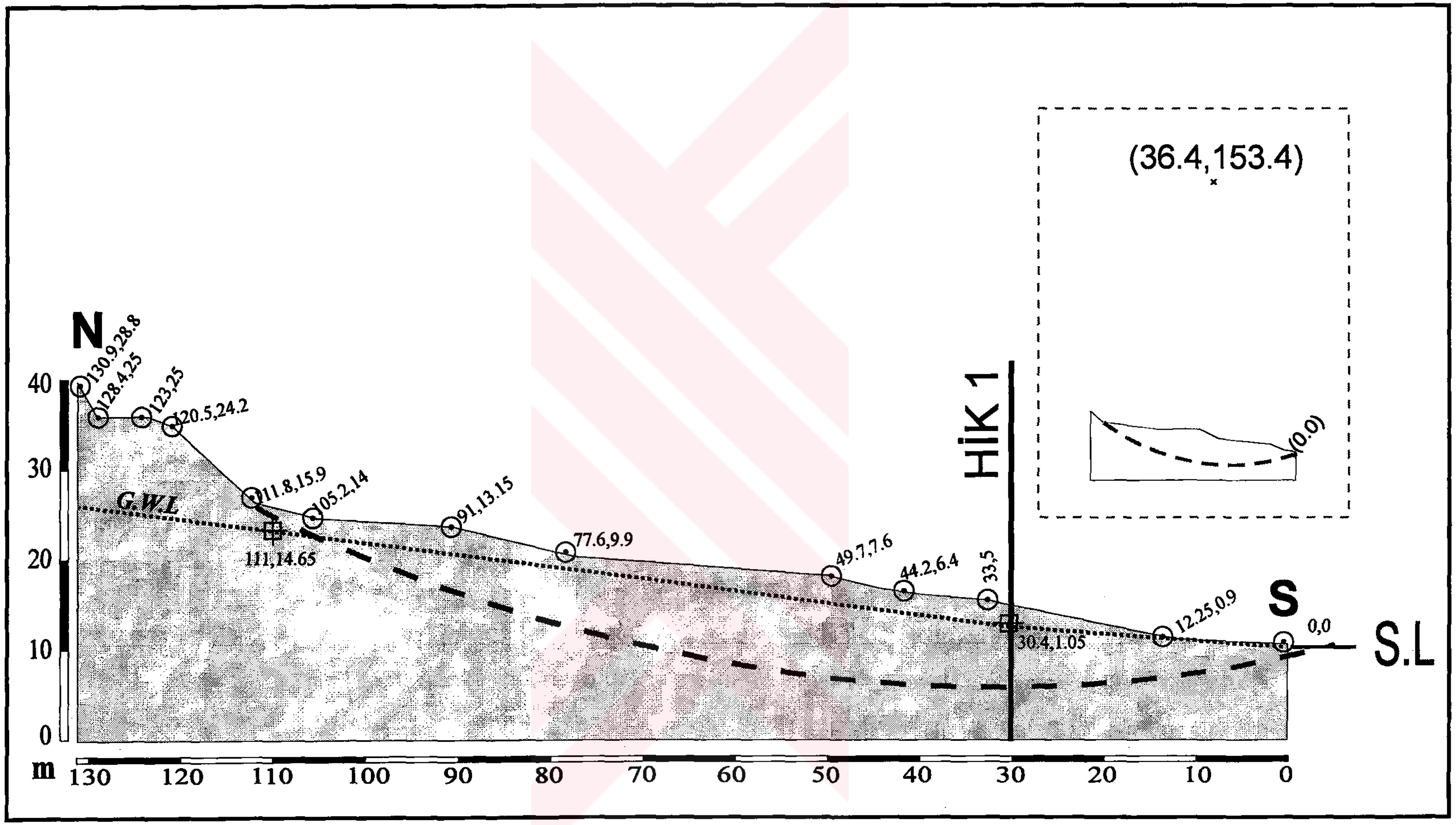


Figure 7.11 Cross section 4

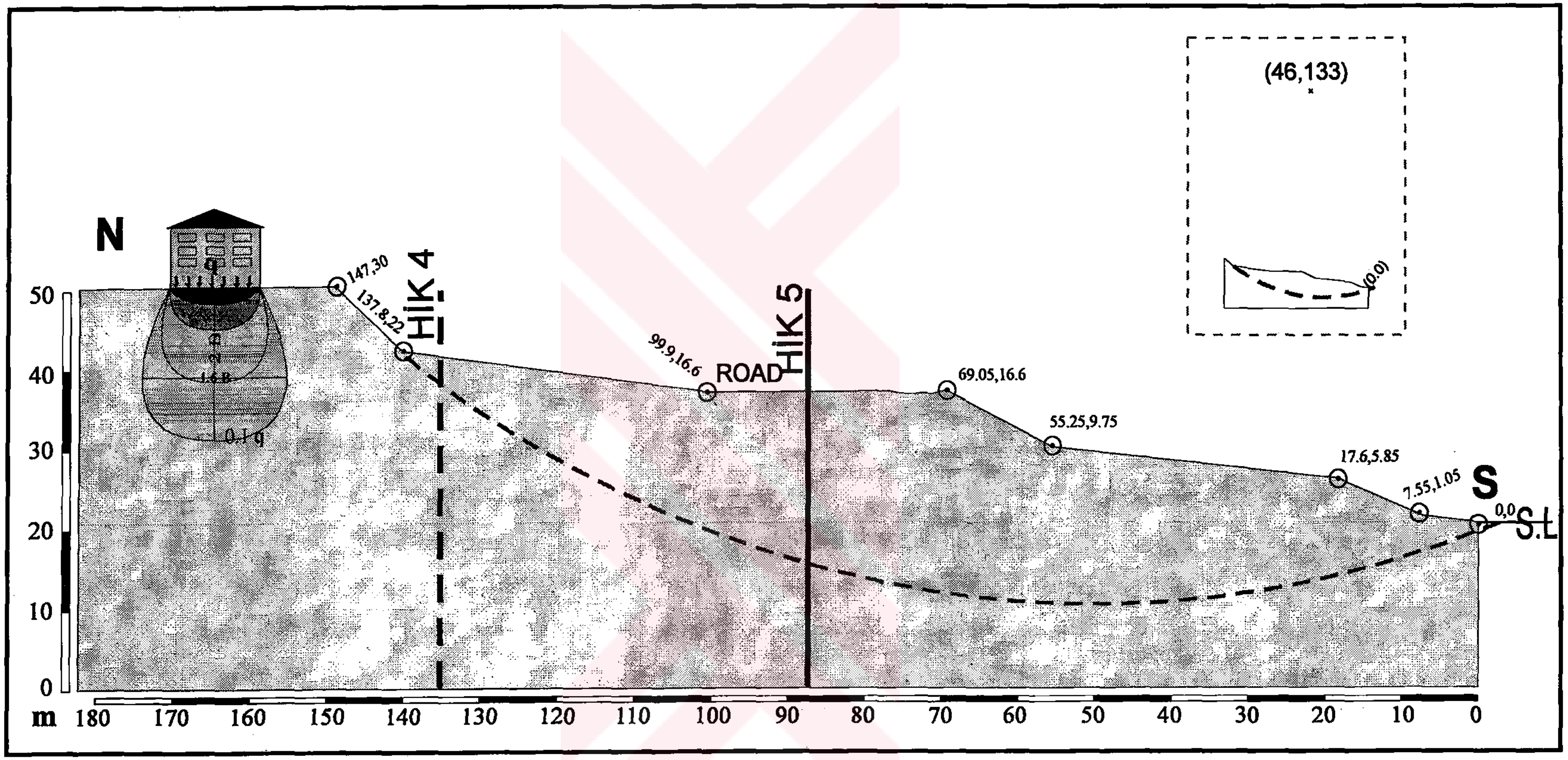


Figure 7.12 Pressure exerted by the building behind scarp 1 on cross - section 1.

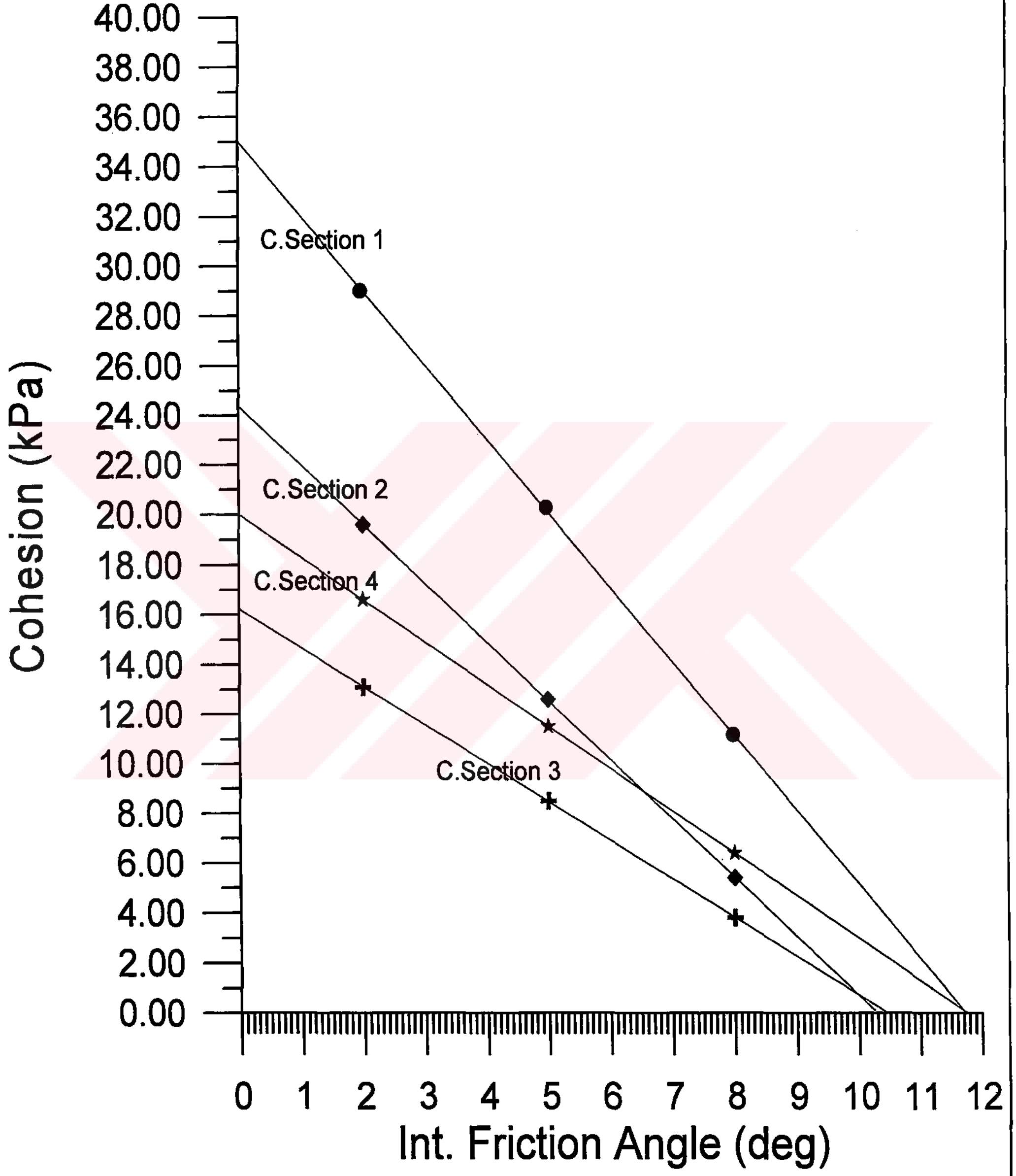


Figure 7.13 Graphical representation of back analysis results

The back analysis results, indicate that for a cohesion value of 0 kPa, 10.3°, 10.45° and 11.7° residual friction angles were obtained. Lines connecting, limiting equilibrium conditions for each cross section tend to intersect at zero cohesion, 10.3° - 11.7° zone. 10.3° friction angle was selected for final static and pseudostatic analyses. Bell (1992), gives similar residual shear strength parameters for Weald and Atherfield Clays (c=0 kPa, $\phi = 6^{\circ} - 14^{\circ}$).

7.3 Assessment for Slope Stabilisation

Generally slope stabilization involves some or all of the following:

- Reducing the destabilising forces in the slope by removing the sliding material or removing material from the upper part of slope.
- Increasing stabilising forces by adding weight to the toe of an unstable area or by increasing the shear strength along the failure surface.
- Supporting unstable areas by the construction of retaining walls.

7.3.1 Selection of Stabilisation Method

Subsurface drainage for 33 Evler landslide is practically impossible because of the low slope angle and low permeability of slope forming soils. Required drain hole length is very long (approximately 60 metres). Gedney and Weber (1978) state that subsurface drainage cannot be used effectively when sliding mass is impervious.

Increasing shear strength along the failure surface, by using laterally loaded piles or by an anchored wall is not practical, because, failure plane is rather deep seated. If an anchored wall is to be used, required pile lengths must be approximately 15 metres and required anchor lengths must be approximately 30 metres. Figure 7.14 shows these design alternatives.

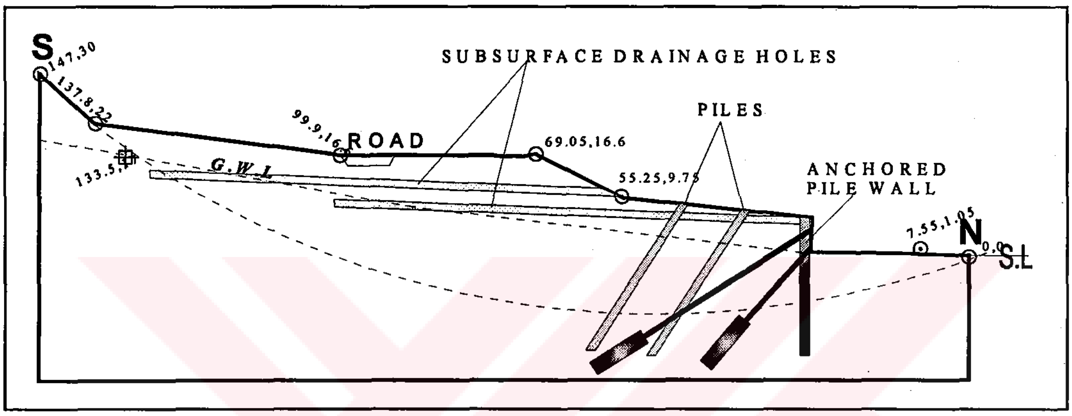


Figure 7.14 Design alternatives evaluated for the stabilisation of the investigated landslide

Because of the difficulties associated with subsurface drainage, pile or pile anchored system briefly discussed above, the toe support with surface drainage is selected as stabilisation technique.

7.3.2 Stabilisation of the Landslide by Toe Butressing

As stated before, rock buttress with surface drainage seems to be as the most suitable, and functional stabilisation method.

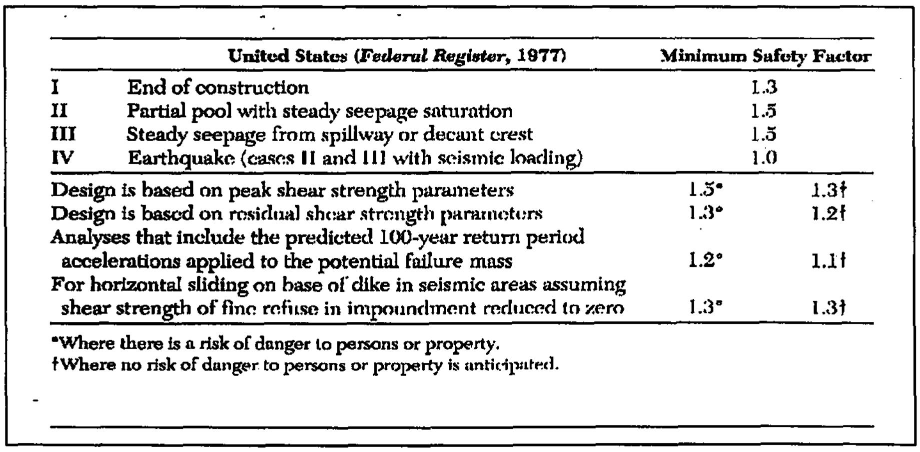
Rock buttress material can be obtained from the volcanic rocks of the Hamsaros formation cropping out at the Boztepe (flow breccia) or from the İnceburun (lava flows, volcanic breccias, agglomerates and volcanic conglomerates) sites, where rock material is fresh. Rock buttress will provide both toe support, and resistance to wave attack. In the slope stability analysis, unit weight of rock buttress material is taken as 25 kN/m^3 according to Fell, et al (1992).

Control of surface water consists of two parts. The collection of run off at the uphill boundary of an unstable area by surface ditches will minimize run off from at the unstable area. Scarp of the landslide and any cracks found on slope must be sealed in order to prevent, infiltration of surface water to failure plane. The volume of water to be collected depends on rainfall intensity, duration and catchment characteristics of the region.

7.3.3 Static and Dynamic Analysis of Stabilised Slope

According to Federal Register (1977), factor of safety of a stabilised slope under static loading is selected as minimally 1.3. Table 7.2 shows factor of safety selection rules according to Federal Register (1977). For seismic slope stability analysis pseudostatic approach was selected, because other methods of seismic analysis (Newmark, 1965; dynamic finite element method) require typical accelogram for bedrock motions.

Table 7.2 Factor of Safety Selection (United States Federal Register, 1977)



	United States (*Federal Register*, 1977)	**Minimum Safety Factor**	
I	End of construction	1.3	
II	Partial pool with steady seepage saturation	1.5	
III	Steady seepage from spillway or decant crest	1.5	
IV	Earthquake (cases II and III with seismic loading)	1.0	
	Design is based on peak shear strength parameters	1.5*	1.3†
	Design is based on residual shear strength parameters	1.3*	1.2†
	Analyses that include the predicted 100-year return period accelerations applied to the potential failure mass	1.2*	1.1†
	For horizontal sliding on base of dike in seismic areas assuming shear strength of fine refuse in impoundment reduced to zero	1.3*	1.3†

*Where there is a risk of danger to persons or property.

†Where no risk of danger to persons or property is anticipated.

Figure 7.15 shows effect of seismic force on a typical slice.

Figures 7.16 through 7.19 show stabilised slope sections 1, 2, 3, 4, respectively. Rock butress shapes were determined by trial and error procedure. For pseudostatic seismic analyses, peak horizontal ground acceleration is taken as 0.093 for 100 year return period and 0.119 for 225 year return period (for details refer to Chapter 5). Horizontal seismic coefficient used in the analysis is selected as 1/3 of PHGA (Marcuson and Franklin, 1983) (for details refer to section 6.4.1).

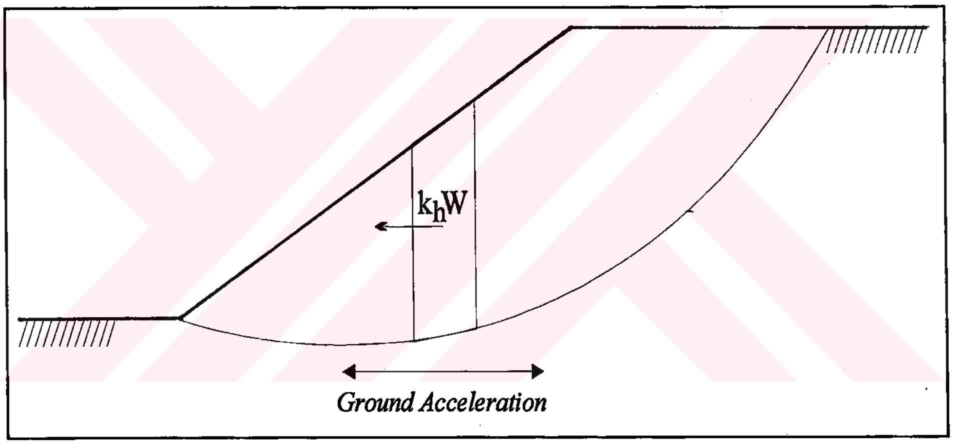


Figure 7.15 Effect of seismic force on a typical slice

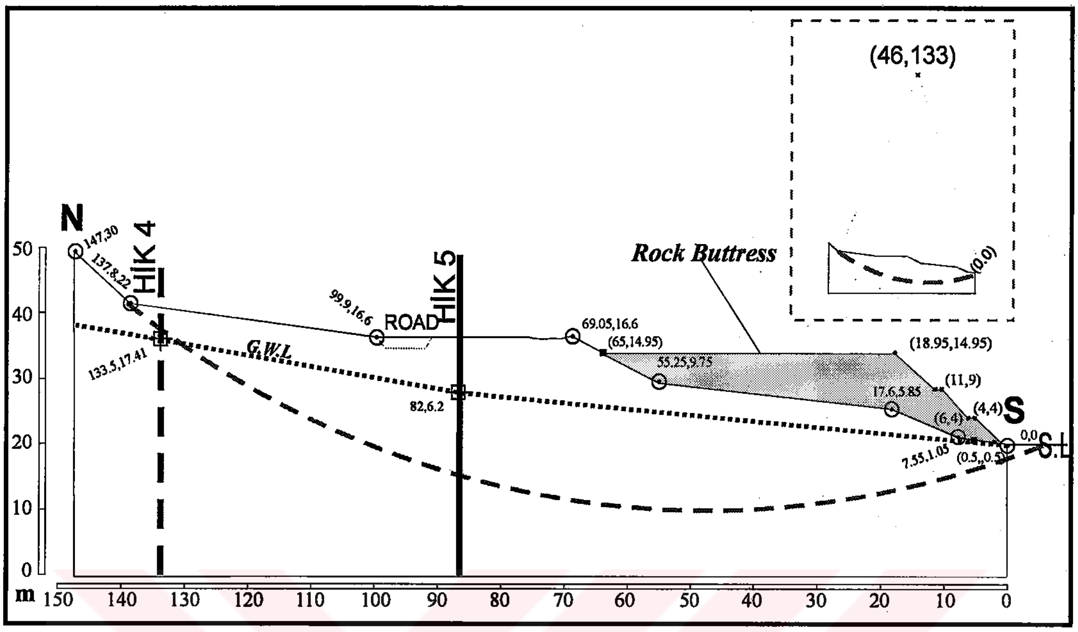


Figure 7.16 Stabilised slope, cross - section 1.

