ASSESSMENT OF SLOPE STABILITY FOR KM: 109+590-128+630 SEGMENT OF KIRIKKALE-YERKÖY SECTION (SECTION-2) OF ANKARA-SİVAS HIGH-SPEED RAILWAY PROJECT

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ABSTRACT

ASSESSMENT OF SLOPE STABILITY FOR KM: 109+590-128+630 SEGMENT OF KIRIKKALE-YERKÖY SECTION (SECTION-2) OF ANKARA-SİVAS HIGH SPEED RAILWAY PROJECT

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Ankara-Sivas High-speed Railway Project is planned to be a part of an arterial railroad that will cross Turkey from west to east. Some slope instability problems occured after the excavation along Kırıkkale-Yerköy section of the project. Purpose of this study is to investigate engineering geological properties of the İncik formation exposed at four cut slopes with failure along the Kırıkkale-Yerköy section, to designate the factors that overbalance the stability of cut slopes and to recommend remedial solutions for problematic sections. In line with this purpose, shear strength parameters (c’ and φ’) of the İncik formation are investigated by back analysis method at four cut slopes KM:109+590, KM:113+120, KM:121+200 and KM:128+630 of Kırıkkale-Yerköy Section of Ankara-Sivas High-Speed Railway Project. As a comparison, shear strength parameters of the cut slopes were also checked against a neighbouring stable cut slope KM:107+100.

According to the slope stability analyses based on limit equilibrium methods, the most assuring solution technique for the failed cut slopes is found out to be piling solution at various levels of the slopes depending on where the highest force of the slices exists. Additionally, the slope at KM:107+100 where a transition zone between the İncik formation and the competent İç Anadolu Group exists is expected to be stable in the long term due to higher mass shear strength parameters of the units.

Keywords: Cut slope, İncik formation, limit equilibrium method, slope stability, Kırıkkale, Turkey

ÖZ

ANKARA-SİVAS YÜKSEK HIZLI DEMİRYOLU PROJESİ KIRIKKALE-YERKÖY KESİMİ (KESİM-2) KM:109+590-128+630 ARALIĞINDA ŞEV DURAYLILIĞININ DEĞERLENDİRİLMESİ

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Ankara-Sivas Yüksek Hızlı Demiryolu Projesi Türkiye'yi doğudan batıya kat edecek olan ana demiryolu hattının bir parçası olarak tasarlanmıştır. Kazı çalışmalarının başlamasına müteakip, projenin Kırıkkale-Yerköy kesimi boyunca bazı şev duraysızlığı problemleri gözlemlenmiştir. Bu çalışmanın amacı, Kırıkkale-Yerköy kesimi boyunca yenilmiş dört ayrı şevde mostra veren İncik formasyonunun mühendislik jeolojisi özelliklerinin araştırılması, şevlerin stabilitesini bozan etkenlerin belirlenmesi ve problemli kısımlar için iyileştirme çözümlerinin önerilmesidir. Bu amaçla, İncik formasyonunun Ankara-Sivas Yüksek Hızlı Demiryolu Projesi Kırıkkale-Yerköy Kesimi KM:109+590, KM:113+120, KM:121+200 ve KM:128+630 yarmalarındaki kesme dayanımı parametreleri (c' ve φ') geri analiz yöntemiyle araştırılmıştır. Karşılaştırma olarak, yenilen şevlerin kesme dayanımı civardaki yenilmemiş bir şev olan KM:107+100 şevindeki kesme dayanımı parametreleri de kıyaslanmıştır.

Limit denge yöntemlerine dayalı şev stabilite analizlerine göre, en fazla kuvvetin etkilediği dilimlerin konumuna bağlı olarak şevlerin farklı düzeylerinde, yenilmiş şevler için kazık yöntemi en güvenilir yöntem olarak belirlenmiştir. Ayrıca, İncik formasyonu ile dayanımlı İç Anadolu Grubu arasındaki geçiş zonunda yer alan KM:107+100 şevinin daha yüksek kesme dayanımı parametrelerine sahip olduğundan uzun vadede duraylı kalması beklenmektedir.

Anahtar kelimeler: Şev, İncik formasyonu, limit denge yöntemi, şev stabilitesi, Kırıkkale, Türkiye

To my family

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INTRODUCTION

1.1. Purpose and Scope

Being a part of Ankara-Sivas High-Speed Railway Project, Kırıkkale-Yerköy Section is projected by Turkish State Railways (TCDD) in order to increase the share of railways in transportation by providing a modern, comfortable and safe railway transportation. It is planned to be a part of an arterial railroad that will cross Turkey from west to east. As a continuation of european railways, continuing as a compact İstanbul-Ankara-Sivas line, it will reach up to Turkey's eastern border; thus will tie Europe with both Caucasia and Middle East. In addition, after both Ankara-İstanbul and Ankara-İzmir high speed trains are put into operation, it is estimated that there will be a heavy traffic on this line that will connect the east and west of Turkey.

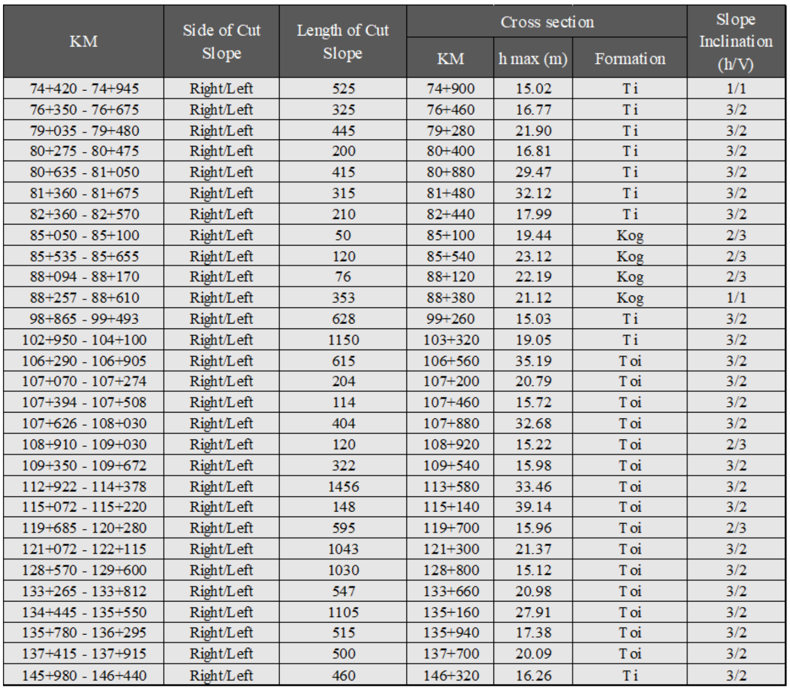
Kırıkkale-Yerköy Section of the project was planned to have a total length of 80 km with a maxiumum speed of 250 km/h, maximum longitudinal slope of %1.6, maximum horizontal curve of 3500 m and total excavation span of 14.5 m.