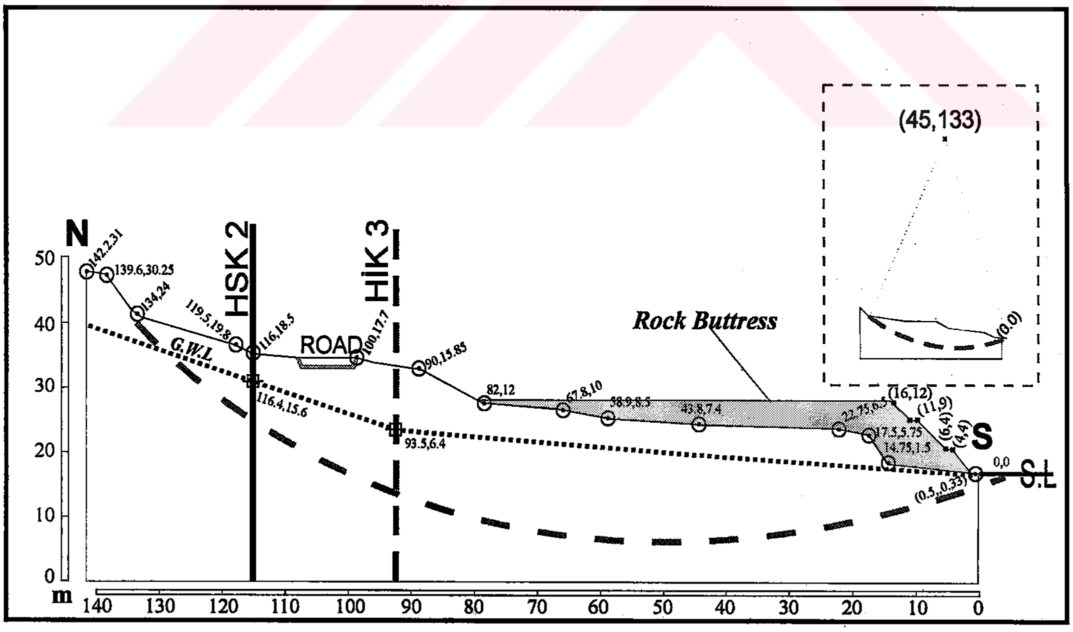


Figure 7.17 Stabilised slope, cross - section 2.

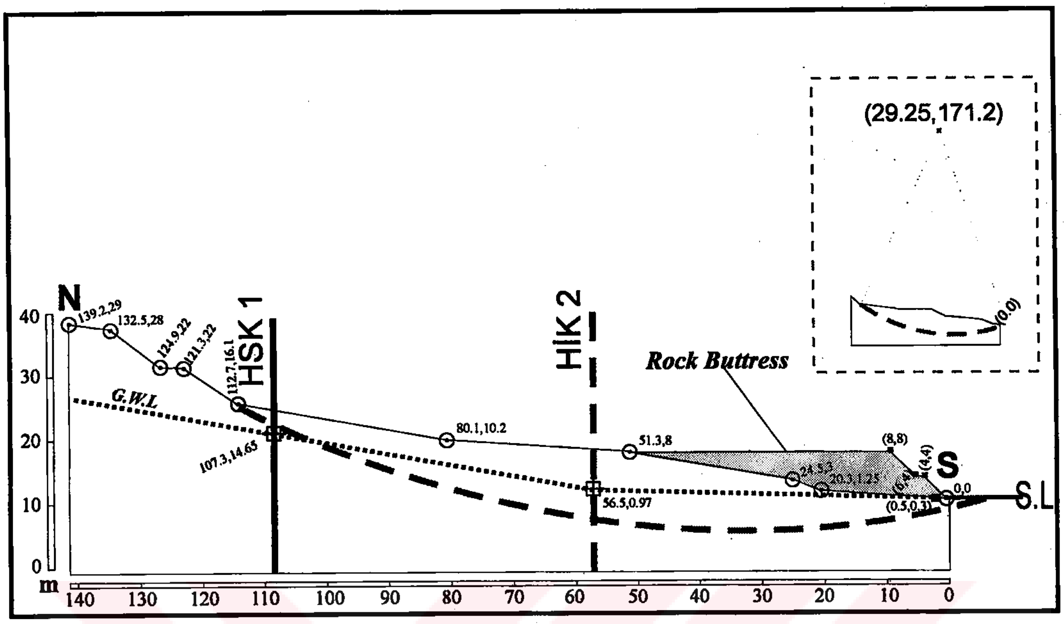


Figure 7.18 Stabilised slope, cross - section 3.

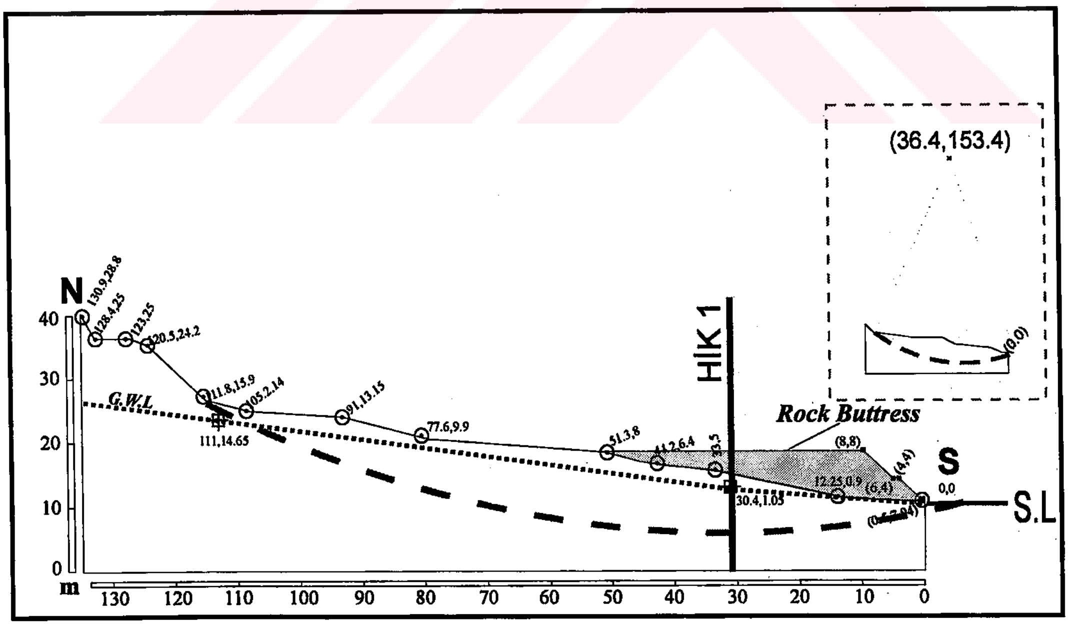


Figure 7.19 Stabilised slope, cross - section 4.

Stability analysis were performed by using Bishop's rigorous method with a computer program called Geosolve ver. 7.01. Table 7.3 shows values of factor of safety of stabilised slope sections under static and seismic conditions.

Table 7.3 Factor of safeties of stabilised slope sections.

CROSS-SECTION NO:	Static Factor of Safety.	Factor of Safety for k_h=0.031	Factor of Safety for k_h=0.04
1	1.645	1.240	1.158
2	1.558	1.200	1.124
3	1.882	1.562	1.441
4	1.882	1.342	1.238
K_h: 1/3 of 0.093 and 0.119			

The most important reason for the slope stability analyses performed on the cross-section 3 which yielded higher factor of safety than in other sections is that the groundwater level is lower than in cross-section 4. The reason of shallow groundwater level determined in this borehole is probably due to erroneous measurements taken during drilling.

According to Federal Register (1977), factor of safety of a slope should be minimally 1.2, for analysis that includes the predicted 100 year return period accelerations applied to the potential failure mass.

Results of the stability analysis show that the slope is stable under both static and seismic conditions, because factor of safety is grater than 1.3 for static case, greater than 1.2 for k_h = 0.031 and greater than 1.2 for k_h = 0.04.

However, because k_h is the most difficult parameter to determine, sensitivity analyses were performed by using k_h values on each cross section. Figure 7.19 and 7.20 show the result of sensitivity analyses.

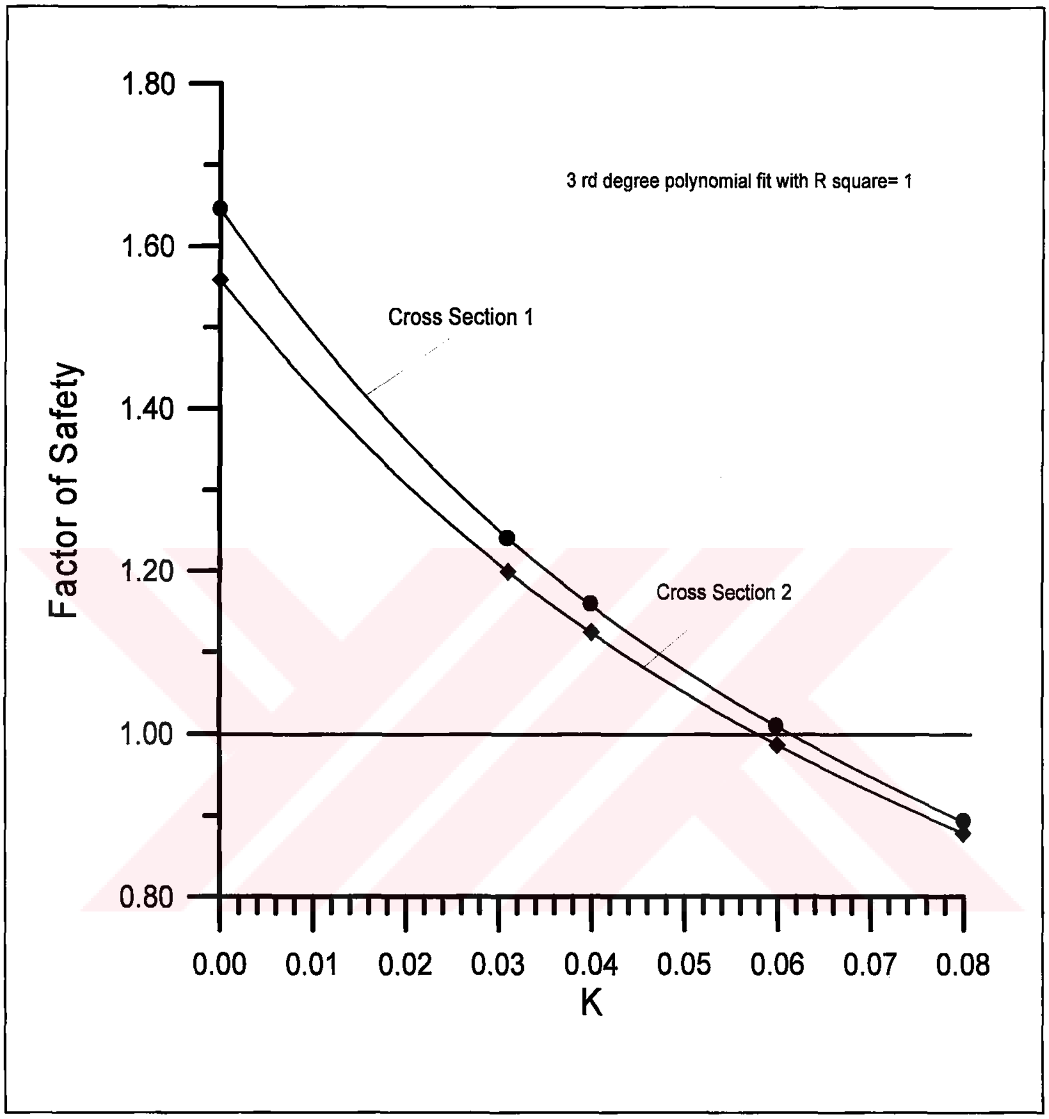


Figure 7.20 Results of the sensitivity analyses performed on cross-section 1 and 2.

According to Figure 7.19, failure will occur if k_h exceeds 0.063 on cross-section 1 and 0.058 on cross-section 2.

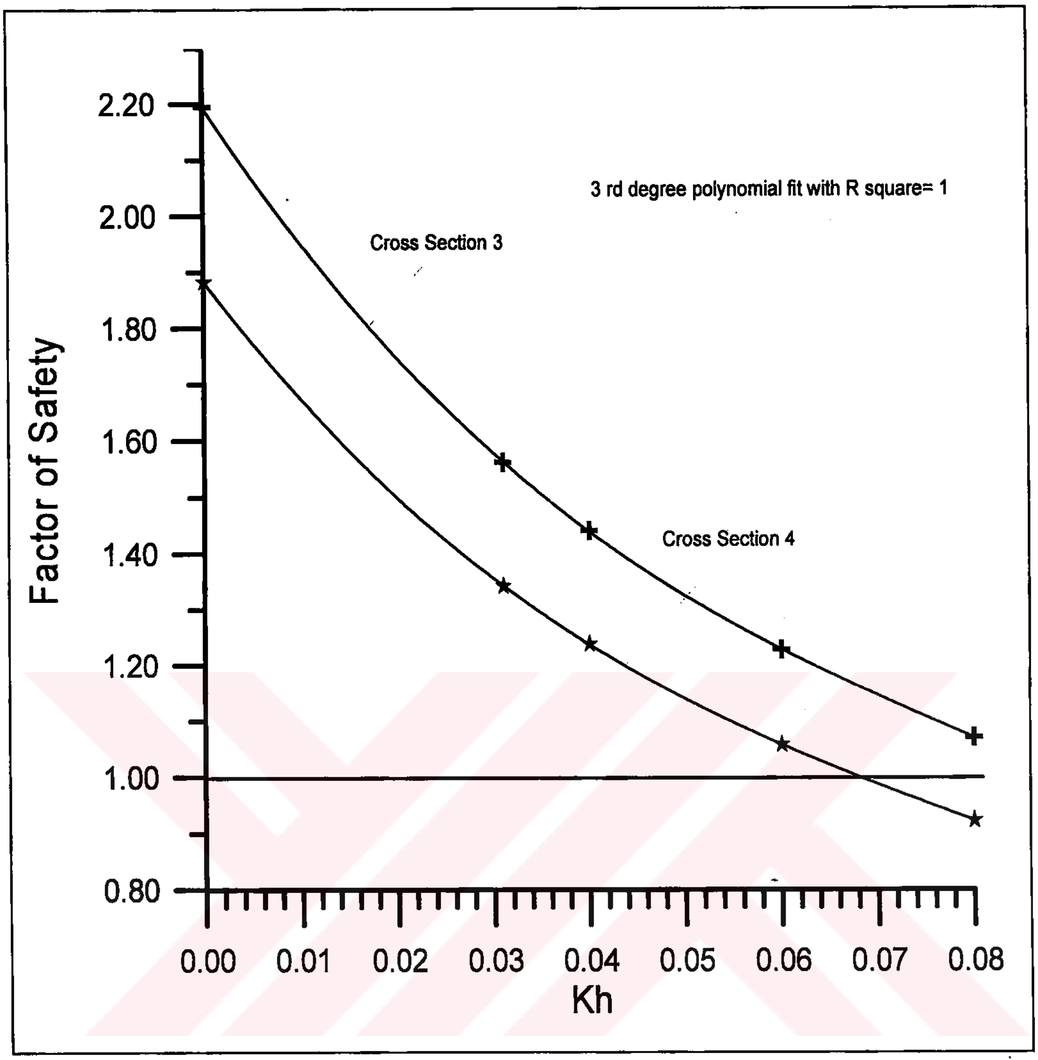


Figure 7.21 Results of the sensitivity analyses performed on cross-section 3 and 4

According to Figure 7.20, failure will occur if k_h exceeds 0.090 on cross-section 3 and 0.068 on cross-section 4.

As a result, rock buttress shown in Figures 7.15, thru 7.18 are adequate for stabilisation of 33 Evler landslide. However, scarp of 33 Evler

Landslide must be stabilised in order to prevent formation of a new failure at the back of the scarp with a proper retaining structure, like a soil nailed wall.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

This research deals with, the causes, analysis, and assessments on the stabilisation of a coastal slide located at the south of Sinop. The study area is located at the 33 Evler region, which is about one kilometer away from Sinop Bus Station.

Possible causes of the 33 Evler landslide were heavy rainfall, removal of toe support due to sand extraction and/or coastal erosion, and excess surcharge loads.

Sinop region is at the fourth degree earthquake zone, according to Turkish Earthquake Zoning Map. The seismicity of Sinop is mainly controlled by the North Anatolian Fault Zone located approximately 100 kilometer south of the town.

In order to investigate soil and rock conditions at the landslide area, Yüksel Project Construction Company drilled seven boreholes, performed SPT measurements, and obtained undisturbed soil samples. At five borehole, inclinometer measurements were taken, in order to determine the location of the slip surface.

Two main lithological units are involved at the landslide area. These are the Saraycık and the Sarıkum formations. The Saraycık formation is described as stiff clay, and Sarıkum Formation as medium stiff clay at the bottom and medium dense sand at the top. XRD analyses conducted on the samples, obtained from Saraycık and Sarıkum Formations show that the dominant minerals are clay, quartz, plagioclase and calcite; and the dominant clay minerals are smectite, illite and kaolinite. The Saraycık Formation contains more clay and calcite minerals than Sarıkum Formation.

According to inclinometer measurements and field observations the shape of the slip surface is circular and the type of the landslide is rotational. Slip surface is located mainly within the Saraycık Formation.

In order to determine residual shear strength parameters associated with the landslide, back analyses were performed along four cross sections by using Bishop Method. According to back analyses results, residual shear strength parameters are determined as $c_r = 0$ and $\phi_r = 10.3^{\circ}$ for the Saraycık Formation.

By considering the size of the landslide, depth of failure surface, characteristics of sliding, and the residual shear strength parameters, it was decided that the most appropriate stabilisation technique is rock buttress with surface drainage.

Static and pseudostatic slope stability analyses were performed using Bishop's Rigorous Method with residual shear strength parameters. It was seen that, the factors of safety are greater than 1.2 under possible earthquake loadings.

Scarp of the 33 Evler landslide must be stabilised in order to prevent formation of a new failure at the back of the scarp with a proper retaining structure, like a soil nailed wall. Any cracks, fissures observed on the landslide must be sealed to prevent infiltration of surface waters.

I strongly not recommend any settlement on the landslide, because buildings constructed on any area other than at toe of the slope will increase the driving forces; in other words, the factor of safety will decrease. At the toe of the slope there will be rock buttress, so buildings can not be constructed on rock buttress.

In order to avoid slope instability problems, carefully designed city planning is required. The plans must be based on the detailed engineering geological studies emphasising susceptibility of the slopes to sliding.

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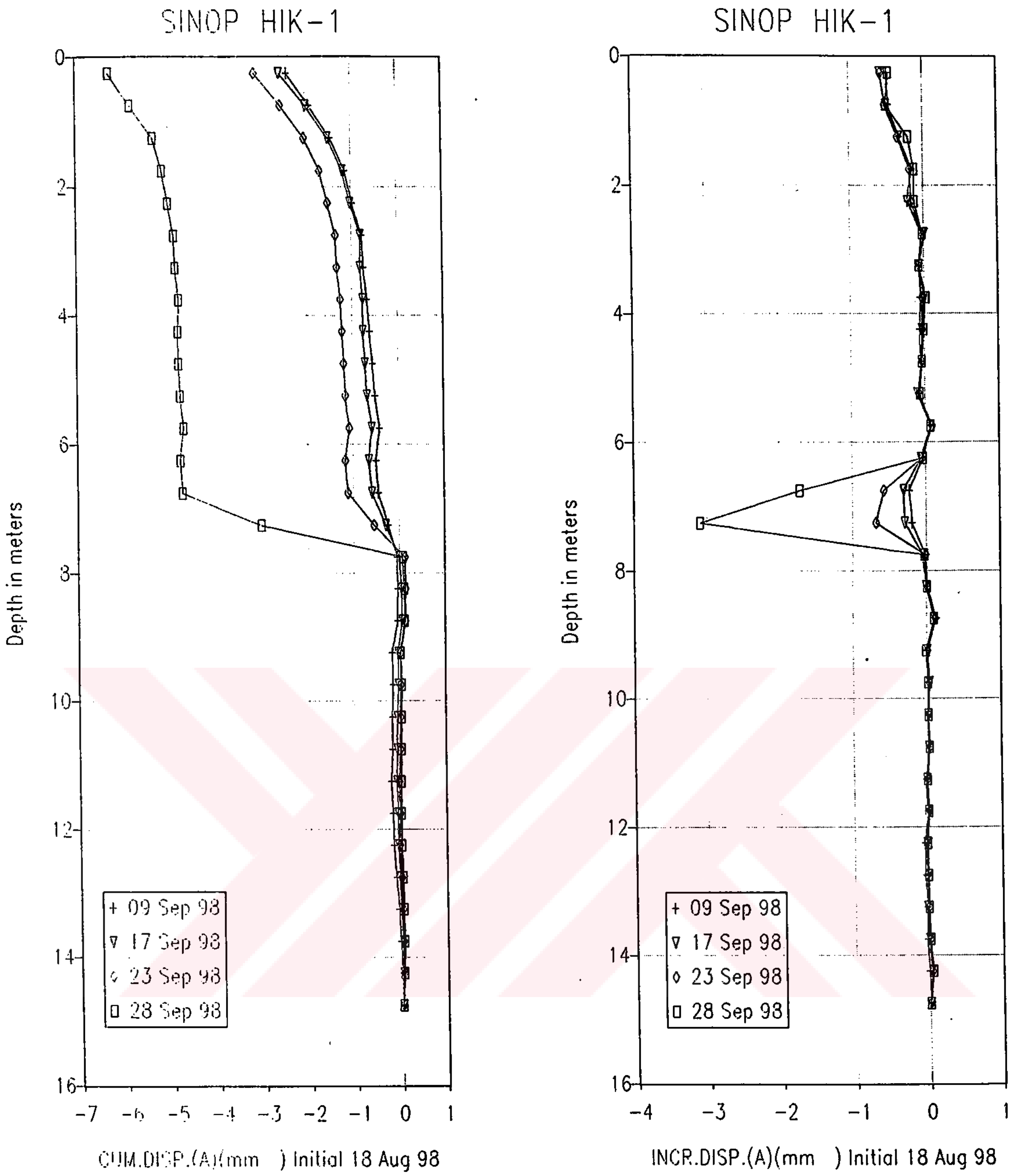
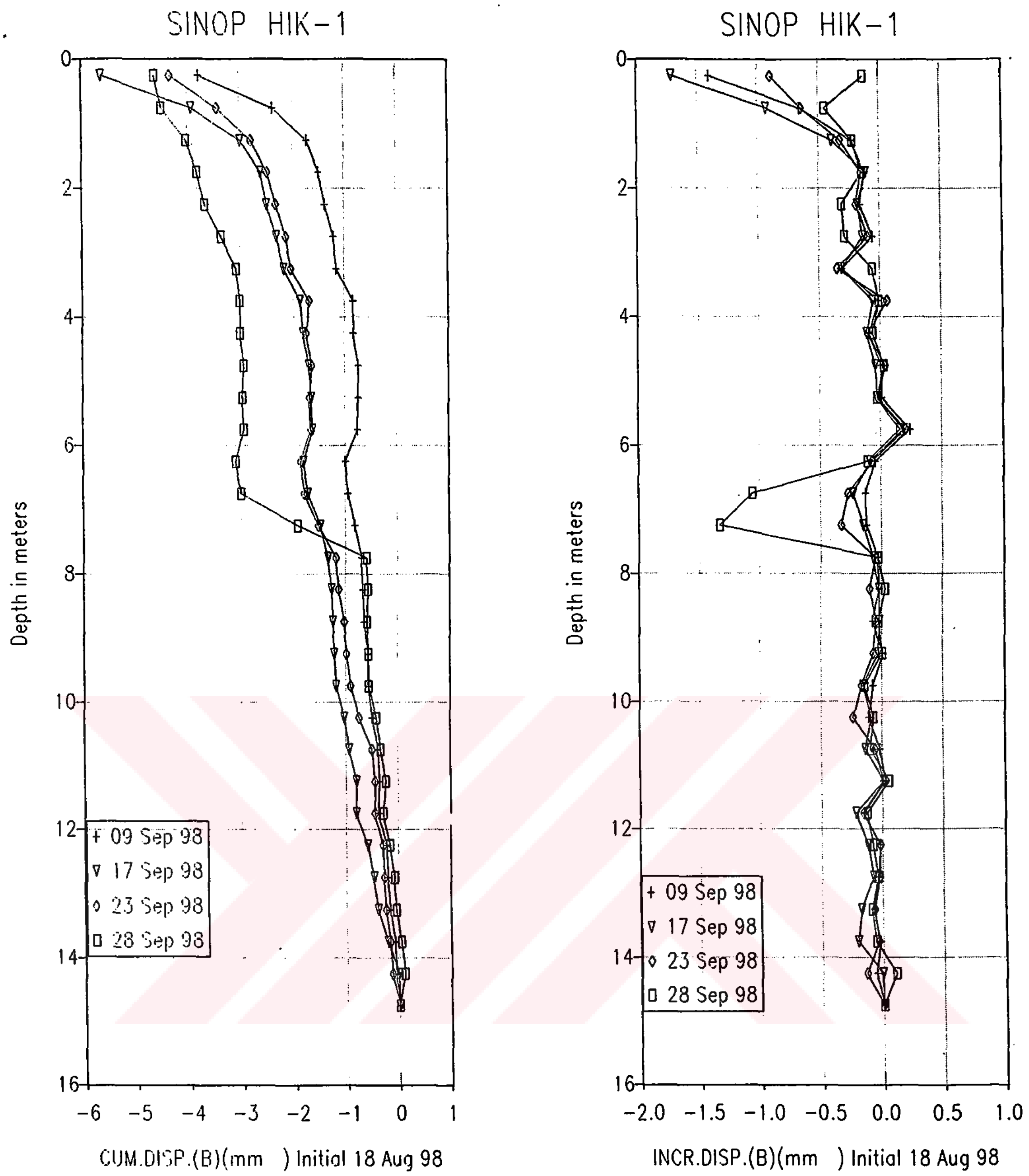
APPENDIX A

INCLINOMETER MEASUREMENT GRAPHS



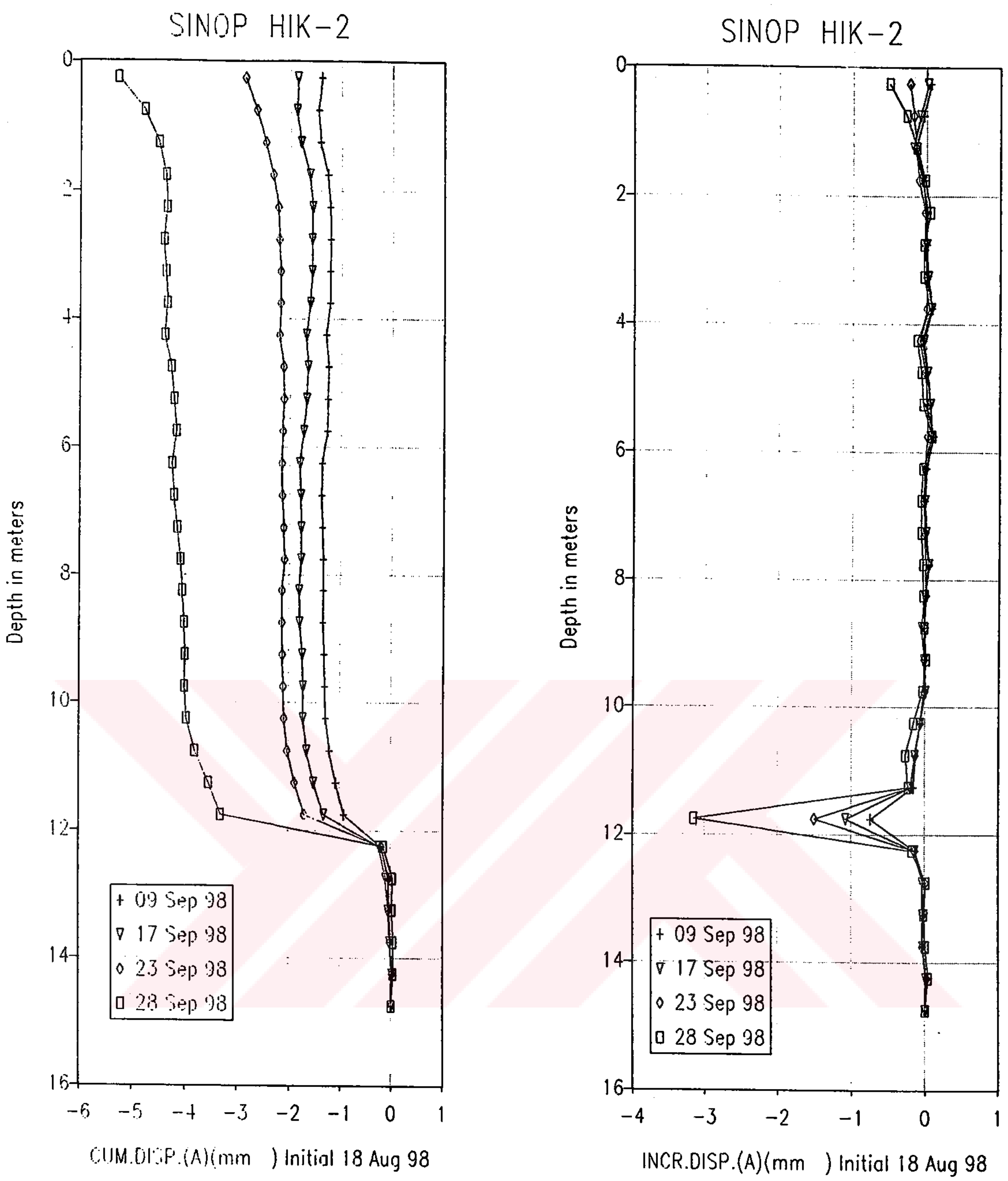
YÜKSEL PROJE Uluslararasi a.s.	SINOP YOLU

Figure A1. Inclinometer measurement graphs of the borehole Hik 1



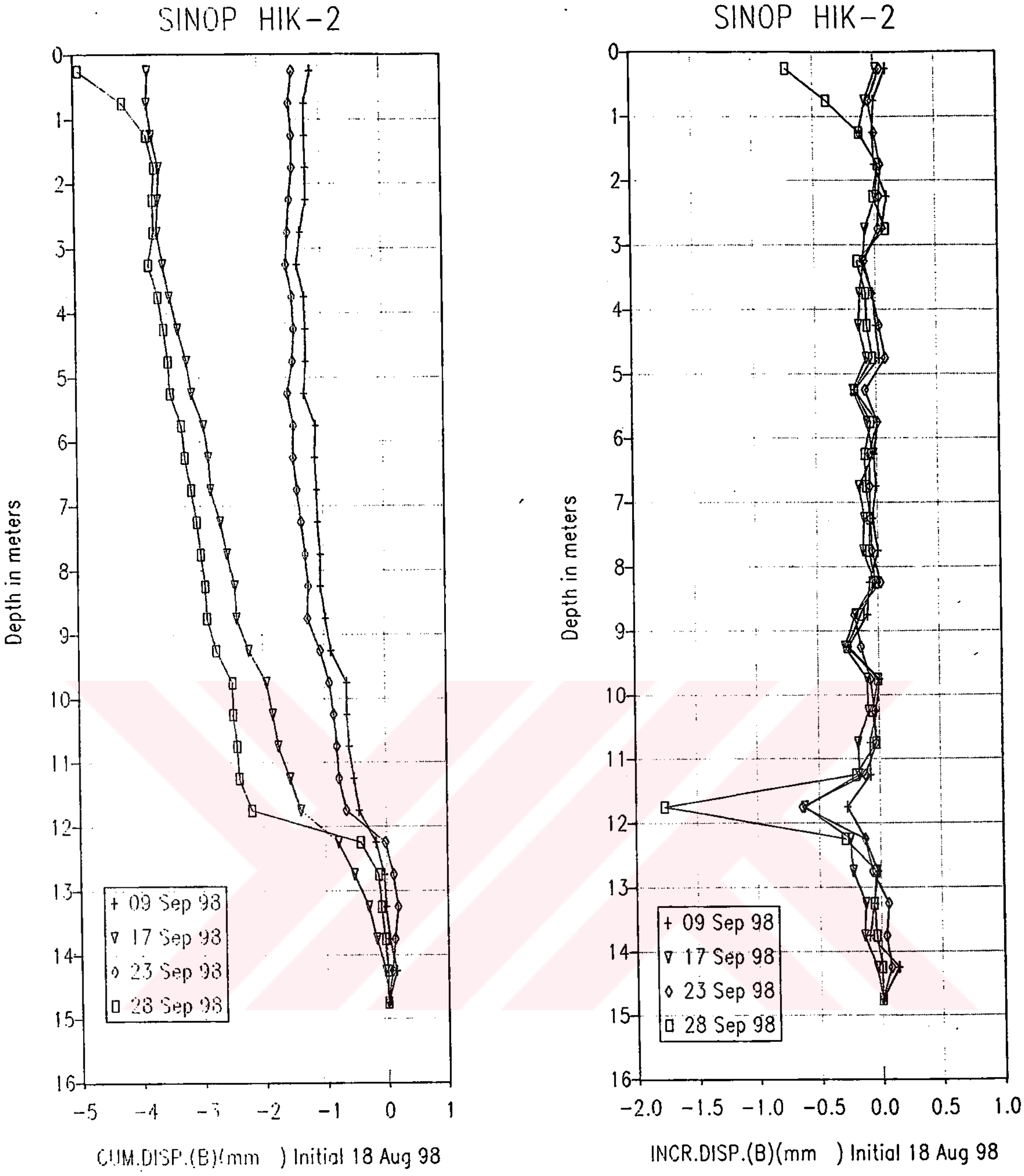
YÜKSEL PROJE Uluslararasi a.s.	SINOP YOLU

"Figure A1 (cont.)"



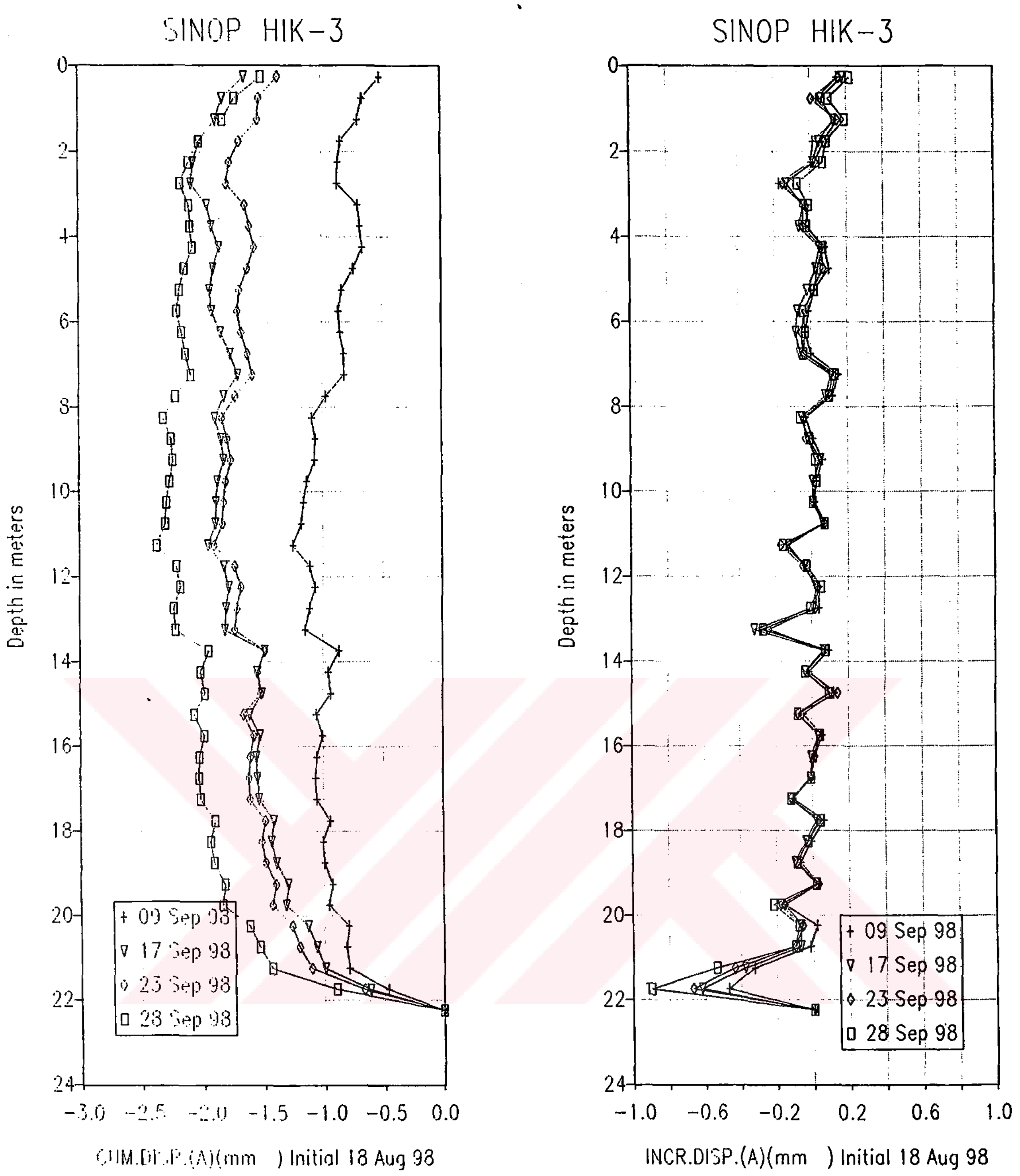
YÜKSEL PROJE Uluslararasi a.s.	SINOP YOLU

Figure A2. Inclinometer measurement graphs of the borehole Hik 2



YÜKSEL PROJE Uluslararasi a.s.	SINOP YOLU

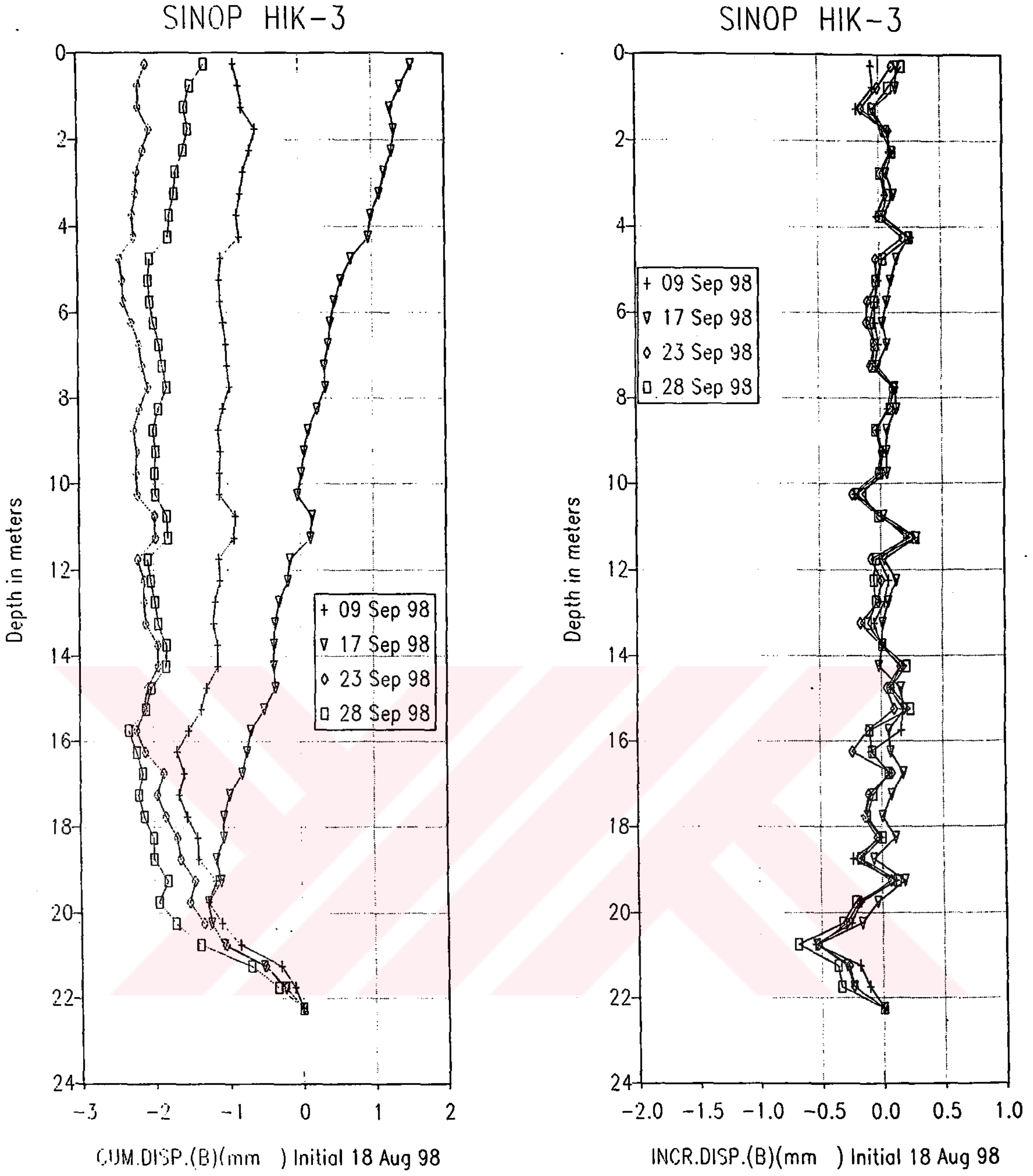
“Figure A2 (cont.)”



YÜKSEL PROJE
Uluslararasi a.s.

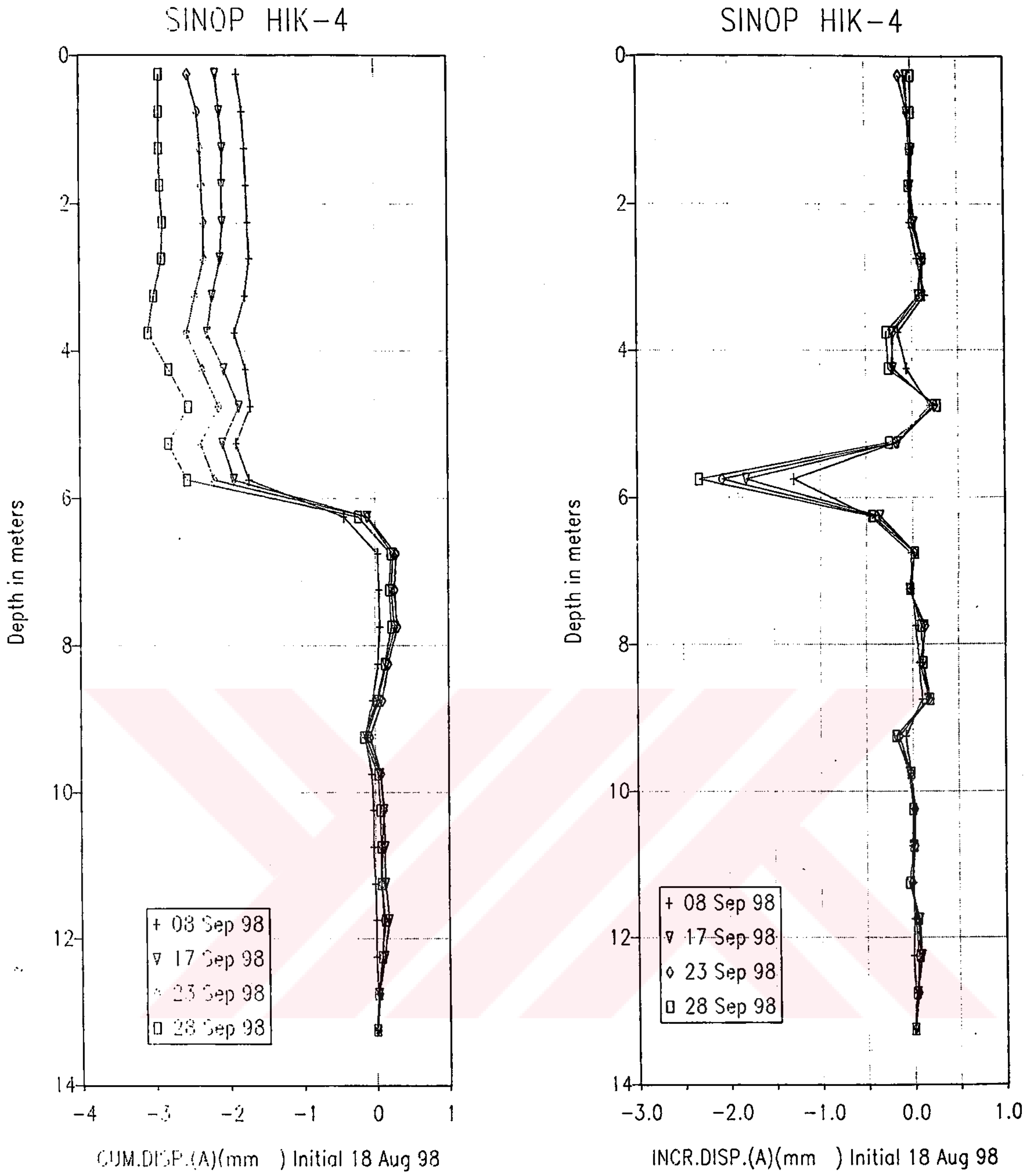
SINOP YOLU

Figure A3. Inclinometer measurement graphs of the borehole Hik 3



YÜKSEL PROJE Uluslararasi a.s.	SINOP YOLU

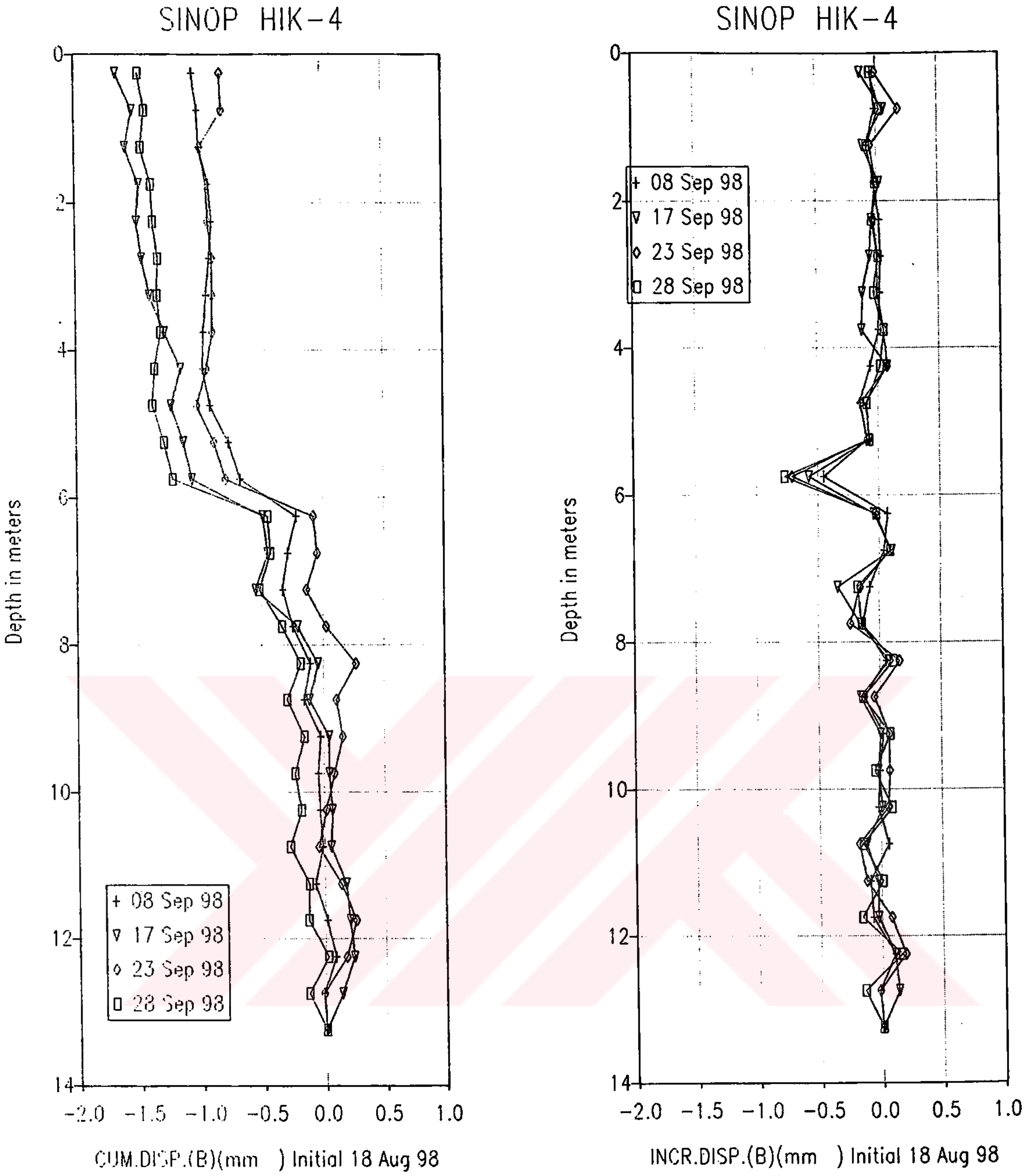
"Figure A3 (cont.)"