Along Kırıkkale-Yerköy Section of the Ankara-Sivas High-Speed Railway Line, there are several lithologies and a great number of cut-slopes. The cut slopes projected along the route of the Kırıkkale-Yerköy Section are being excavated within geological units such as İç Anadolu Group (Ti), Orta Anadolu Granitoid (Kog) and İncik formation (Toi). Cut slopes' inclinations along the route were determined by engineers based on engineering geological surveys, soil conditions and properties and existing road cut slopes in consideration of the rules stated in General Directorate of Highways

(KGM)’s Guide for Projecting Cut Slopes (KGM Şev Projelendirme Rehberi, 1989). Studies on cross sections were carried out on several slope combinations in order to ensure the long-term stability of the projected cut slopes in an economical and practical way. The cut slopes with a height of H>15m were evaluated in the category of “high cut-slope” and inclinations for these cut slopes were projected accordingly. A total of 29 high cut-slopes were projected for Kırıkkale-Yerköy Section of Ankara-Sivas High-Speed Railway Project (Table 1.1). Located within İncik formation, KM: 109+590-128+630 segment of the Kırıkkale-Yerköy Section has four main high cut slopes with instability problems.

The aim of this study is to investigate the shear strength parameters of the İncik formation to maintain stability of four cut slopes for failures, designate the factors that overbalance the stability and to pose feasible solutions for each cut slope within this section.

Table 1.1. Summary information of high cut slopes along Kırıkkale-Yerköy section (Yüksel Proje, 2011a)



KM	Side of Cut Slope	Length of Cut Slope	Cross section			Slope Inclination (h/V)
			KM	h max (m)	Formation	
74+420 - 74+945	Right/Left	525	74+900	15.02	Ti	1/1
76+350 - 76+675	Right/Left	325	76+460	16.77	Ti	3/2
79+035 - 79+480	Right/Left	445	79+280	21.90	Ti	3/2
80+275 - 80+475	Right/Left	200	80+400	16.81	Ti	3/2
80+635 - 81+050	Right/Left	415	80+880	29.47	Ti	3/2
81+360 - 81+675	Right/Left	315	81+480	32.12	Ti	3/2
82+360 - 82+570	Right/Left	210	82+440	17.99	Ti	3/2
85+050 - 85+100	Right/Left	50	85+100	19.44	Kog	2/3
85+535 - 85+655	Right/Left	120	85+540	23.12	Kog	2/3
88+094 - 88+170	Right/Left	76	88+120	22.19	Kog	2/3
88+257 - 88+610	Right/Left	353	88+380	21.12	Kog	1/1
98+865 - 99+493	Right/Left	628	99+260	15.03	Ti	3/2
102+950 - 104+100	Right/Left	1150	103+320	19.05	Ti	3/2
106+290 - 106+905	Right/Left	615	106+560	35.19	Toi	3/2
107+070 - 107+274	Right/Left	204	107+200	20.79	Toi	3/2
107+394 - 107+508	Right/Left	114	107+460	15.72	Toi	3/2
107+626 - 108+030	Right/Left	404	107+880	32.68	Toi	3/2
108+910 - 109+030	Right/Left	120	108+920	15.22	Toi	2/3
109+350 - 109+672	Right/Left	322	109+540	15.98	Toi	3/2
112+922 - 114+378	Right/Left	1456	113+580	33.46	Toi	3/2
115+072 - 115+220	Right/Left	148	115+140	39.14	Toi	3/2
119+685 - 120+280	Right/Left	595	119+700	15.96	Toi	2/3
121+072 - 122+115	Right/Left	1043	121+300	21.37	Toi	3/2
128+570 - 129+600	Right/Left	1030	128+800	15.12	Toi	3/2
133+265 - 133+812	Right/Left	547	133+660	20.98	Toi	3/2
134+445 - 135+550	Right/Left	1105	135+160	27.91	Toi	3/2
135+780 - 136+295	Right/Left	515	135+940	17.38	Toi	3/2
137+415 - 137+915	Right/Left	500	137+700	20.09	Toi	3/2
145+980 - 146+440	Right/Left	460	146+320	16.26	Ti	3/2

1.2.Location and Accessibility

The study area is located at 110 kms away from the city center of Ankara and located 22 km away from city center of Kırıkkale. The site is accessible through the Ankara-Kırıkkale-Yozgat D200 State Highway. The location map belonging to the study area is given in Figure 1.1.

Figure 1.1. Location map of the study area (Adapted from https://tr.wikipedia.org and https://earth.google.com/web/)

1.3.Vegatation and Climate

Being located in Central Anatolia, Kırıkkale city falls into one of Turkey's semi-arid regions. The dominant vegetation within the region is steppe. At higher districts that are free from devastation, forestlands consisting of dwarf oak and partly juniper are observed. The vegetation in the region is dominantly xerophytic and halophilous (Kırıkkale Valiliği, 2018).

The study area is located in an area where continental climate dominates, such that the summers are hot and dry, nights are cool and the winters are cold, rainy and snowy. Monthly average temperature and precipitation values for Kırıkkale city are given in Figure 1.2. The average temperature is highest in July (24.5°C) and it is coldest (-0.6°C) in January through the year. The average annual precipitation in Kırıkkale is 366.2 kg/m^2 (Figure 1.2). The average precipitation is the highest in May (56 mm/month) and the lowest in August (12 mm/month) (MGM, 2016). The average values for humidity and the days with frost are 63% and 80 days, respectively (Figure 1.3).

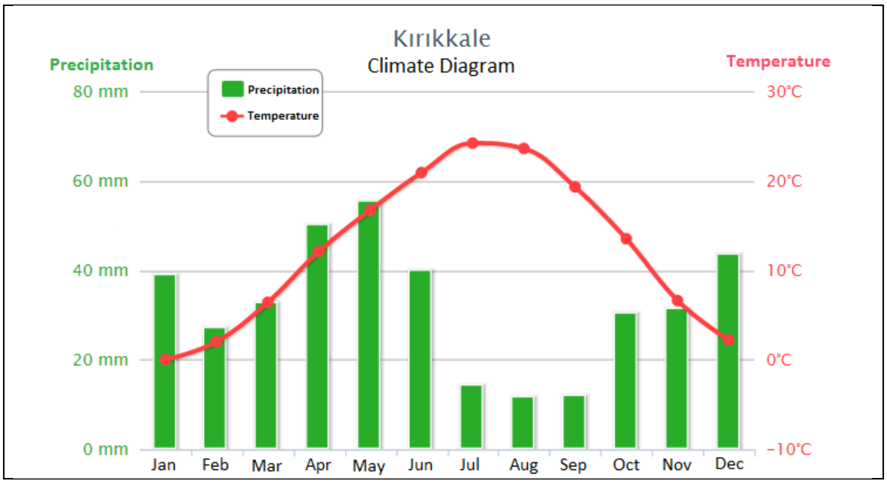


Figure 1.2. Monthly average temperature and precipitation data for Kırıkkale city (MGM, 2016) (Adapted from http://www.mgm.gov.tr)

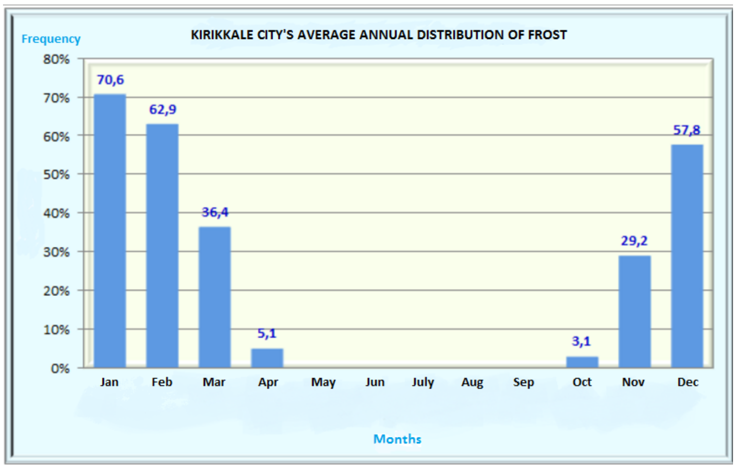


Figure 1.3. Monthly average annual distribution of frost for Kırıkkale city (MGM, 2016) (Adapted from http://www.mgm.gov.tr)

1.4.Method of Study

Firstly, in order to have a general vision of the study area, literature survey was initiated. Studies done around study area by Aral (1990), Kazancı (1999), Meydan (2005), Gülyüz (2009), Savaş (2010), Evcimen (2011), Sönmezer (2016) and some others that are not stated here gave a lot information about stratigraphy, sedimentology, paleontology and tectonics of the study area. Particularly, Kayaş-Yerköy Railway Section-2 (KM:74+100 – KM:153+725) Geological-Geotechnical Investigation Report of Yüksel Proje (2010) was benefited widely.

Secondly, exploratory drilling results and laboratory analyses were examined. Upon TCDD’s request, a sum of 97 exploratory drillings and 88 test pits were set by Yüksel Proje throughout railway route where 4 of the drillings were done at close vicinity of the cut slopes. In order to establish the geological cross-section of the study area, disturbed (SPT), undisturbed (UD) and core samples were taken from the drillings. Grain size distribution, liquid limit, plastic limit, plasticity index, water content in addition to shear strength parameters (c’ and φ’) of soils were specified at Yüksel Proje Uluslararası A.Ş. relevant soil and rock mechanics laboratory and tests were applied on the samples taken from the drillings and test pits (Yüksel Proje, 2011). All the tests

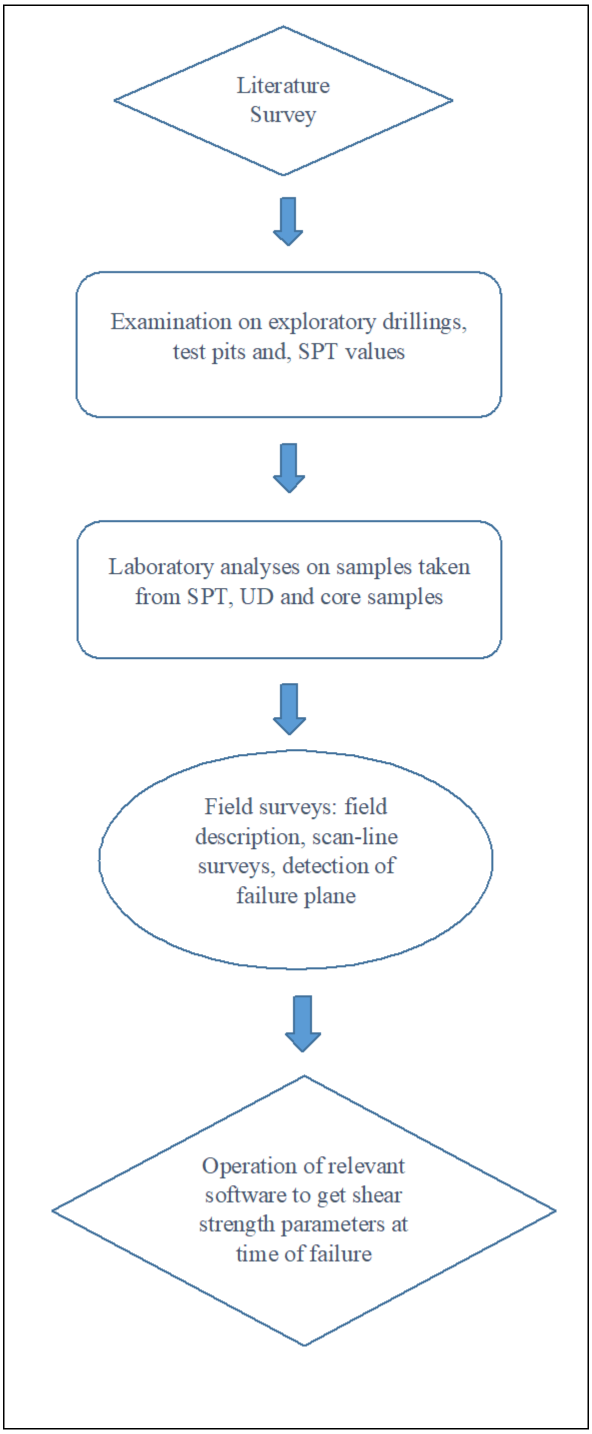


Figure 1.4. Flowchart illustrating the steps on method of study

were conducted in accordance with ASTM and/or TSE standards. Hydrogeology of the study area was examined based on the exploratory drilling data.

Thirdly, field surveys for site observation, data collection and rock mass classification were conducted. Field description, scan-line surveys and detection of scarp, toe, flank and probable failure plane of landslide at the four cut slopes were carried out.

Lastly, the rock mass properties of the İncik formation were tried to handle by operation of a relevant software in order to have the knowledge of the failed cut slopes. The four cut slopes were modeled based on the field survey and a failure criterion for the formation was introduced. Shear strength parameters of the formation at four cut slopes were determined by means of back analyses method via Rocscience SLIDE 6.0 software (Rocscience, 2002a). In the light of these analyses, limit equilibrium analyses and feasible remediation techniques for each cut slope were proposed to maintain long-term slope stability by taking the landslide mechanism, geometry and parameters determined from the geotechnical investigations into account. As a crosscheck, the situation at the four cut slopes were also checked in the neighbouring not-failed cut slope at KM:107+100 via RocLab software (Rocscience, 2002b) for circular failure.