YÜKSEL PROJE
Uluslararasi a.s.

SINOP YOLU

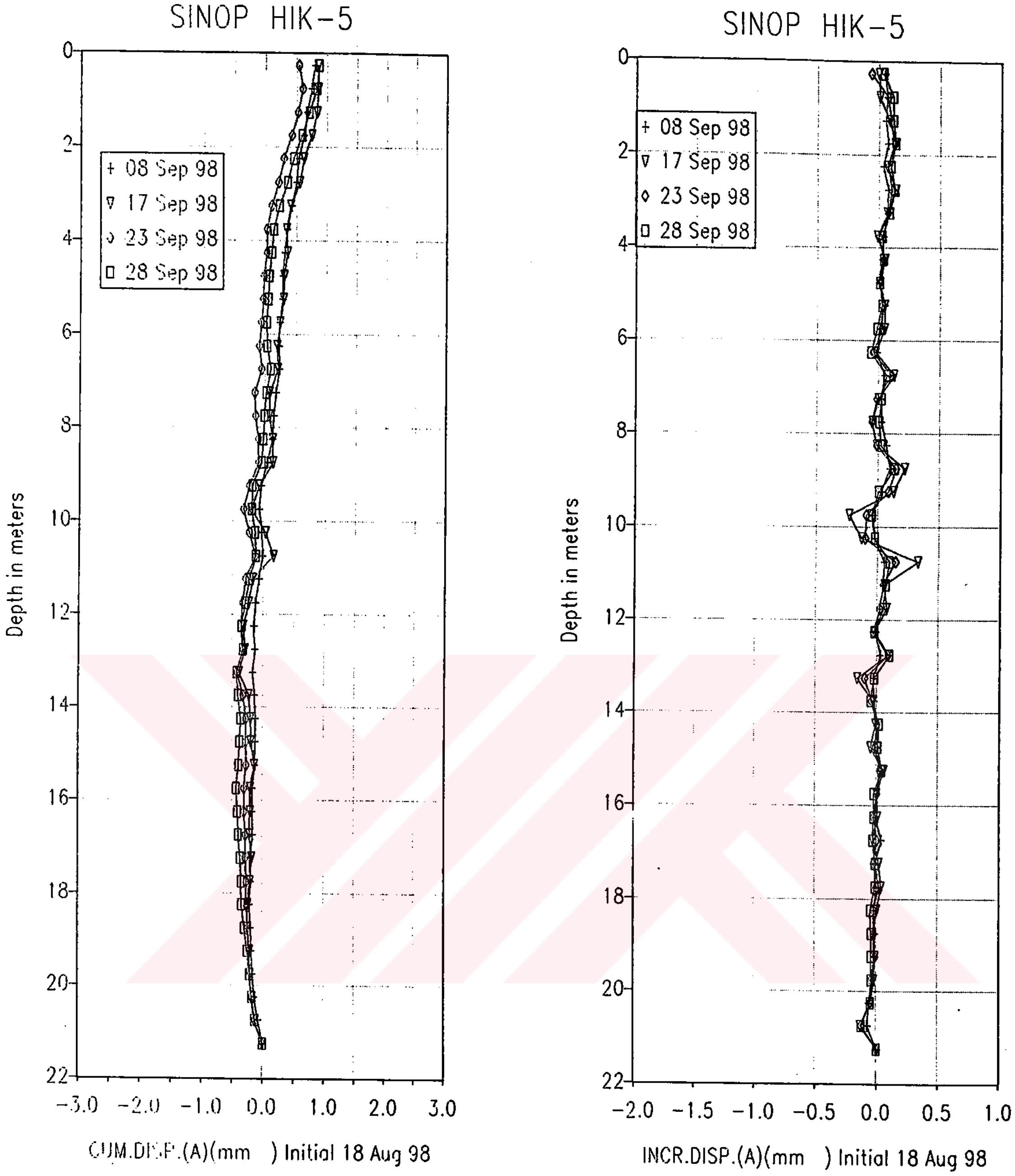
Figure A4. Inclinometer measurement graphs of the borehole Hik 4



YÜKSEL PROJE
Uluslararasi a.s.

SINOP YOLU

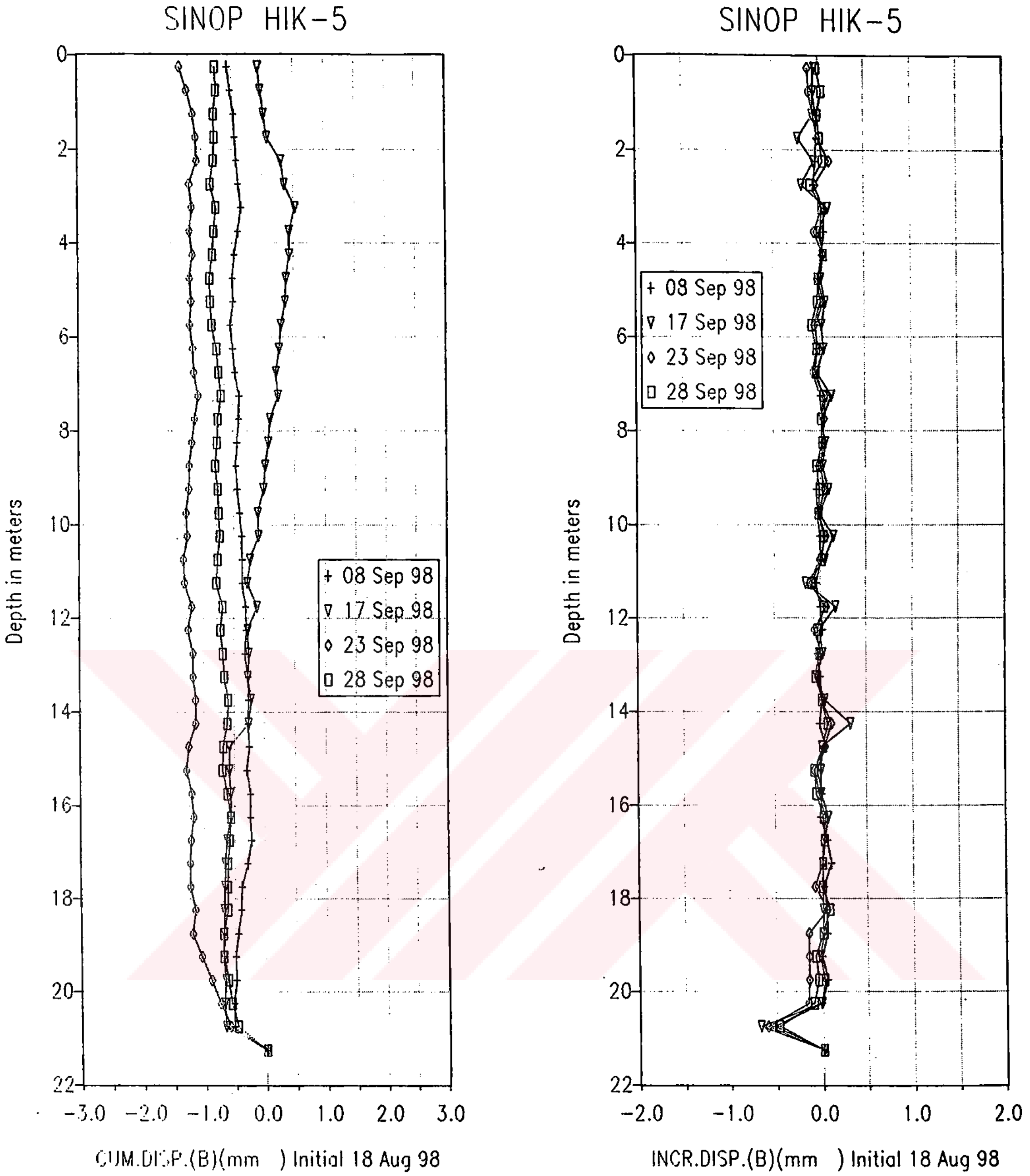
"Figure A4 (cont.)"



YÜKSEL PROJE
Uluslararasi a.s.

SINOP YOLU

Figure A5. Inclinometer measurement graphs of the borehole Hik 5



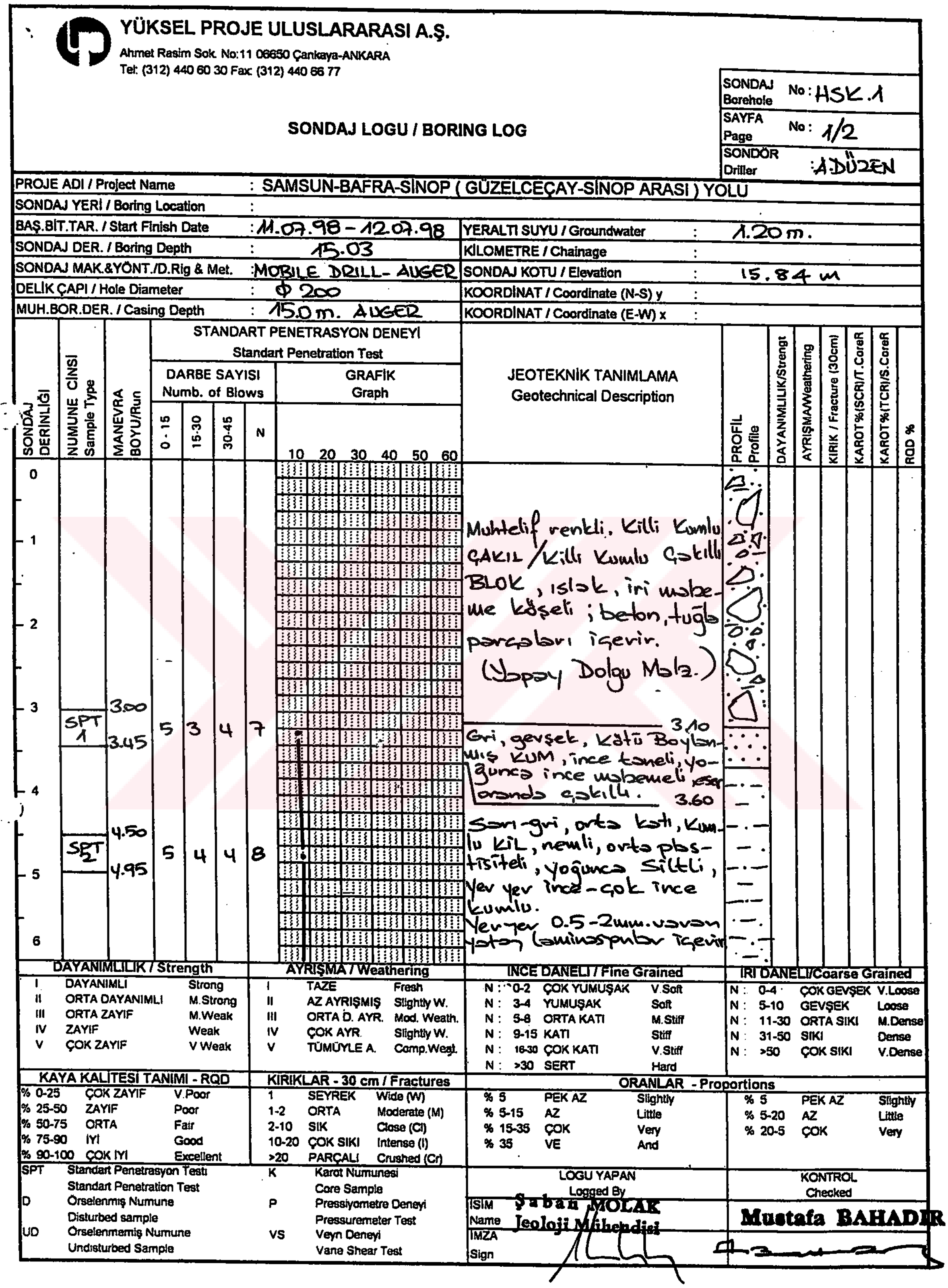
YÜKSEL PROJE
Uluslararasi a.s.

SINOP YOLU

"Figure A5 (cont.)"

APPENDIX B

COMPLETE BORING LOGS



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No : HSK.1
SAYFA Page No : 1/2
SONDÖR Driller : A.DÜZEN

PROJE ADI / Project Name	: SAMSUN-BAFRA-SİNOP (GÜZELCEÇAY-SİNOP ARASI) YOLU		
SONDAJ YERİ / Boring Location	:		
BAŞ.BİT.TAR. / Start Finish Date	: 11.07.98 - 12.07.98	YERALTI SUYU / Groundwater	: 1.20 m.
SONDAJ DER. / Boring Depth	: 15.03	KİLOMETRE / Chainage	:
SONDAJ MAK.&YÖNT./D.Rig & Met.	: MOBILE DRILL- AUGER	SONDAJ KOTU / Elevation	: 15.84 m
DELİK ÇAPI / Hole Diameter	: Φ 200	KOORDİNAT / Coordinate (N-S) y	:
MUH.BOR.DER. / Casing Depth	: 15.0 m. AUGER	KOORDİNAT / Coordinate (E-W) x	:

SONDAJ DERİNLİĞİ	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test — DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
0															
1								Muhtelif renkli, Killi Kumlu ÇAKIL / Killi Kumlu Çakıllı BLOK, ıslak, iri malzeme köşeli; beton, tuğla parçaları içerir. (Yapay Dolgu Malz.)							
2															
3	SPT 1	3.00 3.45	5	3	4	7		3.10 Gri, gevşek, kötü Boylanmış KUM, ince taneli, yoğunca ince malzemeli eser oranda çakıllı. 3.60							
4															
	SPT 2	4.50 4.95	5	4	4	8		Sarı-gri, orta katı, Kumlu KİL, nemli, orta plastisiteli, yoğunca Siltli, yer yer ince-çok ince kumlu. Yer yer 0.5-2mm. uzayan yatay laminasyonlar içerir.							
5															
6															

DAYANIMLILIK / Strength			AYRIŞMA / Weathering			INCE DANELI / Fine Grained			IRI DANELI/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N : 0-4	ÇOK GEVŞEK	V.Loose
II	ORTA DAYANIMLI	M.Strong	II	AZ AYRIŞMIŞ	Slightly W.	N : 3-4	YUMUŞAK	Soft	N : 5-10	GEVŞEK	Loose
III	ORTA ZAYIF	M.Weak	III	ORTA D. AYR.	Mod. Weath.	N : 5-8	ORTA KATI	M.Stiff	N : 11-30	ORTA SIKI	M.Dense
IV	ZAYIF	Weak	IV	ÇOK AYR.	Slightly W.	N : 9-15	KATI	Stiff	N : 31-50	SIKI	Dense
V	ÇOK ZAYIF	V Weak	V	TÜMÜYLE A.	Comp.Weat.	N : 16-30	ÇOK KATI	V.Stiff	N : >50	ÇOK SIKI	V.Dense
						N : >30	SERT	Hard			

KAYA KALITESI TANIMI - RQD			KIRIKLAR - 30 cm / Fractures			ORANLAR - Proportions					
% 0-25	ÇOK ZAYIF	V.Poor	1	SEYREK	Wide (W)	% 5	PEK AZ	Slightly	% 5	PEK AZ	Slightly
% 25-50	ZAYIF	Poor	1-2	ORTA	Moderate (M)	% 5-15	AZ	Little	% 5-20	AZ	Little
% 50-75	ORTA	Fair	2-10	SIK	Close (Cl)	% 15-35	ÇOK	Very	% 20-5	ÇOK	Very
% 75-90	IYI	Good	10-20	ÇOK SIKI	Intense (I)	% 35	VE	And			
% 90-100	ÇOK IYI	Excellent	>20	PARÇALI	Crushed (Cr)						

SPT Standart Penetrasyon Testi / Standart Penetration Test
D Örselenmiş Numune / Disturbed sample
UD Örselenmemiş Numune / Undisturbed Sample
K Karot Numunesi / Core Sample
P Pressiyometre Deneyi / Pressuremeter Test
VS Veyn Deneyi / Vane Shear Test

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR
İMZA Sign		

Figure B1. Boring log of Hsk 1

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Tel: (312) 440 60 30 Fax: (312) 440 66 77

SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HSK.1
SAYFA Page No: 2/2

STANDART PENETRASYON DENEYİ
Standart Penetration Test

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	DARBE SAYISI Numb. of Blows 0 - 15	15-30	30-45	N
6	SPT 3	6.00 6.45	5	3	5	8
7	SPT 4	7.00 7.45	10	6	11	17
8						
9	SPT 5	8.50 8.95	7	5	11	16
10	SPT 6	10.00 10.45	7	4	13	17
11	UD 1	11.00 11.50				
12	SPT 7	11.95	9	10	11	21
13	SPT 8	13.00 13.45	3	5	11	16
14						
15	SPT 9	15.00 15.03	50/3	-	-	R

JEOTEKNİK TANIMLAMA
Geotechnical Description

Kumlu KİL
Tanımı sayfa 1/2'dedir.
6.70

Gri - kahverengi, ortasıkı Çakıllı Kumlu KİL, nemli orta plastisiteli - plastisitesiz yer yer yoğunca Siltli; yoğunca ince-iri, sert kırılgan, köşeli - y. köşeli, iri malzeme içerir.
Tabana doğru gri renkli KİL yoğunluğu artıyor.
10.60

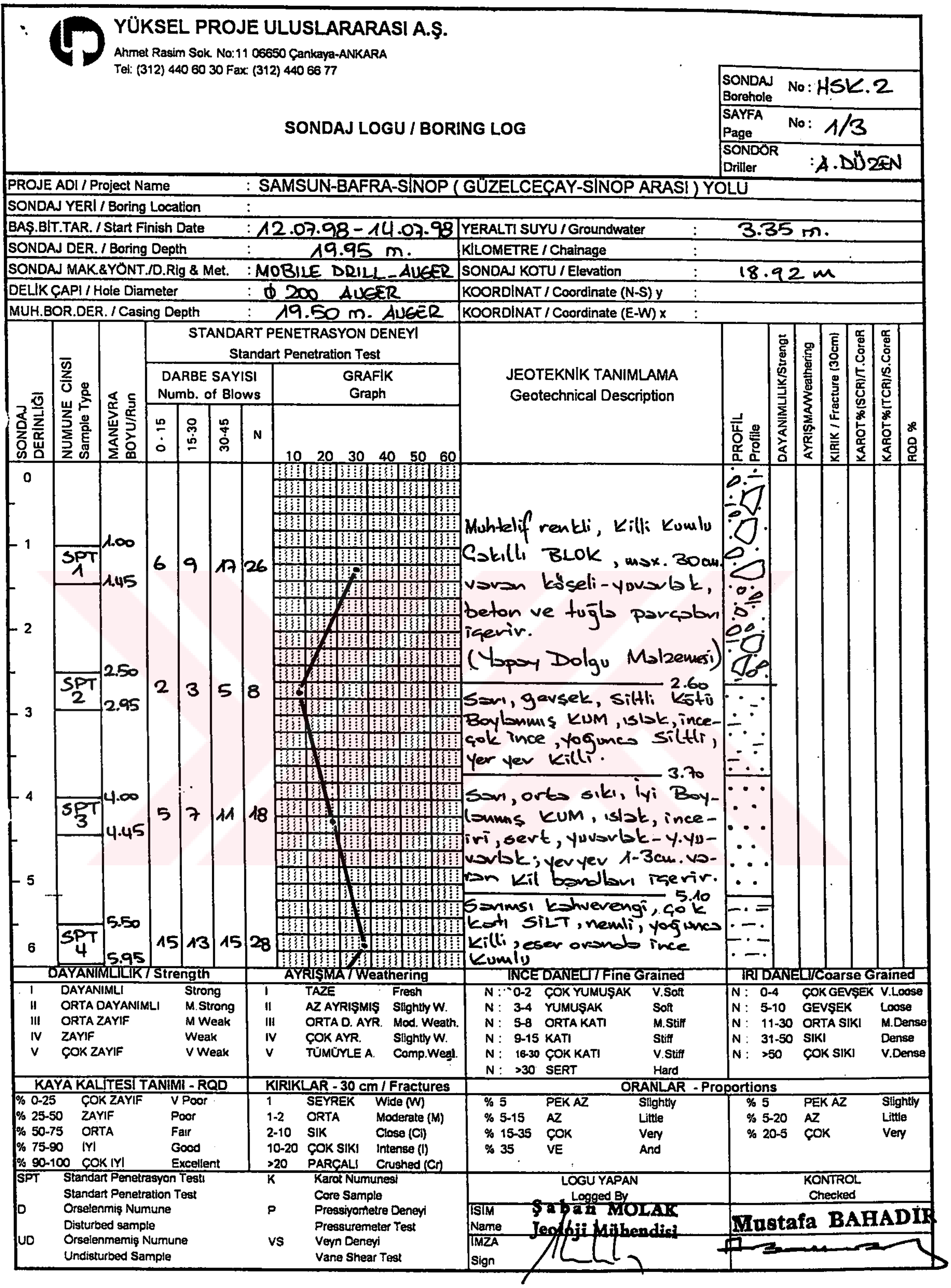
Yeşilimsi gri, çok katı, SİLT, nemli - ıslak; aşağıya doğru artan oranda yoğunca killi, az oranda ince kumlu.
14.50

KUYU SONU : 15.03 m.