1.4.1.Rocscience SLIDE Software

SLIDE 6.0 is a 2D slope stability program for estimating the safety factor in circular or non-circular failure surfaces for rock or soil slopes. Rocscience SLIDE software prospects the slip surfaces’ stability using vertical slice limit equilibrium methods. In each slope, single slip surfaces are analyzed and search methods are applied to determine the location of the critical slip surface in a particular slope. Characteristics of the software include

- Determination of critical surfaces both for circular or non-circular slip surfaces
- Numerious analysis methods such as Bishop, Janbu, Spencer, Morgenstern&Price

- Various materials: Anisotropic, non-linear Mohr Coulomb materials, and some other strength models
- Groundwater conditions: Ru factors, piezo surfaces, pore pressure grids, or steady state groundwater analysis
- Tension cracks
- External loading
- Support types: Soil nails, tiebacks, geotextiles, piles
- Access to all surfaces constituted by search
- Plotting detailed analysis results for discrete slip surfaces.

A number of analyses are implemented to specify the reasons of failure when a slope has failed. If a failure surface is known, a method of “back analysis” can be conducted so as to specify the shear strength, pore pressure and/or other conditions of soil or rock material at failure time. Having back analysis results in hand, the remedial slope stability measures can be designed.

There are a number of methods for employing back analysis:

- Operating trial and error method manually in order to match the specified input data with observed behaviour
- Sensitivity analysis for a single variable
- Probabilistic analysis for two correlated variables
- Advanced probabilistic analysis methods for simultaneously analysing multiple parameters

Under the assumption of cohesion and friction angle parameters are unknown for a formation, a probabilistic analysis can be used to specify a correlation between cohesion and friction angle, where a safety factor of 1 is presumed for a given failure surface in this way there would be an infinite number of solutions to the problem rather than a single exact answer (Rocscience, 2002c).

1.5.Previous Studies

Birgili et al. (1975) named İncik formation and defined its age as Late Eocene-Early Eocene.

Aral (1990) dealed with stratigraphic position, sedimentological properties and evolution of Oligocene aged green copper deposits located at northern side of Delice Creek between Delice and Yerköy, which is in neighbourhood of the study area of this thesis on its noutheastern border. All copper deposits mentioned in his study exist in Topraklıktepe formation which is stated as equivalent of İncik Formation (Meydan, 2005). Aral defined Topraklıktepe formation as red and grey terrestrial sandstone, pebblestone and mudstone.

Kazancı et al. (1999) studied a new Late Miocene mammal taking place in central Anatolia specifically at southern Çankırı-Çorum Basin. This region falls into 80 km northwest of the study area in this thesis. The Çankırı-Çorum Basin is composed of deposits ageing between Late Paleocene to Pleistocene. Mainly three periods of sedimentation, namely; Late Paleocene-Late Eocene marine deposits, Early-Late Miocene fluviolacustrine deposits, and Late Miocene-Pleistocene fluviatile deposits were distinguished by two major angular unconformities (Birgili et al., 1975). The process of Çankırı-Çorum Basin’s filling with volcanic products have started with Eocene which reaches up to a maximum during the Late Miocene-Pliocene interval. The Galatia and Cappadocia volcanic complexes surround and locally cover this basin from north. These volcanic complexes are the two primary sources for sediments filling up the Çankırı-Çorum Basin. According to the researchers mammal-bearing tuff horizon at Akkaşdağı and the areas in vicinity are presumably originated from the northwestern edge of the Cappadocian volcanic complex.

Meydan (2005) studied neotectonics and seismicity of Delice-Çerikli-Salmanlı area, which is in neighbourhood of the study area of this thesis on its southwestern border.

In the study, he defined the İncik formation as poorly bedded and poorly sorted terrestrial formation composed of lacustrine and fluvial facies. Lithology of the formation was described as conglomerate, sandstone, siltstone, mudstone, gypsum and marl. Büyükpolatlı formation (Şenalp, 1974), Bahşili formation (Norman, 1975) and Topraklıktepe formation (Aral, 1990) were stated as equivalent of the İncik formation.

Gülyüz (2009) studied the evolution of Çiçekdağı Basin. This region lies nearly 70 km south of the study area in this thesis. He described the İncik formation as red or brown colored, cross-bedded, thin to thick layered and graded alternations of sandstone and mudstone with local conglomerate lenses, red, brown, gray colored, medium to thick bedded, graded and cross bedded sandstone including conglomerate lenses and red, brown, gray colored, coarse grained, locally consolidated conglomerate and sandstone alternations from bottom to top. Also, he defined magmatic, ophiolitic, rarely metamorphic and reworked limestone fragments and reworked nummulites as constitutive particles of the İncik formation.

Savaş (2010) studied geological and geotechnical analysis of a soilwaste landfill site in Kırıkkale, which falls into 25 km west of the study area in this thesis. He described the İncik formation as talus and fluvial and lacustrine facies formed in terrestrial environment. Accordingly, talus was represented with conglomerate and locally sandstone in a trace of mudstone, and fluvial facies were made up of cross bedded conglomerate, sandstone, siltstone, claystone and marl.

Evcimen (2011) studied geology in the vicinity of Sulakyurt (Kırıkkale) and assesment of Sulakyurt Granitoid which is commonly used in Kırıkkale as facing stone. She described İncik formation to be made up of playa lake and mountain sediment. Evaporites within the unit were stated to be formed as a result of excessive evaporation. The unit was defined as reddish brown, poorly-sorted, angular breccia marl, gypsum alternating with carbonates, cross-bedded sandstone, siltstone and red mudstone. The author mentions that Bahşili formation (Norman, 1972) cropping out near Yahşihan (Kırıkkale), Büyükpolatlı formation (Şenalp, 1974) outcropping near Sungurlu (Çankırı), Bala formation (Arıkan, 1975) outcropping near Lake Salt-Bala (Ankara), as equivalent of the İncik formation, Miskincedere formation (Akyürek et al., 1982) outcropping near Elmadağ (Ankara), Parmaklıktepe formation (Gökten et al., 1988) outcropping near Bağlum-Kazan (Ankara) as also an equivalent of the İncik formation.

Sönmezer (2016) studied earthquake risk analysis and seismic microzonation in the city center of Kırıkkale which falls about 20 km west of the study area in this thesis. Kırıkkale was defined to be surrounded by some active faults like Seyfe Fault Zone, Karakeçili Fault Zone and Kırıkkale-Sungurlu Fault Zone. Akpınar earthquake took place on Seyfe Fault Zone with a magnitude of Ms=6.8 in 1938, 50 km's away from Kırıkkale indicated to be a severe earthquake that examplifies the seismicity within the study area.

CHAPTER 2

BACKGROUND INFORMATION ON SLOPE STABILITY

Investigation of slope stability is the major critical problem specifically encountered in large and outstanding projects such as railways, highways and tunnels (Pourkhosravani and Kalantari, 2011). The stability of earth embankments or slopes, as they are commonly called, should be very thoroughly analyzed since their failure may result in loss of human life and/or colossal-economic loss (Burman et al., 2015).

Slope failures have raised apprehension to the public safety, gave rise to construction delays and lead to costly repair work (Yang, 2005). Ahmadi-Adli (2014) defined slope instability as downward and outward movement of the material forming the slope under the effect of gravitational and some other forces due to shear failure at the boundaries of the moving mass. Changing physical parameters such as increasing water content in the soil or rock during rainy seasons may result in slopes' losing their stability. In addition, the fluctuations in physical parameters due to seasonal variations, anthropogenic intervention and continuous removal of toe material comes up with iterative failure of slope in chips and parts (Sharma et al., 2017).

Surface degradation and/or sectional landslides occasionally come up on cut slopes constituted in soft rocks. Mineral content, preconsolidation background, cement composition in their structure, degree of cementation and texture are the deterministic features of clay-bearing rocks and their attitudes when exposed to external influences on cut slopes. Water has a very important impact in the fluctuation of features of the rocks dominantly composed of clay. This situation is observed through the repeating procedures of wetting and drying, thawing and freezing, and via several chemical processes. The effect is put forward in the dissipation of cement from a clay dominated rock and in deterioration of rock into pieces. Thus, the rock is concurrently disturbed by physical and chemical weathering activities. Additionally, stress release aroused by removing material during excavation has potential of forming new joints. The generation of new joints accelerates physical weathering and serves for thoroughly penetration of chemical weathering impacts (Miscevic and Vlastelica, 2014). Wherever weak zones such as joints and faults form a failure route, they bring a risk of failure into existence (Gökçeoğlu et al., 2000).

Most slope failures are complicated events where evaluating the criteria that ascertain slope stability are hard to measure, especially shear strength parameters and groundwater conditions (Yang, 2005). It is appearent that geometry of the failure surface and clues of the failure mechanism are essential for the analysis and stabilization of unstable slopes (Mahmoud et al., 2011). The purpose of analyzing slope stability must concentrate on whether a slope is safe and on evaluation of the safety factor before failure and guessing the mechanism of failure so as to provide an essential background for the remedial design (Yang, 2005).

In general, natural or cut slope failures come up more frequently compared to other geotechnical failures like tunnel or foundation, for this reason, a huge number of researchers made studies on slope stability analyses techniques (Gökçeoğlu et al., 2000). Previously slope stability analysis methods were mostly based on hand-performed simplistic computations. As more powerful computers became available in time, engineers have used not only complicated but also more accurate methods (Pourkhosravani and Kalantari, 2011).

The prevalent computational slope stability methods are Fellennius (1936) ordinary method of slices, Bishop (1955) simplified method, Bishop (1955) rigorius method, Lowe and Karafiath (1960) method, Morgenstern-Price (1965) method, Spencer (1967) method, Janbu (1968) simplified method, Janbu (1968) generalized method, Modified Swedish method (US Army Corps of Engineers 1970) and Sarma (1973) method. These methods are grouped on the basis of solving the equations formulated on the methods of slices. A comprehensive review and summary on these computational methods were provided by Fredlund and Krahn (1977), Duncan (1996) and Abramson et al. (2002) (Yang, 2005).

The Bishop (1955) simplified, the Janbu (1968) and the Morgenstern-Price (1965) methods are the most widely referred ones because of their readiness in calculation of safety factor in slip surfaces (Abramson et al. 2002). Still, safety factor dominantly specified based on a given slip surface (Yang, 2005). For this reason, it is important to put a complete, repetitive research in progress for a critical slip surface in order to acquire the minimum safety factor, without bothering the calculation method of analysis (Duncan, 1996).

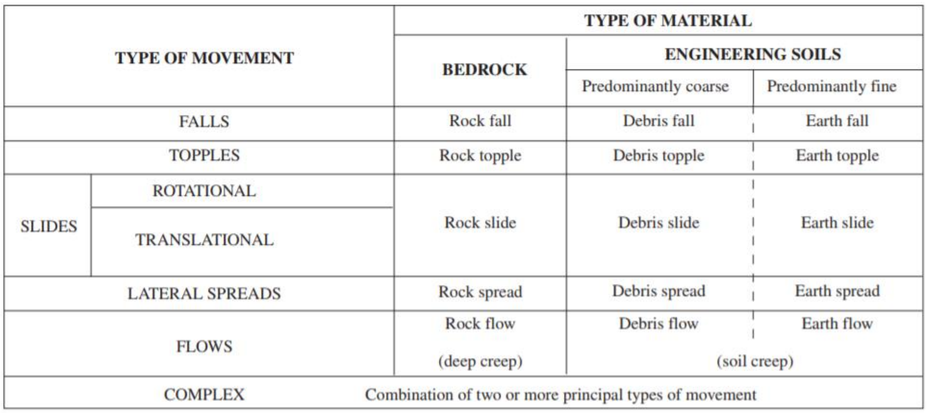
2.1. Classification of Landslides and Failure Types of Slopes

2.1.1. Classification of Landslides

Alpine countries are the pioneers in suggestion of the earliest landslide classification systems. Baltzer (1875) in Switzerland is the first one to differentiate the numerous types of motion such as slide, fall and flow. That classification have stood until now, with inclusion of toppling and spreading (Hungr et al., 2013). The 1978 version of the "Varnes Classification System" is dominantly accepted by engineers in many countries, providing with some modifications i.e. Highland and Bobrowsky (2008) and Dikau et al. (1996). A framework of 16 landslide types in which rows demonstrate the type of movement where columns demonstrate the type of material are given as the combinations of movements and materials in Table 2.1 (Varnes, 1978).

The types of movement are divided into five groups i.e. falls, slides, topples, spreads, and flows. As an additional type, complex slope movement is a combination of the other five types. Material types are comprised of two classes i.e. rock and engineering soil where soil is divided into two subtitles as debris and earth. In the table, landslide categorization is given upon two features. The first feature represents the type of material i.e. rock, earth, soil, mud or debris. Second term represents the movement type whether it is flow, spread, slide, fall or topple (Varnes, 1978).

Table 2.1. Abbreviated version of Varnes' classification of slope movements (Varnes, 1978)



TYPE OF MOVEMENT		TYPE OF MATERIAL		
		BEDROCK	ENGINEERING SOILS	
			Predominantly coarse	Predominantly fine
FALLS		Rock fall	Debris fall	Earth fall
TOPPLES		Rock topple	Debris topple	Earth topple
SLIDES	ROTATIONAL	Rock slide	Debris slide	Earth slide
	TRANSLATIONAL			
LATERAL SPREADS		Rock spread	Debris spread	Earth spread
FLOWS		Rock flow	Debris flow	Earth flow
		(deep creep)	(soil creep)	
COMPLEX		Combination of two or more principal types of movement		

A velocity scale for landslides is given in Table 2.2 which is developed by International Geotechnical Society's UNESCO Working Party on World Landslide Inventory (WP/WLI) (1995) and completed by Cruden and Varnes (1996).