PROFİL Profile | DAYANIMLILIK/Strengt | AYRIŞMA/Weathering | KIRIK / Fracture (30cm) | KAROT%(SCR)/T.CoreR | KAROT%(TCR)/S.CoreR | RQD %

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR
İMZA Sign		

"Figure B1 (cont.)"



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HSK.2
SAYFA Page No: 1/3
SONDÖR Driller : A.DÜZEN

PROJE ADI / Project Name : SAMSUN-BAFRA-SİNOP (GÜZELCEÇAY-SİNOP ARASI) YOLU
SONDAJ YERİ / Boring Location :
BAŞ.BİT.TAR. / Start Finish Date : 12.07.98 - 14.07.98
YERALTI SUYU / Groundwater : 3.35 m.
SONDAJ DER. / Boring Depth : 19.95 m.
KİLOMETRE / Chainage :
SONDAJ MAK.&YÖNT./D.Rig & Met. : MOBILE DRILL - AUGER
SONDAJ KOTU / Elevation : 18.92 m
DELİK ÇAPI / Hole Diameter : ϕ 200 AUGER
KOORDİNAT / Coordinate (N-S) y :
MUH.BOR.DER. / Casing Depth : 19.50 m. AUGER
KOORDİNAT / Coordinate (E-W) x :

SONDAJ DERİNLİĞİ	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	0-15	15-30	30-45	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
0															
1	SPT 1	1.00 1.45	6	9	17	26		Muhtelif renkli, Killi Kumlu Çakıllı BLOK, max. 30cm. varan köşeli-yuvarlak, beton ve tuğla parçaları içerir. (Yapay Dolgu Malzemesi)							
2								2.60							
3	SPT 2	2.50 2.95	2	3	5	8		Sarı, gevşek, siltli Kötü Boylanmış KUM, ıslak, ince-çok ince, yoğunca siltli, yer yer killi.							
4	SPT 3	4.00 4.45	5	7	11	18		3.70 Sarı, orta sıkı, İyi Boylanmış KUM, ıslak, ince-iri, sert, yuvarlak-y.yuvarlak; yer yer 1-3cm. varan Kil bandları içerir.							
5								5.10							
6	SPT 4	5.50 5.95	15	13	15	28		Sarımsı kahverengi, çok katı SİLT, nemli, yoğunca killi, eser oranda ince kumlu							

DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE DANELİ / Fine Grained			İRİ DANELİ/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N : 0-4	ÇOK GEVŞEK	V.Loose
II	ORTA DAYANIMLI	M.Strong	II	AZ AYRIŞMIŞ	Slightly W.	N : 3-4	YUMUŞAK	Soft	N : 5-10	GEVŞEK	Loose
III	ORTA ZAYIF	M Weak	III	ORTA D. AYR.	Mod. Weath.	N : 5-8	ORTA KATI	M.Stiff	N : 11-30	ORTA SIKI	M.Dense
IV	ZAYIF	Weak	IV	ÇOK AYR.	Slightly W.	N : 9-15	KATI	Stiff	N : 31-50	SIKI	Dense
V	ÇOK ZAYIF	V Weak	V	TÜMÜYLE A.	Comp.Weal.	N : 16-30	ÇOK KATI	V.Stiff	N : >50	ÇOK SIKI	V.Dense
						N : >30	SERT	Hard			

KAYA KALİTESİ TANIMI - RQD			KIRIKLAR - 30 cm / Fractures			ORANLAR - Proportions					
% 0-25	ÇOK ZAYIF	V Poor	1	SEYREK	Wide (W)	% 5	PEK AZ	Slightly	% 5	PEK AZ	Slightly
% 25-50	ZAYIF	Poor	1-2	ORTA	Moderate (M)	% 5-15	AZ	Little	% 5-20	AZ	Little
% 50-75	ORTA	Fair	2-10	SIK	Close (Cl)	% 15-35	ÇOK	Very	% 20-5	ÇOK	Very
% 75-90	İYİ	Good	10-20	ÇOK SIKI	Intense (I)	% 35	VE	And			
% 90-100	ÇOK İYİ	Excellent	>20	PARÇALI	Crushed (Cr)						

SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample	LOGU YAPAN Logged By	KONTROL Checked
D	Örselenmiş Numune Disturbed sample	P	Pressiyometre Deneyi Pressuremeter Test	İSİM Name: Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR
UD	Örselenmemiş Numune Undisturbed Sample	VS	Veyn Deneyi Vane Shear Test	İMZA Sign	

Figure B2. Boring log of Hsk 2

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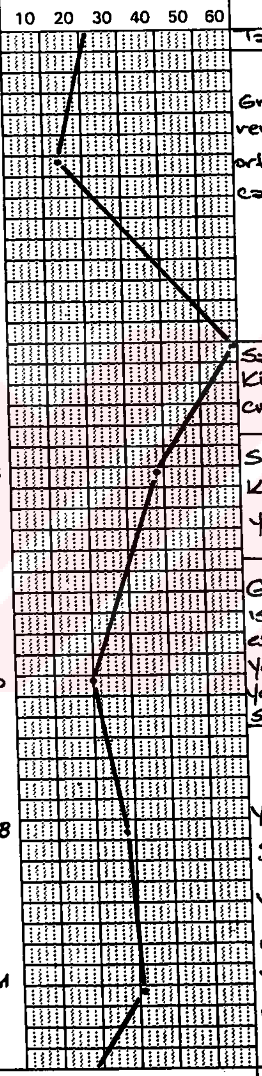
SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HSK.2
SAYFA Page No: 2/3

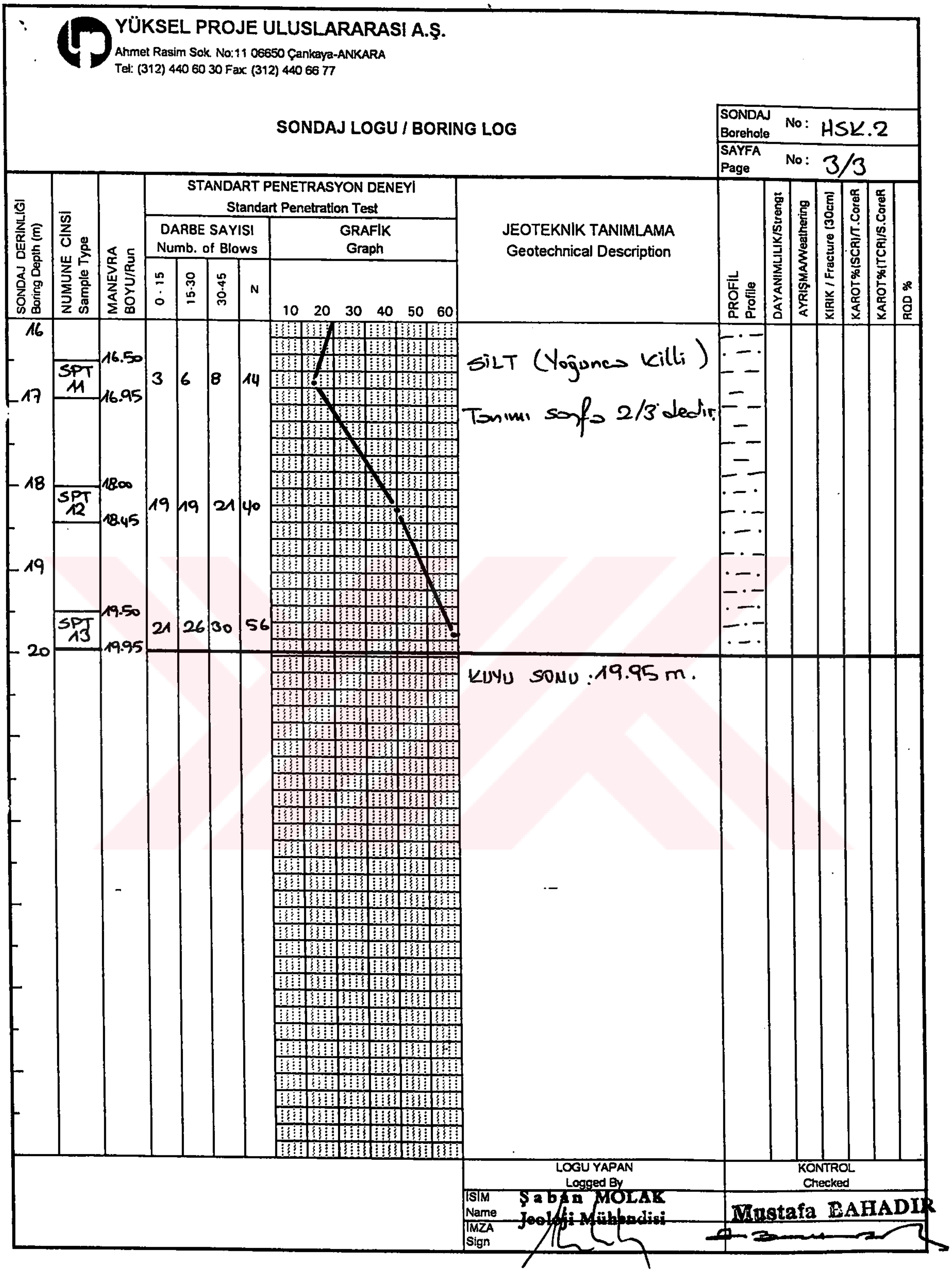
STANDART PENETRASYON DENEYİ / Standart Penetration Test — GRAFİK Graph (N: 10 20 30 40 50 60)



SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
6							Tanımı sayfa 1/3'dedir. 6.20							
7	SPT 5	7.00 7.45	10	8	6	14	Gri-yer yer sarımsı kahverengi, katı, KİL, nemli, orta-az plastisiteli, yoğunca Siltli.							
8	UD 1	8.50 9.00					9.00							
9	SPT.6	9.00 9.20	48	50/5	–	R	Sarımsı kahverengi, çok sıkı, Killi Kumlu ÇAKIL, max. 3-7 cm. uzun, sert, köşeli. 9.90							
10	SPT 7	10.00 10.45	15	19	19	38	Sarımsı kahverengi, sert, KİL, nemli, orta plastisiteli, yoğunca Siltli 11.10							
11	UD 2	11.50 12.00					Gri, katı, KİL, nemli-ıslak, orta-y. plastisiteli, eser oranda iri malzemeli. Yer yer yoğunca yatay foliasyonlu ve yer yer karbonat sıvalı. 12.70							
12	SPT 8	12.00 12.45	12	10	10	20								
13	SPT 9	13.50 13.95	12	12	16	28	Yeşilimsi gri, çok katı-sert, Silt, nemli, yoğunca Killi. Yer yer birime Kil hakim oluyor. Yer yer dağılgan killeşmiş çakıllar içerir.							
14														
15	SPT 10	15.00 15.45	12	13	18	31								
16														

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR
İMZA Sign		

"Figure B2 (cont.)"



YÜKSEL PROJE ULUSLARARASI A.Ş.
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Tel: (312) 440 60 30 Fax: (312) 440 66 77

SONDAJ LOGU / BORING LOG

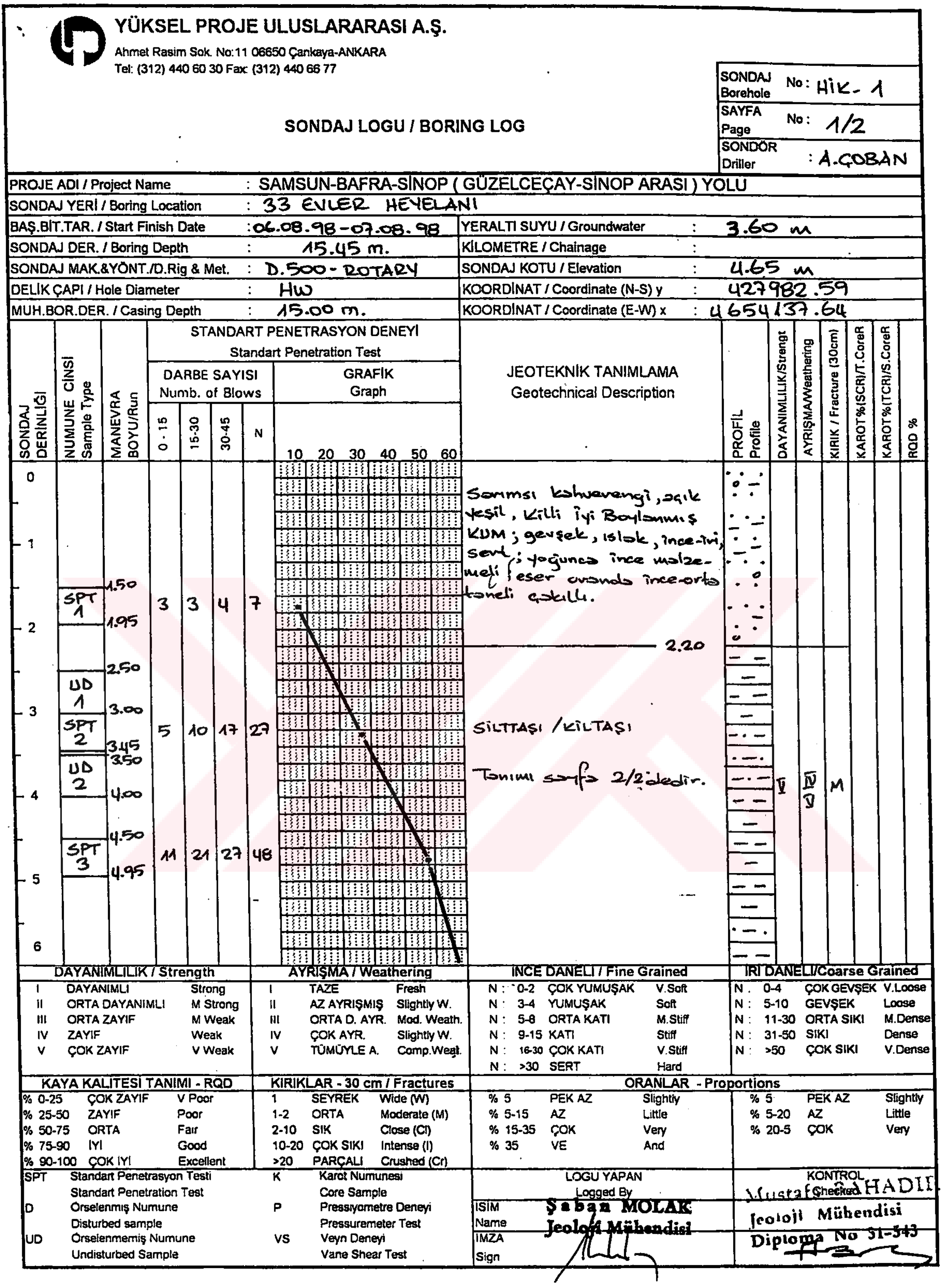
SONDAJ Borehole No : HSK.2
SAYFA Page No : 3/3

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test — DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
16															
	SPT 11	16.50 16.95	3	6	8	14		SİLT (Yoğunca killi) Tanımı sayfa 2/3'dedir.							
17															
18	SPT 12	18.00 18.45	19	19	21	40									
19															
	SPT 13	19.50 19.95	21	26	30	56									
20								KUYU SONU : 19.95 m.							

LOGU YAPAN Logged By
İSİM Name: Şaban MOLAK Jeoloji Mühendisi
İMZA Sign

KONTROL Checked
Mustafa BAHADIR

"Figure B2 (cont.)"



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HİK-1
SAYFA Page No: 1/2
SONDÖR Driller: A.ÇOBAN

PROJE ADI / Project Name	: SAMSUN-BAFRA-SİNOP (GÜZELCEÇAY-SİNOP ARASI) YOLU		
SONDAJ YERİ / Boring Location	: 33 EVLER HEYELANI		
BAŞ.BİT.TAR. / Start Finish Date	: 06.08.98-07.08.98	YERALTI SUYU / Groundwater	: 3.60 m
SONDAJ DER. / Boring Depth	: 15.45 m.	KİLOMETRE / Chainage	:
SONDAJ MAK.&YÖNT./D.Rig & Met.	: D.500-ROTARY	SONDAJ KOTU / Elevation	: 4.65 m
DELİK ÇAPI / Hole Diameter	: HW	KOORDİNAT / Coordinate (N-S) y	: 427982.59
MUH.BOR.DER. / Casing Depth	: 15.00 m.	KOORDİNAT / Coordinate (E-W) x	: 4654137.64

SONDAJ DERİNLİĞİ	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	JEOTEKNİK TANIMLAMA Geotechnical Description	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
0							Sarımsı kahverengi, açık yeşil, Killi İyi Boylanmış KUM; gevşek, ıslak, ince-iri, sert; yoğunca ince malzemeli, eser oranda ince-orta taneli çakıllı.						
1													
	SPT 1	1.50 1.95	3	3	4	7							
2							2.20						
	UD 1	2.50 3.00											
3	SPT 2	3.00 3.45	5	10	17	27	SİLTTAŞI / KİLTAŞI						
	UD 2	3.50 4.00					Tanımı sayfa 2/2'dedir.	V	IV V	M			
4													
	SPT 3	4.50 4.95	11	21	27	48							
5													
6													

DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE DANELİ / Fine Grained			İRİ DANELİ/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N : 0-4	ÇOK GEVŞEK	V.Loose
II	ORTA DAYANIMLI	M Strong	II	AZ AYRIŞMIŞ	Slightly W.	N : 3-4	YUMUŞAK	Soft	N : 5-10	GEVŞEK	Loose
III	ORTA ZAYIF	M Weak	III	ORTA D. AYR.	Mod. Weath.	N : 5-8	ORTA KATI	M.Stiff	N : 11-30	ORTA SIKI	M.Dense
IV	ZAYIF	Weak	IV	ÇOK AYR.	Slightly W.	N : 9-15	KATI	Stiff	N : 31-50	SIKI	Dense
V	ÇOK ZAYIF	V Weak	V	TÜMÜYLE A.	Comp.Weat.	N : 16-30	ÇOK KATI	V.Stiff	N : >50	ÇOK SIKI	V.Dense
						N : >30	SERT	Hard			

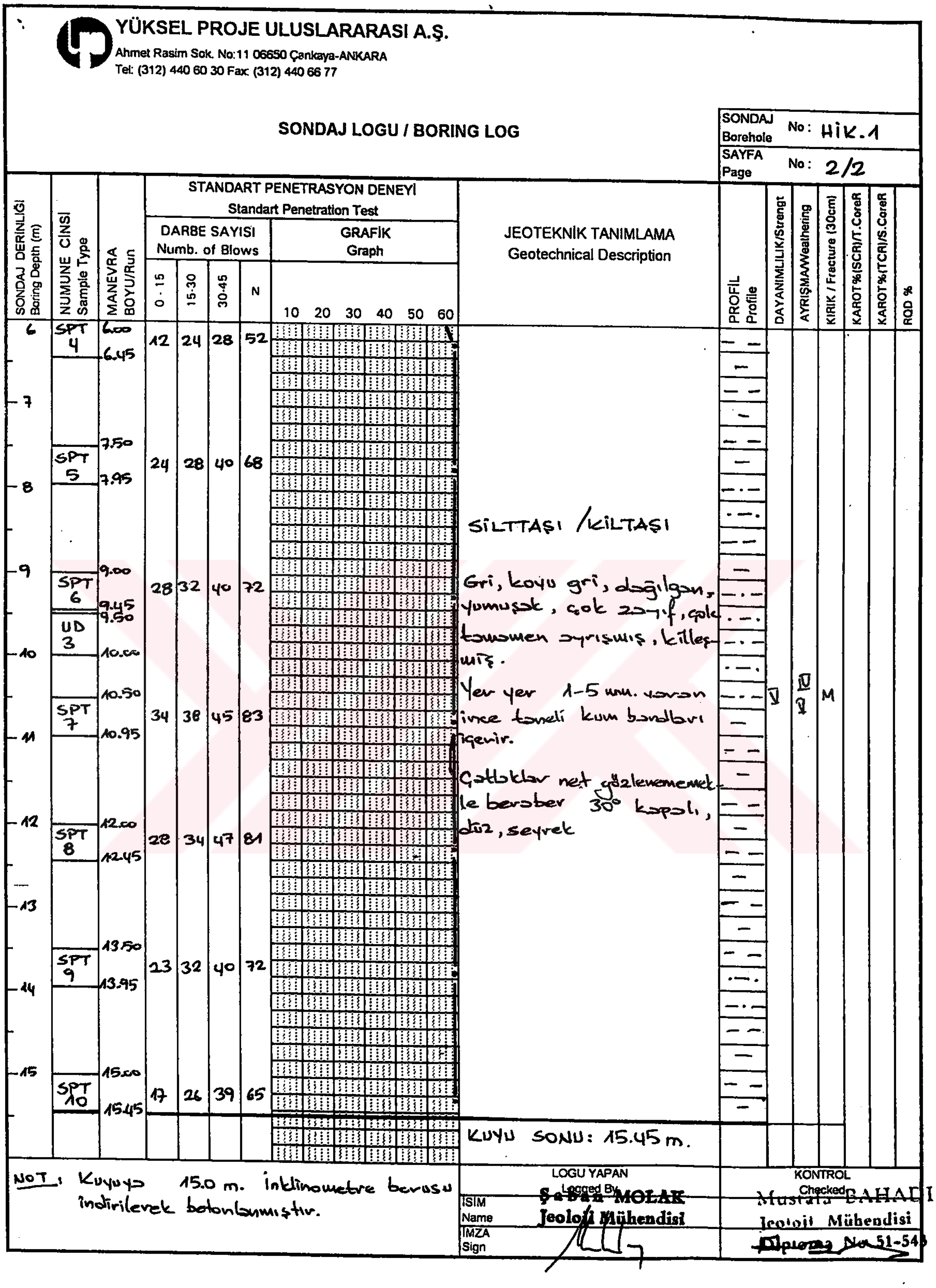
KAYA KALİTESİ TANIMI - RQD			KIRIKLAR - 30 cm / Fractures			ORANLAR - Proportions					
% 0-25	ÇOK ZAYIF	V Poor	1	SEYREK	Wide (W)	% 5	PEK AZ	Slightly	% 5	PEK AZ	Slightly
% 25-50	ZAYIF	Poor	1-2	ORTA	Moderate (M)	% 5-15	AZ	Little	% 5-20	AZ	Little
% 50-75	ORTA	Fair	2-10	SIK	Close (Cl)	% 15-35	ÇOK	Very	% 20-5	ÇOK	Very
% 75-90	İYİ	Good	10-20	ÇOK SIKI	Intense (I)	% 35	VE	And			
% 90-100	ÇOK İYİ	Excellent	>20	PARÇALI	Crushed (Cr)						

SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample
D	Örselenmiş Numune Disturbed sample	P	Pressıyometre Deneyi Pressuremeter Test
UD	Örselenmemiş Numune Undisturbed Sample	VS	Veyn Deneyi Vane Shear Test