Table 2.2. Landslide velocity scale (Adapted from WP/WLI, 1995 and Cruden and Varnes, 1996)

Velocity Class	***Description***	***Velocity (mm/s)***	***Typical Velocity***	***Response***
7	Extremely rapid	5x10^{3}	5 m/s	Nill
6	Very rapid	5x10^{1}	3 m/min	Nill
5	Rapid	5x10^{-1}	1.8 m/h	Evacuation
4	Moderate	5x10^{-3}	13 m/month	Evacuation
3	Slow	5x10^{-5}	1.6 m/year	Maintenance
2	Very slow	5x10-7	16 mm/year	Maintenance
1	Extremely slow			Nill

In case of falls, a mass with any size seperates from a slope along a face where there is almost no shear displacement, and drops dominantly by free fall, bouncing, rolling, and sliding. Mass moves very rapidly resulting in progressive excursion of the piece from its source (Varnes, 1978). Topples are accepted as a divergent type of movement recently. This type of movement is comprised of the rotation of rocks about some spindle points under the impact of gravity that is implemented by surrounding rocks or by fluids in voids (Varnes, 1978). In true slides, movements are comprised of shear strain and displacement throughout a single or multiple surfaces which are noticeable or may sensibly be presumable. The slide can be progressive, where shear failure does not preliminarily take place on a previously particular surface of rupture, instead it may develop from a site of local failure (Varnes, 1978). Lateral spreads are typical since they develop on relatively mild slopes or flat terrains. The prevalent movement type is a lateral extension coming along fractures formed by shear or tensile forces. Thus, failure is induced by liquefaction, an algorithm where loose, cohesionless, saturated sediments turn to a liquefied state from solid (Islam and Ryan, 2016).

In many slopes, it is not possible to categorize movements as falls, topples, slides, or spreads. For unconsolidated materials, the movement is usually in the form of fast or slow and/or wet or dry flow. In the case of bedrock, the movements are more troublesome to classify including the ones that are slow and distributed between many frequent, non-interconnected fractures or the movements in the rock mass that resulted in folding and bending. In many cases, the characteristics of velocities resembles that of viscous fluids, thereby, the movements may be described as a form of intact rock’s flow (Varnes, 1978).

2.1.2. Failure Types

Slope failures categorized into four types based on the discontinuity’s geometrical and mechanical constitution and the rock mass conditions as shown in Figure 2.1. Circular failures take place where the formations are notably fractured or are comprised of very weak content. Planar failures take place at locations where a discontinuity strikes parallel to the slope face and dives into the cut slope with an angle larger than the friction angle that is the angle with peak value that the slope will stand without sliding. Wedge failures implicate a rock mass restricted by two discontinuities with a concurrence line which trend out from the slope face where the trend of the junction line is considerably larger than the friction angle. Toppling failures are comprised of rock slabs restricted by discontinuities which dip steeply into the slope face (Terry and Kyu, 2007).

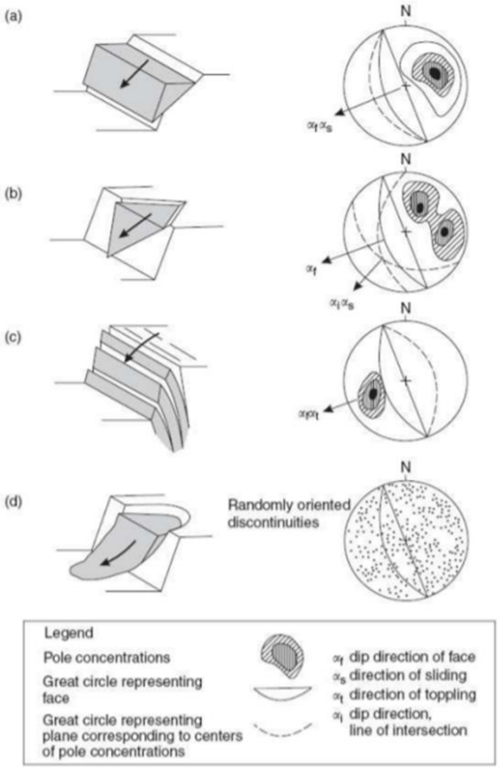


Figure 2.1. Four types of rock slope failure (a) Planar Failure (b) Wedge Failure (c) Toppling (d) Circular Failure (modified from Hoek and Bray, 1981)

2.2. Slope Stability and Shear Strength of Weak Rocks

Weak rocks are critical geomaterials since they incorporate numerous problems. Firstly, they usually exhibit undesirable behaviors like poor strength, disaggregation, crumbling, high plasticity, slaking and weathering. Secondly, soft rocks possess strength ratings between soils and hard rocks. Occasionally, they are too weak to be tested in rock mechanics tests and too strong for soil mechanics tests (Kanji, 2004). Besides, the shear strength parameters may alter because of any disturbance i.e. excavation or weathering (Ersöz, 2017; Ersöz and Topal, 2018a, b). Shear failure is extremely prevalent in cut slopes that is made up of weak, weathered or crushed rocks (Goodman, 1989). Due to such kind of variations, stability of rock or soil slopes comes up to be a significant matter for engineers (Ersöz, 2017¸ Ersöz and Topal, 2018a, b).

Presently numerous slope stability calculation methods are used depending on the balance of forces, moments or energy balances (Harabinová, 2017). No researches can be get through without the mathematical description of the soil-water characteristic curve (SWCC) on unsaturated soils' mechanical properties. The mathematical expression of the SWCC is crucial for the strength formula development and formative relation for unsaturated soils. Presently, there are four mathematical models generated for the estimation of SWCC (Sheng 2011). These are exponential model (Van Genuchten and Leiji, 1992), log-exponential model (Fredlund and Morgenstern, 1977) and (Fredlund and Rhadajo, 1993), multi-fractal model (Campbell, 1974), and non-linear logarithmic model (Brooks and Corey, 1964). The unsaturated Fredlund method requires the entry of the soil-water characteristic curve (SWCC), depending on the volume of water present in the soil at a particular suction level (Fredlund and Xing, 1994), i.e.

$$\tau_{ff} = c' + \left(\sigma_f - u_w\right)_f tan\phi' + (u_a - u_w)_f tan\phi'' \tag{2.1}$$

where τ_{ff} = shear strength, $(\sigma_f - u_w)_f$ = net normal stress state with respect to the pore pressure on the failure plane at failure, $(u_a - u_w)_f$ = the matric suction at failure, **f**"= friction angle associated with the matric suction stress state variable (Batali and Andreea, 2016).

For these models, the Mohr-Coulomb criterion is operated most frequently in order to formulate the mechanical strength of unsaturated soil (Li et al., 2017). The shear strength of the geological formation in Mohr-Coulomb failure criterion is calculated by the formula given below, where c is cohesion, ϕ is internal friction angle, σ is normal and τ is shear stress:

$$\tau = c + \sigma tan\phi \tag{2.2}$$

Knowing that most geotechnical softwares are still expressed by means of the Mohr-Coulomb failure criterion, procuring internal friction angle and cohesive strength for all rock masses are essential. This can be practiced by fitting an approximate linear relationship to the curve constituted by figuring out Equation 2.2 for minor principal stresses stated as $\sigma_t < \sigma < \sigma'_{3max}$, given in Figure 2.2. Fitting procedure comprises compensating the above and below fields in the Mohr-Coulomb graph. The process comes up with the equations below for the internal friction angle and cohesion of rock mass (Hoek et al., 2002):

$$\phi' = \sin^{-1}\left[\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3n})^{a-1}}\right]$$

$$c' = \frac{\sigma_{ci}\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+\left(6am_b(s+m_b\sigma'_{3n})^{a-1}\right)/\left((1+a)(2+a)\right)}}$$

$$\text{where} \quad \sigma_{3n} = \sigma'_{3\max}/\sigma_{ci} \tag{2.3}$$

Currently, latest release of generalized Hoek-Brown failure criterion is implemented through various softwares like Rocscience RocLab for specifying rock mass parameters. This software procures a simple and intuitional prosecution of the Hoek-Brown failure criterion, permitting to attain realistic prediction of rock mass properties and to envision the results of altering rock mass parameters in the failure envelopes. RocLab determines the generalized Hoek-Brown strength parameters of a rock mass by implementing Hoek-Brown classification parameters given below:

- the unconfined compressive strength of intact rock σ_{ci}
- the geological strength index GSI
- the intact rock parameter m_i and
- the disturbance factor D

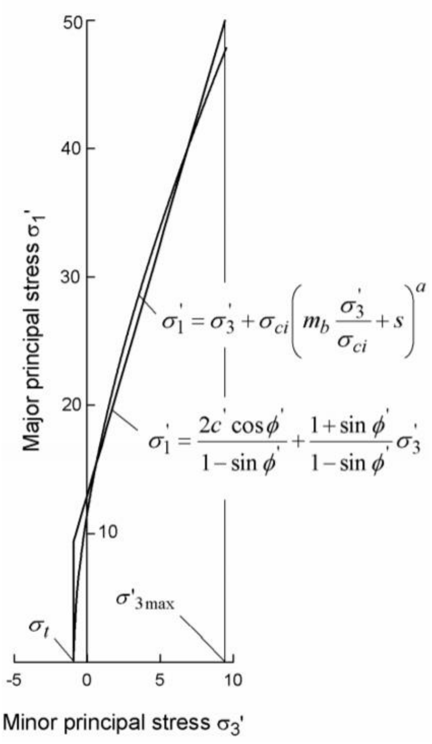


Figure 2.2. Relationships between major and minor principal stresses for Hoek-Brown and equivalent Mohr-Coulomb criteria (Hoek et el., 2002)

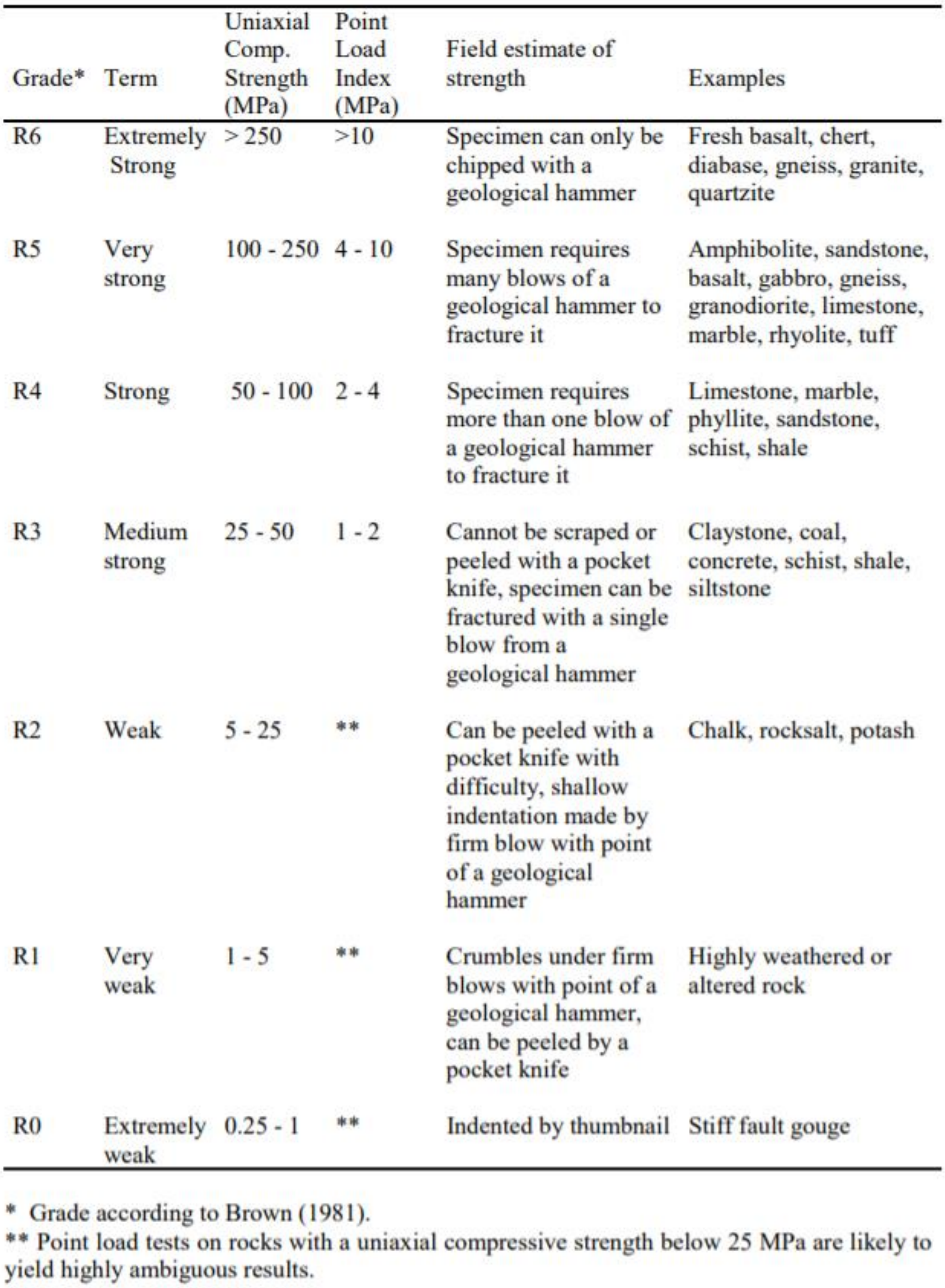
These 4 parameters (σ_{ci}, GSI, mi and D) can easily be presumed from constituted charts and tables, depending on rock type, geological conditions, etc. (Rocscience, 2002b).