LOGU YAPAN
Logged By
İSİM Name: Şaban MOLAK
Jeoloji Mühendisi
İMZA Sign

KONTROL
Checked
Mustafa ŞAHADIL
Jeoloji Mühendisi
Diploma No 31-343

Figure B3. Boring log of Hik 1



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SONDAJ LOGU / BORING LOG

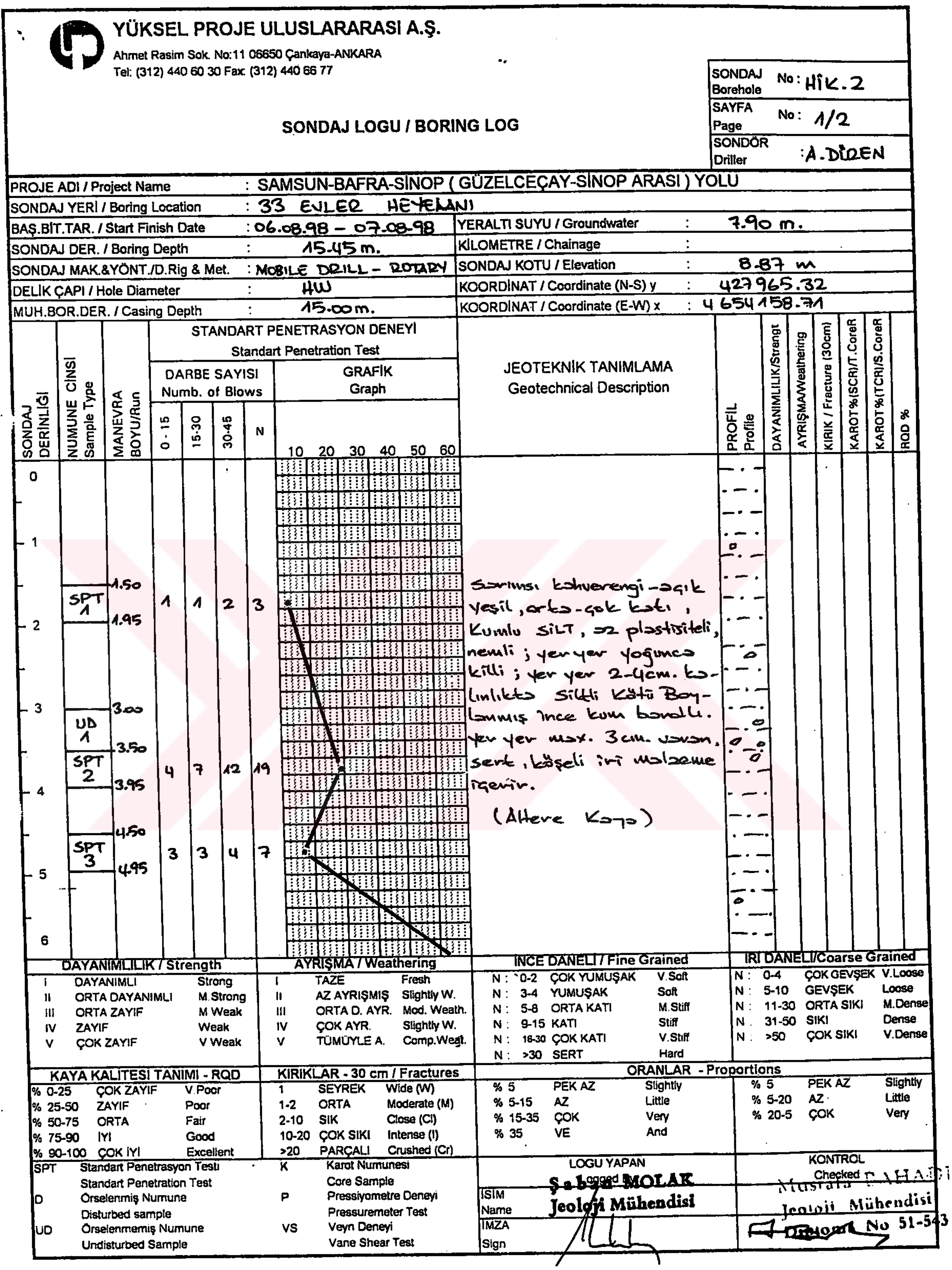
SONDAJ Borehole No: HİK.1
SAYFA Page No: 2/2

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test: DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT% (SCR)/T.CoreR	KAROT% (TCR)/S.CoreR	RQD %
6	SPT 4	6.00 6.45	12	24	28	52									
7															
	SPT 5	7.50 7.95	24	28	40	68									
8								SİLTTAŞI /KİLTAŞI							
9	SPT 6	9.00 9.45	28	32	40	72		Gri, koyu gri, dağılgan, yumuşak, çok zayıf, çok tamamen ayrışmış, killeşmiş.							
	UD 3	9.50 10.00													
10	SPT 7	10.50 10.95	34	38	45	83		Yer yer 1-5 mm. varan ince taneli kum bandları içerir.		V	IV V	M			
11								Çatlaklar net gözlenememekle beraber 30° kapalı, düz, seyrek							
12	SPT 8	12.00 12.45	28	34	47	81									
13															
	SPT 9	13.50 13.95	23	32	40	72									
14															
15	SPT 10	15.00 15.45	17	26	39	65									
								KUYU SONU: 15.45 m.							

NOT: Kuyuya 15.0 m. İnklinometre borusu indirilerek betonlanmıştır.

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR Jeoloji Mühendisi
İMZA Sign		Diploma No 51-54

"Figure B3 (cont.)"



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole	No: HİK.2
SAYFA Page	No: 1/2
SONDÖR Driller	: A.DİREN

PROJE ADI / Project Name : SAMSUN-BAFRA-SİNOP (GÜZELCEÇAY-SİNOP ARASI) YOLU
SONDAJ YERİ / Boring Location : 33 EVLER HEYELANI

BAŞ.BİT.TAR. / Start Finish Date	: 06.08.98 – 07.08.98	YERALTI SUYU / Groundwater	: 7.90 m.
SONDAJ DER. / Boring Depth	: 15.45 m.	KİLOMETRE / Chainage	:
SONDAJ MAK.&YÖNT./D.Rig & Met.	: MOBILE DRILL – ROTARY	SONDAJ KOTU / Elevation	: 8.87 m
DELİK ÇAPI / Hole Diameter	: HW	KOORDİNAT / Coordinate (N-S) y	: 427965.32
MUH.BOR.DER. / Casing Depth	: 15.00 m.	KOORDİNAT / Coordinate (E-W) x	: 4 654158.71

SONDAJ DERİNLİĞİ	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	0-15	15-30	30-45	N
1.50–1.95	SPT 1		1	1	2	3
3.00–3.50	UD 1					
3.50–3.95	SPT 2		4	7	12	19
4.50–4.95	SPT 3		3	3	4	7

STANDART PENETRASYON DENEYİ / Standart Penetration Test — DARBE SAYISI / Numb. of Blows — GRAFİK / Graph (10 20 30 40 50 60)

JEOTEKNİK TANIMLAMA / Geotechnical Description

Sarımsı kahverengi-açık yeşil, orta-çok katı, Kumlu SİLT, az plastisiteli, nemli; yer yer yoğunca killi; yer yer 2-4cm. kalınlıkta siltli kötü boylanmış ince kum bantlı. Yer yer max. 3cm. uzayan, sert, köşeli iri malzeme içerir.

(Altere Kaya)

PROFİL Profile | DAYANIMLILIK/Strengt | AYRIŞMA/Weathering | KIRIK / Fracture (30cm) | KAROT%(SCR)/T.CoreR | KAROT%(TCR)/S.CoreR | RQD %

DAYANIMLILIK / Strength		
I	DAYANIMLI	Strong
II	ORTA DAYANIMLI	M.Strong
III	ORTA ZAYIF	M Weak
IV	ZAYIF	Weak
V	ÇOK ZAYIF	V Weak

AYRIŞMA / Weathering		
I	TAZE	Fresh
II	AZ AYRIŞMIŞ	Slightly W.
III	ORTA D. AYR.	Mod. Weath.
IV	ÇOK AYR.	Slightly W.
V	TÜMÜYLE A.	Comp.Weat.

İNCE DANELİ / Fine Grained		
N : 0-2	ÇOK YUMUŞAK	V.Soft
N : 3-4	YUMUŞAK	Soft
N : 5-8	ORTA KATI	M.Stiff
N : 9-15	KATI	Stiff
N : 16-30	ÇOK KATI	V.Stiff
N : >30	SERT	Hard

IRI DANELİ/Coarse Grained		
N : 0-4	ÇOK GEVŞEK	V.Loose
N : 5-10	GEVŞEK	Loose
N : 11-30	ORTA SIKI	M.Dense
N : 31-50	SIKI	Dense
N : >50	ÇOK SIKI	V.Dense

KAYA KALİTESİ TANIMI - RQD		
% 0-25	ÇOK ZAYIF	V.Poor
% 25-50	ZAYIF	Poor
% 50-75	ORTA	Fair
% 75-90	İYİ	Good
% 90-100	ÇOK İYİ	Excellent

KIRIKLAR - 30 cm / Fractures		
1	SEYREK	Wide (W)
1-2	ORTA	Moderate (M)
2-10	SIK	Close (Cl)
10-20	ÇOK SIKI	Intense (I)
>20	PARÇALI	Crushed (Cr)

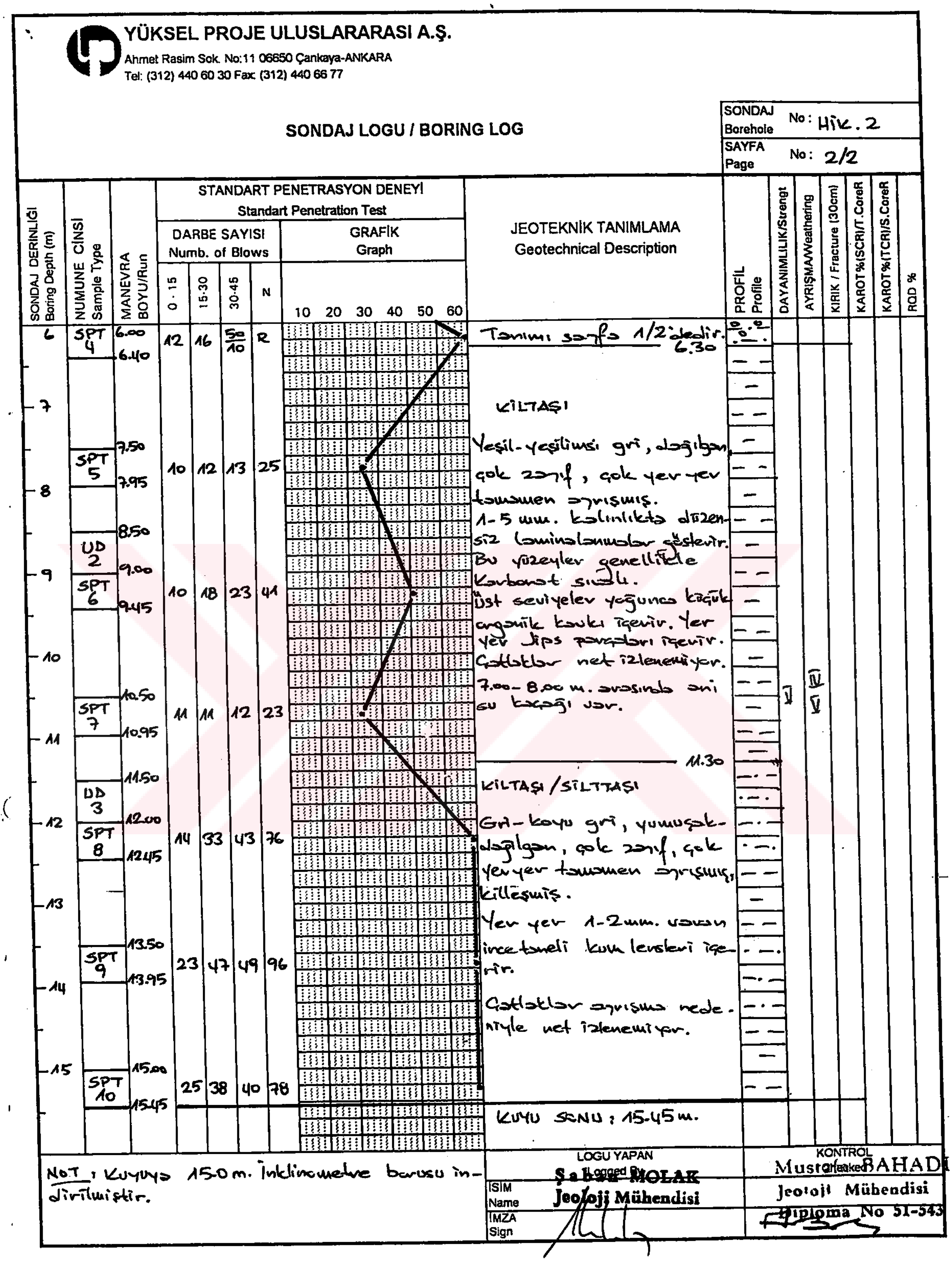
ORANLAR - Proportions

% 5	PEK AZ	Slightly	% 5	PEK AZ	Slightly
% 5-15	AZ	Little	% 5-20	AZ	Little
% 15-35	ÇOK	Very	% 20-5	ÇOK	Very
% 35	VE	And			

SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample
D	Örselenmiş Numune Disturbed sample	P	Pressiyometre Deneyi Pressuremeter Test
UD	Örselenmemiş Numune Undisturbed Sample	VS	Veyn Deneyi Vane Shear Test

	LOGU YAPAN Logged	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa Jeoloji Mühendisi No 51-543
İMZA Sign		

Figure B4. Boring log of Hik 2



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: Hik. 2
SAYFA Page No: 2/2

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N
6	SPT 4	6.00 6.40	12	16	50/10	R
7						
	SPT 5	7.50 7.95	10	12	13	25
8						
	UD 2	8.50 9.00				
9	SPT 6	9.00 9.45	10	18	23	41
10						
	SPT 7	10.50 10.95	11	11	12	23
11						
	UD 3	11.50 12.00				
12	SPT 8	12.00 12.45	14	33	43	76
13						
	SPT 9	13.50 13.95	23	47	49	96
14						
15	SPT 10	15.00 15.45	25	38	40	78

JEOTEKNİK TANIMLAMA
Geotechnical Description

Tanımı sayfa 1/2'dedir.
6.30

KİLTAŞI

Yeşil-yeşilimsi gri, dağılgan, çok zayıf, çok yer yer tamamen ayrışmış. 1-5 mm. kalınlıkta düzensiz laminalanmalar gösterir. Bu yüzeyler genellikle karbonat sıvalı. Üst seviyeler yoğunca küçük organik kavkı içerir. Yer yer jips parçaları içerir. Çatlaklar net izlenemiyor. 7.00-8.00 m. arasında ani su kaçağı var.

11.30

KİLTAŞI / SİLTTAŞI

Gri-koyu gri, yumuşak-dağılgan, çok zayıf, çok yer yer tamamen ayrışmış, killeşmiş.
Yer yer 1-2 mm. varan ince taneli kum lensleri içerir.

Çatlaklar ayrışma nedeniyle net izlenemiyor.

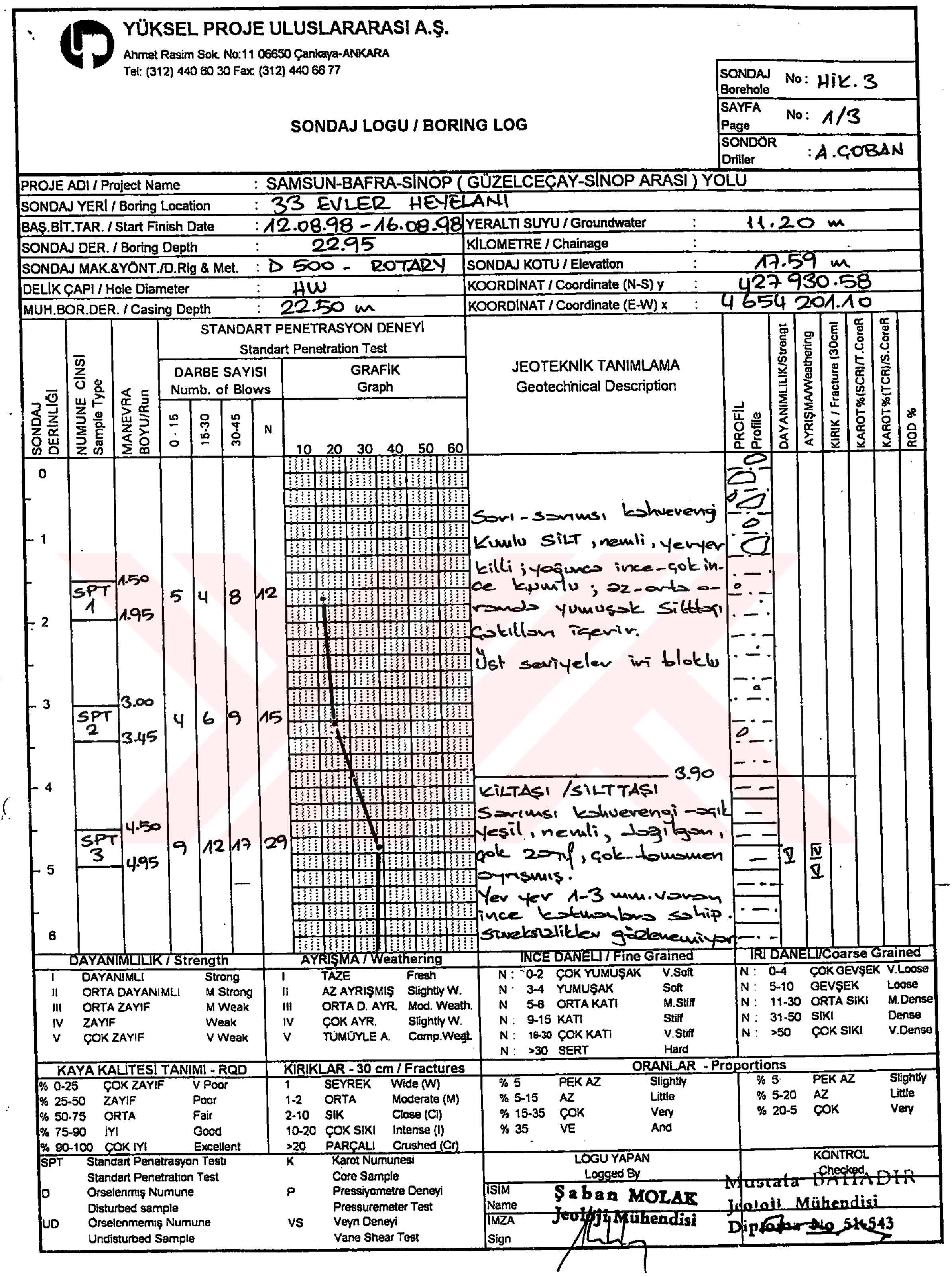
KUYU SONU: 15.45 m.

NOT: Kuyuya 15.0 m. İnklinometre borusu indirilmiştir.

LOGU YAPAN Logged By
İSİM Name: Şaban MOLAK Jeoloji Mühendisi
İMZA Sign

KONTROL Checked
Mustafa BAHADIR
Jeoloji Mühendisi
Diploma No 51-543

"Figure B4 (cont.)"



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole	No: HİK. 3
SAYFA Page	No: 1/3
SONDÖR Driller	: A.ÇOBAN

PROJE ADI / Project Name	: SAMSUN-BAFRA-SİNOP (GÜZELCEÇAY-SİNOP ARASI) YOLU		
SONDAJ YERİ / Boring Location	: 33 EVLER HEYELANI		
BAŞ.BİT.TAR. / Start Finish Date	: 12.08.98 - 16.08.98	YERALTI SUYU / Groundwater	: 11.20 m
SONDAJ DER. / Boring Depth	: 22.95	KİLOMETRE / Chainage	:
SONDAJ MAK.&YÖNT./D.Rig & Met.	: D 500 - ROTARY	SONDAJ KOTU / Elevation	: 17.59 m
DELİK ÇAPI / Hole Diameter	: HW	KOORDİNAT / Coordinate (N-S) y	: 427 930.58
MUH.BOR.DER. / Casing Depth	: 22.50 m	KOORDİNAT / Coordinate (E-W) x	: 4 654 201.10

SONDAJ DERİNLİĞİ	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test — DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
0								Sarı-sarımsı kahverengi Kumlu SİLT, nemli, yer yer killi; yoğunca ince-çok ince kumlu; az-orta oranda yumuşak silttaşı çakılları içerir. Üst seviyeler iri bloklu							
1															
2	SPT 1	1.50 1.95	5	4	8	12									
3	SPT 2	3.00 3.45	4	6	9	15									
4								3.90 KİLTAŞI /SİLTTAŞI Sarımsı kahverengi-açık yeşil, nemli, dağılgan, çok zayıf, çok-tamamen ayrışmış. Yer yer 1-3 mm. varan ince katmanlara sahip. Süreksizlikler gözlenemiyor							
5	SPT 3	4.50 4.95	9	12	17	29				V	IV V				
6															

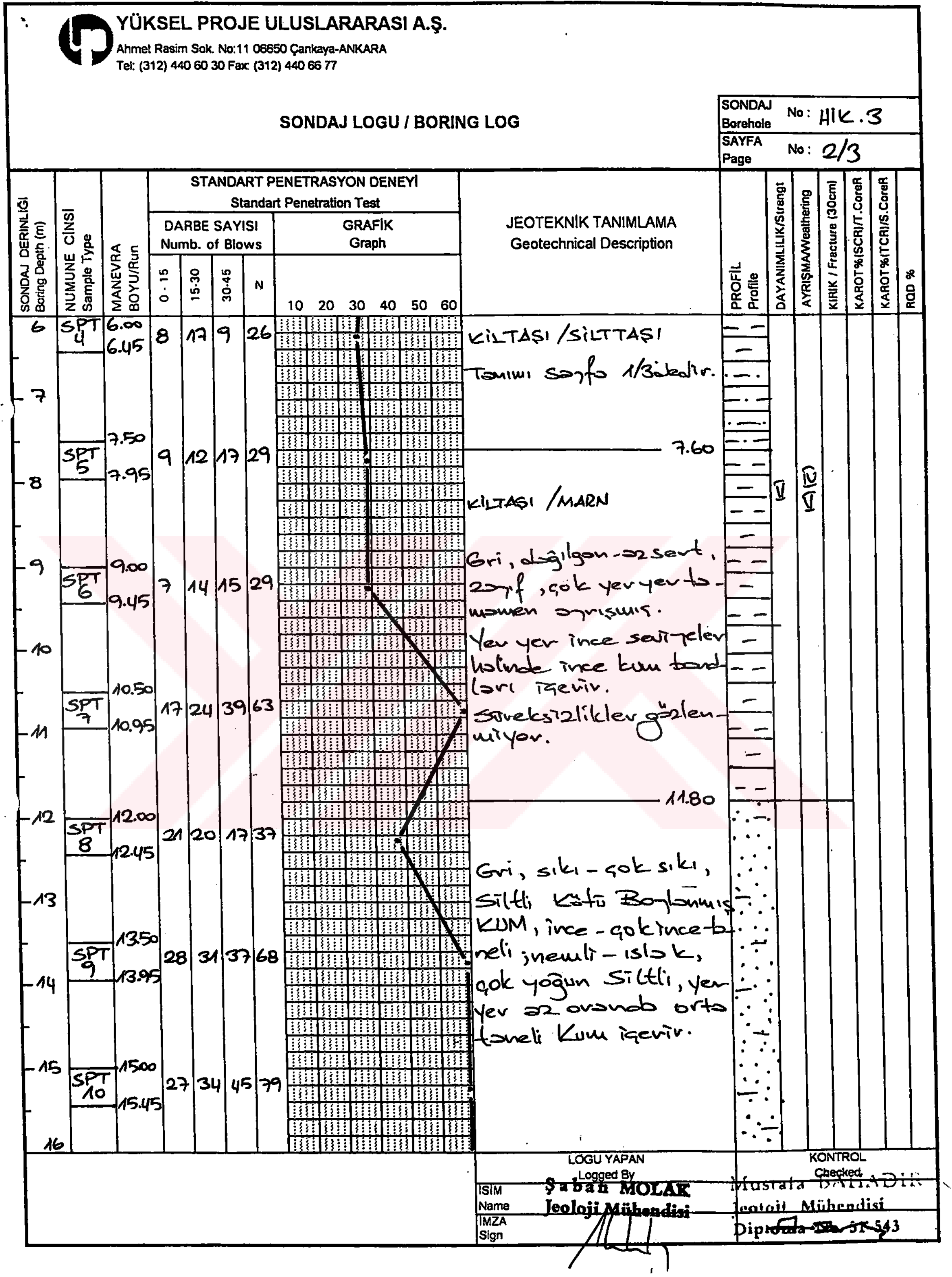
DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE DANELİ / Fine Grained			İRİ DANELİ/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N : 0-4	ÇOK GEVŞEK	V.Loose
II	ORTA DAYANIMLI	M.Strong	II	AZ AYRIŞMIŞ	Slightly W.	N : 3-4	YUMUŞAK	Soft	N : 5-10	GEVŞEK	Loose
III	ORTA ZAYIF	M Weak	III	ORTA D. AYR.	Mod. Weath.	N 5-8	ORTA KATI	M.Stiff	N : 11-30	ORTA SIKI	M.Dense
IV	ZAYIF	Weak	IV	ÇOK AYR.	Slightly W.	N : 9-15	KATI	Stiff	N : 31-50	SIKI	Dense
V	ÇOK ZAYIF	V Weak	V	TÜMÜYLE A.	Comp.Weat.	N : 16-30	ÇOK KATI	V.Stiff	N : >50	ÇOK SIKI	V.Dense
						N : >30	SERT	Hard			

KAYA KALİTESİ TANIMI - RQD			KIRIKLAR - 30 cm / Fractures			ORANLAR - Proportions					
% 0-25	ÇOK ZAYIF	V Poor	1	SEYREK	Wide (W)	% 5	PEK AZ	Slightly	% 5	PEK AZ	Slightly
% 25-50	ZAYIF	Poor	1-2	ORTA	Moderate (M)	% 5-15	AZ	Little	% 5-20	AZ	Little
% 50-75	ORTA	Fair	2-10	SIK	Close (Cl)	% 15-35	ÇOK	Very	% 20-5	ÇOK	Very
% 75-90	İYİ	Good	10-20	ÇOK SIKI	Intense (I)	% 35	VE	And			
% 90-100	ÇOK İYİ	Excellent	>20	PARÇALI	Crushed (Cr)						

SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample
D	Örselenmiş Numune Disturbed sample	P	Pressiyometre Deneyi Pressuremeter Test
UD	Örselenmemiş Numune Undisturbed Sample	VS	Veyn Deneyi Vane Shear Test

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR Jeoloji Mühendisi Diploma No 51543
İMZA Sign		

Figure B5. Boring log of Hik 3



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HİK.3
SAYFA Page No: 2/3

STANDART PENETRASYON DENEYİ / Standart Penetration Test

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N
6	SPT 4	6.00 6.45	8	17	9	26
7						
	SPT 5	7.50 7.95	9	12	17	29
8						
9	SPT 6	9.00 9.45	7	14	15	29
10						
	SPT 7	10.50 10.95	17	24	39	63
11						
12	SPT 8	12.00 12.45	21	20	17	37
13						
	SPT 9	13.50 13.95	28	31	37	68
14						
15	SPT 10	15.00 15.45	27	34	45	79
16						

JEOTEKNİK TANIMLAMA / Geotechnical Description

KİLTAŞI / SİLTTAŞI

Tanımı sayfa 1/3'tedir.

7.60

KİLTAŞI / MARN

Gri, dağılgan-az sert, zayıf, çok yer yer tamamen ayrışmış. Yer yer ince seviyeler halinde ince kum bantları içerir. Süreksizlikler gözlenmiyor.

11.80

Gri, sıkı-çok sıkı, Siltli Kötü Boylanmış KUM, ince-çok ince taneli, nemli-ıslak, çok yoğun Siltli, yer yer az oranda orta taneli Kum içerir.

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR Jeoloji Mühendisi
İMZA Sign		Diploma No: 543

"Figure B5 (cont.)"

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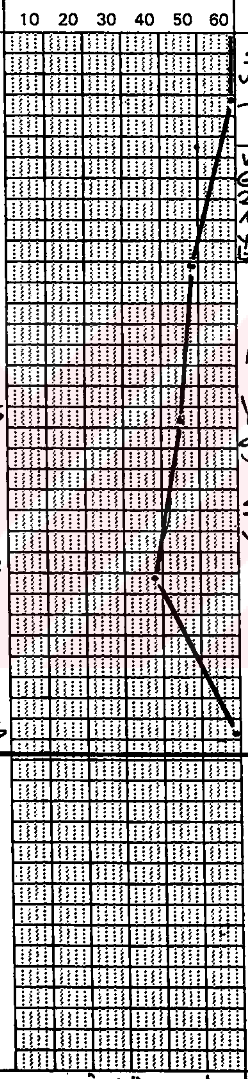
SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HİK.3
SAYFA Page No: 3/3

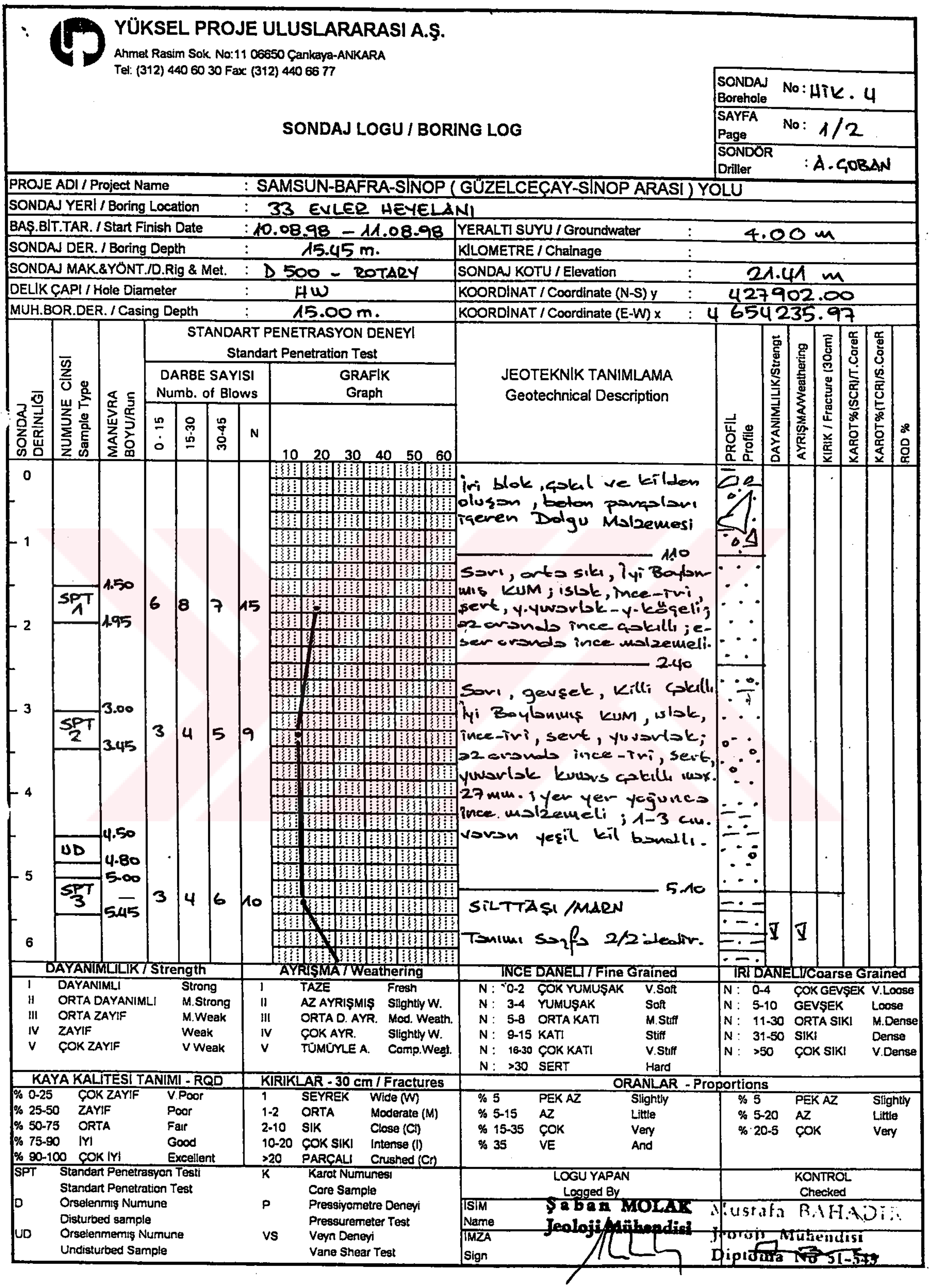
SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test – DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
16							Siltli Kötü Boylanmış KUM Tanımı sayfa 2/3'dedir							
16.50–16.95	SPT 11	16.50 16.95	24	30	38	68								
17							17.10							
							KİLTAŞI Gri-yeşilimsi gri, dağılgan, zayıf, çok ayrışmış. 1-5mm kalınlıkta 60°-90° yüzeyleri karbonat sıvalı laminasyonlara sahip. 18.20							
18	SPT 12	18.00 18.45	17	20	28	48								
19							MARN Yeşil-grimsi yeşil, dağılgan, zayıf, çok ayrışmış Süreksizlikler izlenemiyor.							
19.50–19.95	SPT 13	19.50 19.95	18	18	27	45			I	IV V				
20														
21	SPT 14	21.00 21.45	11	17	21	38								
22														
22.50–22.95	SPT 15	22.50 22.95	23	27	29	56								
23							KUYU SONU : 22.95 m.							



NOT : Kuyuya 22.50 m. İnklinometre borusu indirilerek betonlanmıştır.

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR
İMZA Sign		Jeoloji Mühendisi ~~Diploma No 51543~~

“Figure B5 (cont.)”



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SONDAJ LOGU / BORING LOG

SONDAJ / Borehole No : HİK. 4
SAYFA / Page No : 1/2
SONDÖR / Driller : A. ÇOBAN

PROJE ADI / Project Name : SAMSUN-BAFRA-SİNOP (GÜZELCEÇAY-SİNOP ARASI) YOLU
SONDAJ YERİ / Boring Location : 33 EVLER HEYELANI
BAŞ.BİT.TAR. / Start Finish Date : 10.08.98 – 11.08.98
YERALTI SUYU / Groundwater : 4.00 m
SONDAJ DER. / Boring Depth : 15.45 m.
KİLOMETRE / Chainage :
SONDAJ MAK.&YÖNT./D.Rig & Met. : D 500 - ROTARY
SONDAJ KOTU / Elevation : 21.41 m
DELİK ÇAPI / Hole Diameter : HW
KOORDİNAT / Coordinate (N-S) y : 427902.00
MUH.BOR.DER. / Casing Depth : 15.00 m.
KOORDİNAT / Coordinate (E-W) x : 4 654235.97

SONDAJ DERİNLİĞİ	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test: DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
0								İri blok, çakıl ve kilden oluşan, beton parçaları içeren Dolgu Malzemesi							
1								1.10							
	SPT 1	1.50 1.95	6	8	7	15		Sarı, orta sıkı, İyi Boylanmış KUM; ıslak, İnce-iri, sert, y.yuvarlak-y.köşeli; az oranda ince çakıllı; eser oranda ince malzemeli.							
2								2.40							
3	SPT 2	3.00 3.45	3	4	5	9		Sarı, gevşek, Killi Çakıllı İyi Boylanmış KUM, ıslak, ince-iri, sert, yuvarlak; az oranda ince-iri, sert, yuvarlak kuvars çakıllı max. 27 mm.; yer yer yoğunca ince malzemeli; 1-3 cm. varan yeşil kil bantlı.							
4	UD	4.50 4.80													
5	SPT 3	5.00 5.45	3	4	6	10		5.10							
6								SİLTTAŞI / MARN Tanımı Sayfa 2/2'dedir.		V	V				

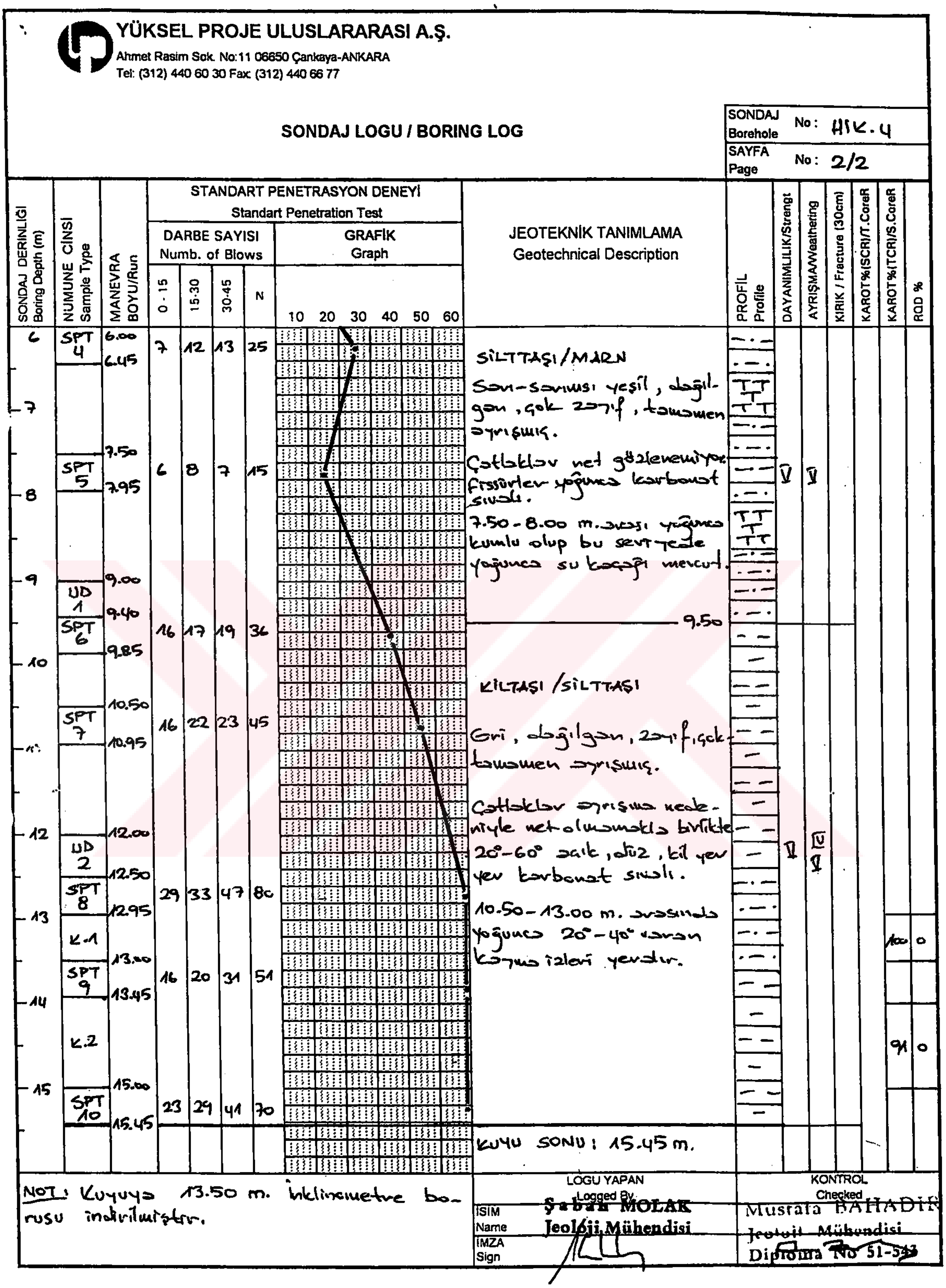
DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE DANELİ / Fine Grained			İRİ DANELİ / Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N : 0-4	ÇOK GEVŞEK	V.Loose
II	ORTA DAYANIMLI	M.Strong	II	AZ AYRIŞMIŞ	Slightly W.	N : 3-4	YUMUŞAK	Soft	N : 5-10	GEVŞEK	Loose
III	ORTA ZAYIF	M.Weak	III	ORTA D. AYR.	Mod. Weath.	N : 5-8	ORTA KATI	M.Stiff	N : 11-30	ORTA SIKI	M.Dense
IV	ZAYIF	Weak	IV	ÇOK AYR.	Slightly W.	N : 9-15	KATI	Stiff	N : 31-50	SIKI	Dense
V	ÇOK ZAYIF	V Weak	V	TÜMÜYLE A.	Comp.Weat.	N : 16-30	ÇOK KATI	V.Stiff	N : >50	ÇOK SIKI	V.Dense
						N : >30	SERT	Hard			

KAYA KALİTESİ TANIMI - RQD			KIRIKLAR - 30 cm / Fractures			ORANLAR - Proportions					
% 0-25	ÇOK ZAYIF	V.Poor	1	SEYREK	Wide (W)	% 5	PEK AZ	Slightly	% 5	PEK AZ	Slightly
% 25-50	ZAYIF	Poor	1-2	ORTA	Moderate (M)	% 5-15	AZ	Little	% 5-20	AZ	Little
% 50-75	ORTA	Fair	2-10	SIK	Close (Cl)	% 15-35	ÇOK	Very	% 20-5	ÇOK	Very
% 75-90	İYİ	Good	10-20	ÇOK SIKI	Intense (I)	% 35	VE	And			
% 90-100	ÇOK İYİ	Excellent	>20	PARÇALI	Crushed (Cr)						

SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample
D	Örselenmiş Numune Disturbed sample	P	Pressiyometre Deneyi Pressuremeter Test
UD	Örselenmemiş Numune Undisturbed Sample	VS	Veyn Deneyi Vane Shear Test

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR Jeoloji Mühendisi Diploma No 51-543
İMZA Sign		

Figure B6. Boring log of Hik 4



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HIK.4
SAYFA Page No: 2/2

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	0-15	15-30	30-45	N
6	SPT 4	6.00 6.45	7	12	13	25
7						
	SPT 5	7.50 7.95	6	8	7	15
8						
9	UD 1	9.00 9.40				
	SPT 6	9.40 9.85	16	17	19	36
10						
	SPT 7	10.50 10.95	16	22	23	45
11						
12	UD 2	12.00 12.50				
	SPT 8	12.50 12.95	29	33	47	80
13	K-1	13.00				
	SPT 9	13.00 13.45	16	20	31	51
14	K.2					
15	SPT 10	15.00 15.45	23	29	41	70

STANDART PENETRASYON DENEYİ / Standart Penetration Test — DARBE SAYISI / Numb. of Blows; GRAFİK / Graph: 10 20 30 40 50 60

JEOTEKNİK TANIMLAMA / Geotechnical Description

SİLTTAŞI/MARN

Sarı-sarımsı yeşil, dağılgan, çok zayıf, tamamen ayrışmış.

Çatlaklar net gözlenemiyor. Fissürler yoğunca karbonat sıvalı.

7.50-8.00 m. arası yoğunca kumlu olup bu seviyede yoğunca su kaçağı mevcut.

9.50

KİLTAŞI/SİLTTAŞI

Gri, dağılgan, zayıf, çok-tamamen ayrışmış.

Çatlaklar ayrışma nedeniyle net olmamakla birlikte 20°-60° açık, düz, kil yer yer karbonat sıvalı.

10.50-13.00 m. arasında yoğunca 20°-40° varan kayma izleri vardır.

KUYU SONU: 15.45 m.

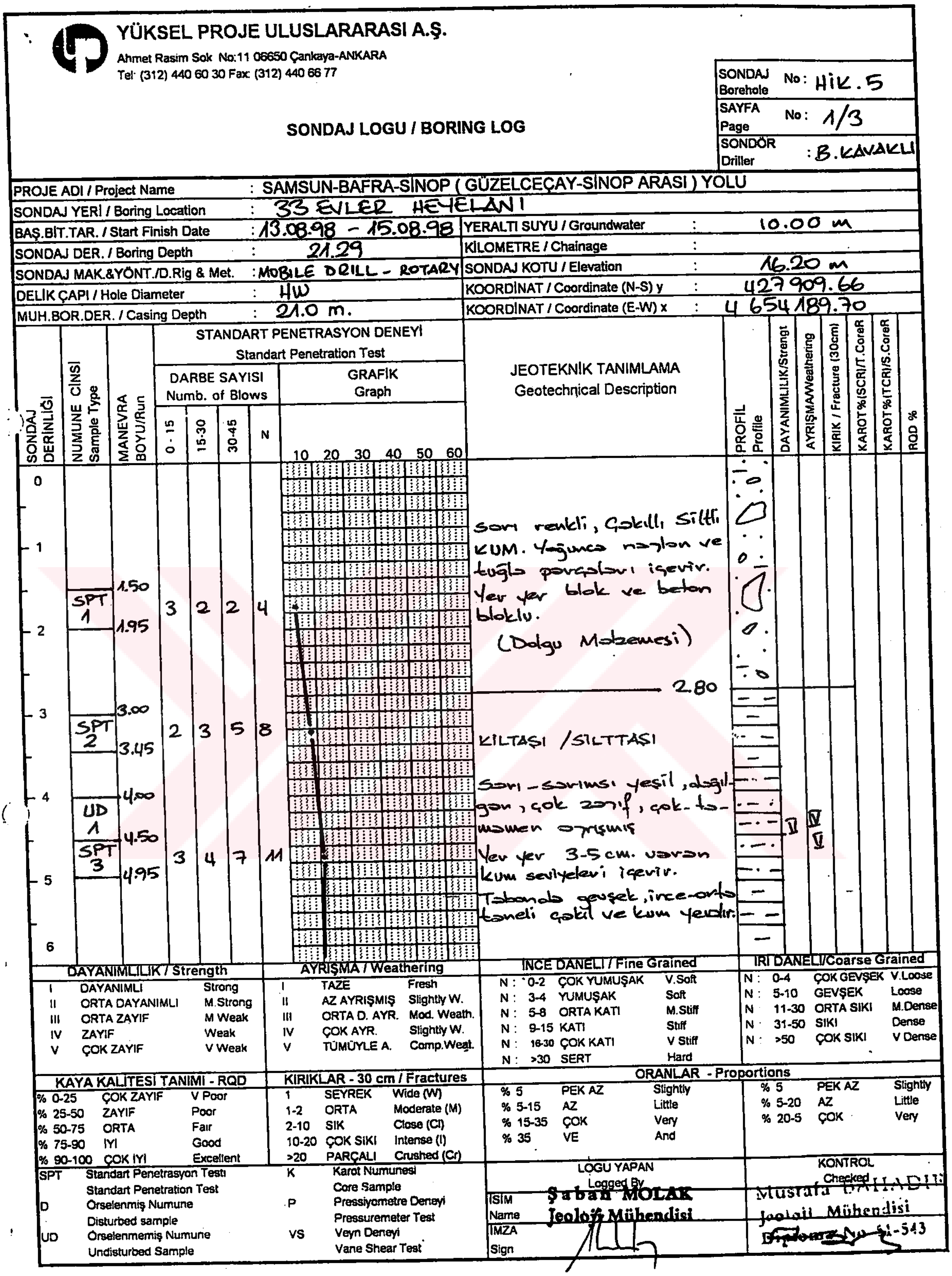
PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
6.00–9.50	V	V				
9.50–15.45	II	IV V				
					100	0
					91	0

NOT: Kuyuya 13.50 m. İnklinometre borusu indirilmiştir.

LOGU YAPAN / Logged By
İSİM / Name: Şaban MOLAK, Jeoloji Mühendisi
İMZA / Sign

KONTROL / Checked
Mustafa BAHADIR
Jeoloji Mühendisi
Diploma No 51-543

"Figure B6 (cont.)"



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No:	Hik.5
SAYFA Page No:	1/3
SONDÖR Driller	: B.KAVAKLI

PROJE ADI / Project Name	: SAMSUN-BAFRA-SİNOP (GÜZELCEÇAY-SİNOP ARASI) YOLU		
SONDAJ YERİ / Boring Location	: 33 EVLER HEYELANI		
BAŞ.BİT.TAR. / Start Finish Date	: 13.08.98 - 15.08.98	YERALTI SUYU / Groundwater	: 10.00 m
SONDAJ DER. / Boring Depth	: 21.29	KİLOMETRE / Chainage	:
SONDAJ MAK.&YÖNT./D.Rig & Met.	: MOBILE DRILL - ROTARY	SONDAJ KOTU / Elevation	: 16.20 m
DELİK ÇAPI / Hole Diameter	: HW	KOORDİNAT / Coordinate (N-S) y	: 427 909.66
MUH.BOR.DER. / Casing Depth	: 21.0 m.	KOORDİNAT / Coordinate (E-W) x	: 4 654 189.70

SONDAJ DERİNLİĞİ	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test — DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%/SCR/T.CoreR	KAROT%/TCR/S.CoreR	RQD %
0								sarı renkli, Çakıllı Siltli KUM. Yoğunca naylon ve tuğla parçaları içerir. Yer yer blok ve beton bloklu. (Dolgu Malzemesi)							
1.50 - 1.95	SPT 1	1.50 / 1.95	3	2	2	4									
2.80								2.80							
3.00 - 3.45	SPT 2	3.00 / 3.45	2	3	5	8		KİLTAŞI /SİLTTAŞI							
4.00 - 4.50	UD 1	4.00 / 4.50						Sarı-sarımsı yeşil, dağılgan, çok zayıf, çok-tamamen ayrışmış		V	IV V				
4.50 - 4.95	SPT 3	4.50 / 4.95	3	4	7	11		Yer yer 3-5 cm. varan kum seviyeleri içerir. Tabanda gevşek, ince-orta taneli çakıl ve kum yer alır.							
5															
6															

DAYANIMLILIK / Strength			AYRIŞMA / Weathering		
I	DAYANIMLI	Strong	I	TAZE	Fresh
II	ORTA DAYANIMLI	M.Strong	II	AZ AYRIŞMIŞ	Slightly W.
III	ORTA ZAYIF	M Weak	III	ORTA D. AYR.	Mod. Weath.
IV	ZAYIF	Weak	IV	ÇOK AYR.	Slightly W.
V	ÇOK ZAYIF	V Weak	V	TÜMÜYLE A.	Comp.Weat.

İNCE DANELİ / Fine Grained			IRI DANELİ/Coarse Grained		
N : 0-2	ÇOK YUMUŞAK	V.Soft	N : 0-4	ÇOK GEVŞEK	V.Loose
N : 3-4	YUMUŞAK	Soft	N : 5-10	GEVŞEK	Loose
N : 5-8	ORTA KATI	M.Stiff	N 11-30	ORTA SIKI	M.Dense
N : 9-15	KATI	Stiff	N 31-50	SIKI	Dense
N : 16-30	ÇOK KATI	V Stiff	N : >50	ÇOK SIKI	V Dense
N : >30	SERT	Hard			

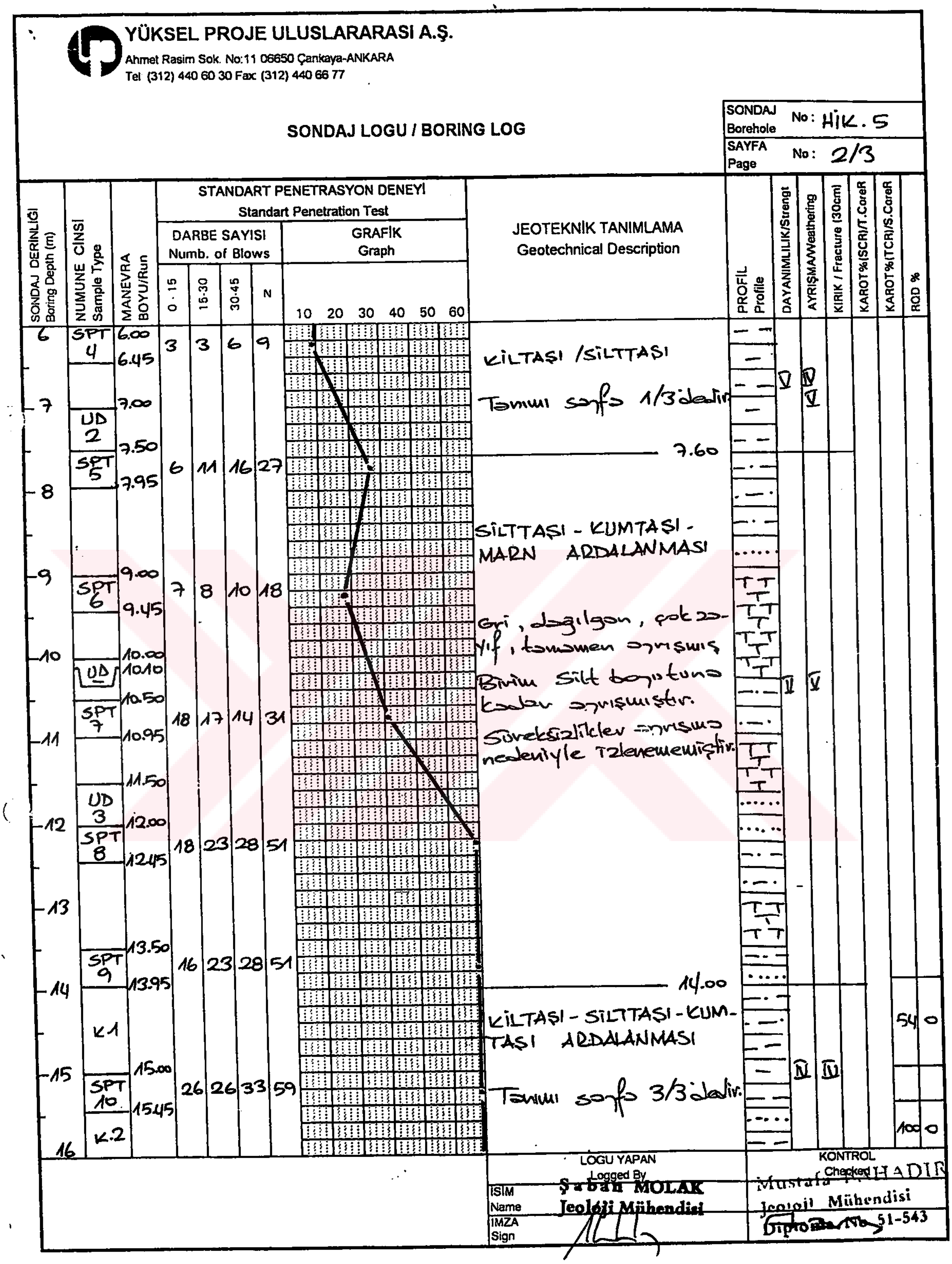
KAYA KALİTESİ TANIMI - RQD			KIRIKLAR - 30 cm / Fractures		
% 0-25	ÇOK ZAYIF	V Poor	1	SEYREK	Wide (W)
% 25-50	ZAYIF	Poor	1-2	ORTA	Moderate (M)
% 50-75	ORTA	Fair	2-10	SIK	Close (Cl)
% 75-90	İYİ	Good	10-20	ÇOK SIKI	Intense (I)
% 90-100	ÇOK İYİ	Excellent	>20	PARÇALI	Crushed (Cr)

ORANLAR - Proportions					
% 5	PEK AZ	Slightly	% 5	PEK AZ	Slightly
% 5-15	AZ	Little	% 5-20	AZ	Little
% 15-35	ÇOK	Very	% 20-5	ÇOK	Very
% 35	VE	And			

SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample
D	Örselenmiş Numune Disturbed sample	P	Pressiyometre Deneyi Pressuremeter Test
UD	Örselenmemiş Numune Undisturbed Sample	VS	Veyn Deneyi Vane Shear Test

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR Jeoloji Mühendisi Diploma No 51-543
İMZA Sign		

Figure B7. Boring log of Hik 5



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HİK.5
SAYFA Page No: 2/3

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	0-15	15-30	30-45	N
6	SPT 4	6.00–6.45	3	3	6	9
7	UD 2	7.00–7.50				
	SPT 5	7.50–7.95	6	11	16	27
8						
9	SPT 6	9.00–9.45	7	8	10	18
10	UD	10.00–10.10				
	SPT 7	10.50–10.95	18	17	14	31
11	UD 3	11.50–12.00				
12	SPT 8	12.00–12.45	18	23	28	51
13	SPT 9	13.50–13.95	16	23	28	51
14	K1					
15	SPT 10	15.00–15.45	26	26	33	59
16	K.2					

JEOTEKNİK TANIMLAMA
Geotechnical Description

KİLTAŞI /SİLTTAŞI

Tanımı sayfa 1/3'dedir.

7.60

SİLTTAŞI - KUMTAŞI - MARN ARDALANMASI

Gri, dağılgan, çok zayıf, tamamen ayrışmış. Birim silt boyutuna kadar ayrışmıştır. Süreksizlikler ayrışma nedeniyle izlenememiştir.

14.00

KİLTAŞI - SİLTTAŞI - KUMTAŞI ARDALANMASI

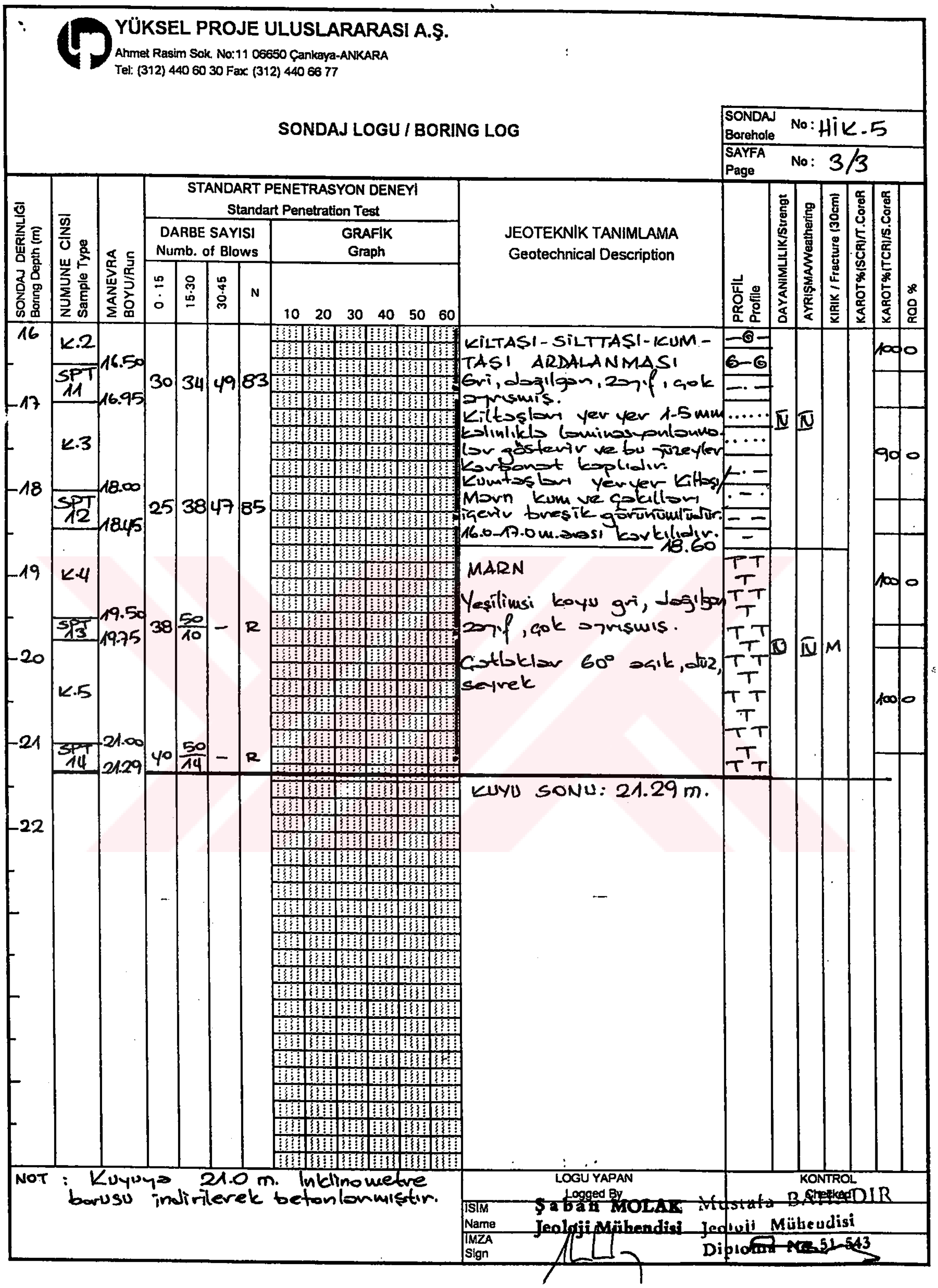
Tanımı sayfa 3/3'dedir.

PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
	V	IV V				
	V	V				
					54	0
	IV	IV				
					100	0

LOGU YAPAN
Logged By
İSİM Name: Şaban MOLAK
Jeoloji Mühendisi
İMZA Sign

KONTROL
Checked
Mustafa [illegible] HADIR
Jeoloji Mühendisi
Diploma No: 51-543

"Figure B7 (cont.)"



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No: HİK-5
SAYFA Page No: 3/3

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU/Run	STANDART PENETRASYON DENEYİ Standart Penetration Test — DARBE SAYISI Numb. of Blows 0-15	15-30	30-45	N	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strengt	AYRIŞMA/Weathering	KIRIK / Fracture (30cm)	KAROT%(SCR)/T.CoreR	KAROT%(TCR)/S.CoreR	RQD %
16	K.2						KİLTAŞI-SİLTTAŞI-KUMTAŞI ARDALANMASI Gri, dağılgan, zayıf, çok ayrışmış. Kiltaşları yer yer 1-5 mm kalınlıkta laminasyonlanmalar gösterir ve bu yüzeyler karbonat kaplıdır. Kumtaşları yer yer kiltaşı/marn kum ve çakılları içerir breşik görünümlüdür. 16.0-17.0 m. arası kırıklıdır. 18.60		IV	IV			100	0
	SPT 11	16.50 16.95	30	34	49	83								
17	K.3												90	0
18	SPT 12	18.00 18.45	25	38	47	85								
19	K.4						MARN Yeşilimsi koyu gri, dağılgan, zayıf, çok ayrışmış. Çatlaklar 60° açık, düz, seyrek		V	IV	M		100	0
	SPT 13	19.50 19.75	38	50/10	-	R								
20	K.5												100	0
21	SPT 14	21.00 21.29	40	50/14	-	R								
							KUYU SONU: 21.29 m.							
22														

GRAFİK Graph: 10 20 30 40 50 60

NOT: Kuyuya 21.0 m. inklinometre borusu indirilerek betonlanmıştır.

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Şaban MOLAK Jeoloji Mühendisi	Mustafa BAHADIR Jeoloji Mühendisi Diploma No.51-543
İMZA Sign		

"Figure B7 (cont.)"

APPENDIX C

LABORATORY TEST RESULTS

Table C1. Laboratory test results

SAMPLE			Wn	e	γ_n	G_s	ATTERBERG LIMITS			SIEVE ANALYSIS		SOIL TYPE	UNIAX IAL	TRIAXIAL TESTS (UU)	
Bore. No:	Samp. No.	Depth (m)	%		(t/m^3)		LL	PL	PI	+4 %	-200 %	USCS	q_u kg/cm^2	C kg/cm^2	ϕ°
HSK1	SPT1	3.1-3.45	19				26	17	9	8	38	SC			
	SPT2	4.5-4.95	32				33	20	13	1	71	CL			
	SPT4	7-7.45	17				37	21	16	47	32	GC			
	SPT6	10-10.45	26				41	22	19	11	80	CL			
	UD1	11-11.5	29				32	22	10	-	91	CL			
	SPT8	13-13.45	29				33	22	11	-	93	CL			
HSK2	SPT1	1-1.45	12				51	29	22	37	7	SW-SC			
	SPT3	4-4.45	22				-	-	-	6	24	SM			
	SPT4	5.5-5.95	27				37	25	12	-	91	ML			
	SPT5	7-7.45	33				59	30	29	-	93	CH			
	UD1	8.5-9	51				51	29	22	-	97	CH-MH			
	SPT7	10-10.45	20				50	26	24	-	78	CL-CH			
	UD2	11.5-12	33				64	34	30	-	99	MH			
	SPT8	12-12.45	31				64	36	28	4	95	MH			
	SPT9	13.5-13.95	29				58	31	27	-	99	MH			
	SPT11	16.5-16.95	33				57	30	27	-	85	MH			
	SPT13	19.5-19.95	31				59	30	29	-	99	CH			

"Table C1 (cont.)"

SAMPLE			Wn	e	γ_n	G_s	ATTERBERG LIMITS			SIEVE ANALYSIS		SOIL TYPE	UNIAXIAL	TRIAXIAL TESTS (UU)	
Bore. No:	Samp. No.	Depth (m)	%		(t/m^3)		LL	PL	PI	+4 %	-200 %	USCS	q_u kg/cm^2	C kg/cm^2	ϕ°
HIK1	SPT1	1.5-1.95	22				32	15	17	4	40	SC			
	UD1	2.5-3	39	1,03	1,85	2,7	59	29	30	1	72	CH	1,45		
	SPT2	3-3.45	28				65	31	34	-	99	CH			
	UD2	3.5-4	32	0,85	1,93	2,7	62	33	29	-	99	MH	2,99	1,20	5
	SPT4	6-6.45	33				64	31	33	-	100	CH			
	UD3	9.5-10	31	0,82	1,94	2,7	59	30	29	-	100	CH	4,49	2,75	2
	SPT8	12-12.45	29				70	30	40	-	100	CH			
	SPT10	15-15.45	30				63	31	32	-	100	CH			
HIK2	SPT1	1.5-1.95	28				45	22	23	-	88	CL			
	UD1	3-3.5	37				64	28	36	-	98	CH			
	SPT3	4.5-4.95	30				NOT ENOUGH			-	86	CH			
	SPT5	7.5-7.95	27				51	27	24	-	97	CH			
	UD2	8.5-9	27	0,69	2,00	2,67	50	26	24	-	99	CL/CH	2,22	2,60	1
	SPT7	10.5-10.95	29				51	27	24	-	99	CH			
	UD3	11.5-12	34	0,81	1,92	2,60	64	32	32	-	99	MH	2,63	2,60	2
	SPT8	12-12.45	29				67	29	38	-	99	CH			
	SPT10	15-15.45	28				65	30	35	-	99	CH			
HIK3	SPT1	1.5-1.95	27				36	22	14	-	71	CL			

"Table C1 (cont.)"

SAMPLE			Wn	e	γ_n	G_s	ATTERBERG LIMITS			SIEVE ANALYSIS		SOIL TYPE	UNIAXIAL	TRIAXIAL TESTS (UU)	
Bore. No:	Samp. No.	Depth (m)	%		(t/m^3)		LL	PL	PI	+4 %	-200 %	USCS	q_u kg/cm^2	C kg/cm^2	ϕ°
HIK3	SPT3	4.5-4.95	26				65	27	38	-	98	CH			
	SPT6	9-9.45	26				30	21	9	-	84	CL			
	SPT8	12-12.45	29				-	-	-	-	45	SM			
	SPT10	15-15.45	28				NOT ENOUGH			-	57	CL			
	SPT13	19.5-19.95	34				64	33	31	-	100	MH			
HIK4	SPT1	1.5-1.95	11				-	-	-	7	3	SP			
	SPT2	3-3.45	19				-	-	-	15	29	SM			
	SPT4	6-6.45	29				NOT ENOUGH			-	89	CL			
	UD1	9-9.4	31				62	32	30	-	98	CH			
	SPT7	10.5-10.95	30				69	32	37	-	100	CH			
	UD2	12-12.50	30				62	32	30	-	100	CH			
	SPT9	13-13.45	32				70	32	38	-	100	CH			
HIK5	SPT1	1.5-1.95	17				-	-	-	31	35	SM			
	SPT2	3-3.45	28				36	19	17	-	88	CL			
	UD1	4-4.50	37	1.03	1.85	2.74	50	24	26	-	96	CH/CL	0.70		
	SPT4	6-6.45	27				52	25	27	-	95	CH			
	UD2	7-7.5	34				49	24	25	-	25	CL			

“Table C1 (cont.)”

SAMPLE			Wn	e	γ_n	G_s	ATTERBERG LIMITS			SIEVE ANALYSIS		SOIL TYPE	UNIAXIAL	TRIAXIAL TESTS (UU)	
Bore. No:	Samp. No.	Depth (m)	%		(t/m^3)		LL	PL	PI	+4 %	-200 %	USCS	q_u kg/cm^2	C kg/cm^2	ϕ°
HIK5	SPT6	9-945	24				52	27	25	-	90	CH			
	UD3	11.5-12.00	26	0.72	1.98	2.70	31	20	11	-	95	CL	0.57		
	SPT8	12.-12.45	30				NOT ENOUGH			-	68	CL			
	SPT10	15-15.45	25				52	28	24	-	98	CH			
	SPT12	18-18.45	20				36	18	18	-	73	CL			