2.2.1. The Uniaxial Compressive Strength (σ_{ci})

σ_{ci} is the uniaxial compressive strength of the intact rock which forms the rock mass. Still, this estimation may not be consistent with uniaxial compressive strength attained from laboratory analyses or the UCS tests. Certainly, nearly all samples picked from rocks will contain some discontinuities i.e. bedding, schistosity planes or joints. For instance, while working on rock masses like flysch, it can be pretty hard to obtain an intact specimen for uniaxial compressive testing. Eventually, the laboratory tests conducted on core samples may conclude in strength values which are lower than that of uniaxial compressive strength σ_{ci} essential as data for the Hoek-Brown criterion. Utilizing the results of these kind of tests may lead to more errors on the strength so may give illusively poor ratings for the rock mass strength. Occasionally, when the rock masses are very closely jointed and if it is possible to get undisturbed core samples, uniaxial compressive strength tests are conducted directly on the rock mass (Jaeger, 1971).

By applying the Point Load Test on specimens where the load is applied normally on bedding or joints can be stated as one of the maneouvres that can be taken to sort out this dilemma. Dealing with very weak rocks like clayey shales, notches of the loading spots may come up with plastic deformation instead of rupturing of sample. Point Load Test does not yield dependable results for such cases. The only feasible choice is to upt for a qualitative expression of the rock so as to determine the uniaxial compressive strength of the intact rock at times it is impossible to attain specimens for Point Load Testing. Table 2.3 gives an eligible description of field ratings of uniaxial compressive strength based on Hoek and Brown Criterion (1997).

Table 2.3. Field estimates of uniaxial compressive strength (Hoek, 2007)



Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field estimate of strength	Examples
R6	Extremely Strong	> 250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 - 250	4 - 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, phyllite, sandstone, schist, shale
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Claystone, coal, concrete, schist, shale, siltstone
R2	Weak	5 - 25	**	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, rocksalt, potash
R1	Very weak	1 - 5	**	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock
R0	Extremely weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

* Grade according to Brown (1981).

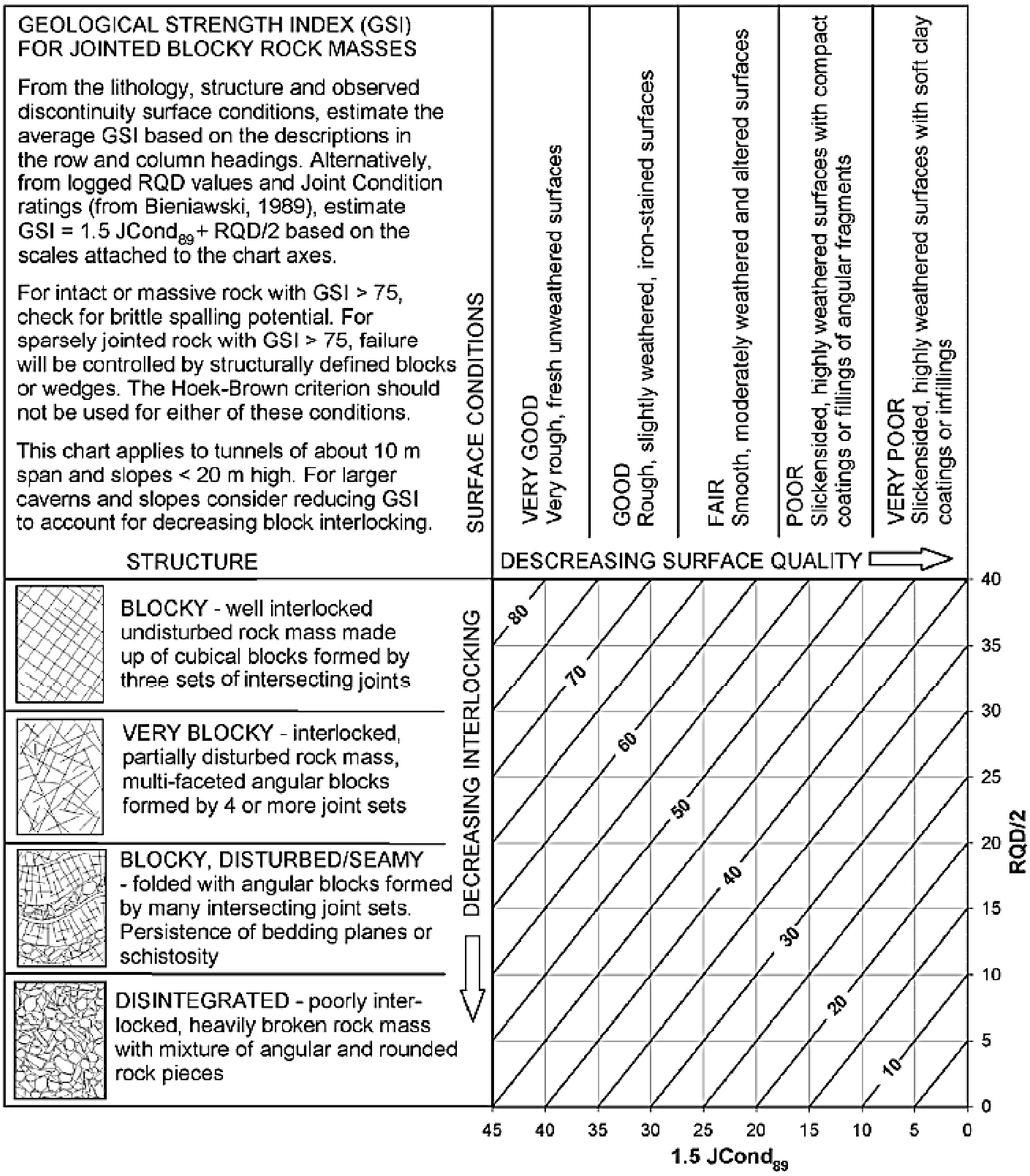
** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

2.2.2. The Geological Strength Index (GSI)

The Geological Strength Index (GSI) system, is a genuine rock mass classification system associated with the rock mass strength and the generalized Hoek-Brown and Mohr-Coulomb failure criteria-based deformation parameters. Standard charts, field observations of rock blocks and discontinuity conditions can be utilised to determine the GSI value. The GSI provides a quantitative statement of the geotechnical quality of a rock mass (Hong et al., 2017).

Hoek and Brown presented the Geological Strength Index (GSI) in 1997 for weak and intact rocks. For this categorization, in the first phase five qualitative categorizations of rock masses were proposed to be intact-massive, blocky, very blocky, blocky disturbed and disintegrated. In addition, five particular surface circumstances were recommended that are comparible with RMR (Rock Mass Rating) (Bieniawski, 1989) which is a discontinuity condition descriptions for rock. Between the years of 1997 and 2013, a classification chart has been updated and laminated-sheared section and a particular chart for rock masses of flysch type have been added to GSI rating system (Yertutanol, 2015). Recently, Hoek et al. (2013) published a paper proposing a method for evaluating GSI using the joint condition rating of RMR system, the Rock Quality Designation (RQD), and the joint condition factor (JCond89) by Bieniawski (1989) (Hong et al., 2017). Hoek et al. (2013) added some quantifications related to jointed rock mass in GSI chart due to the lack of quantifiable parameters describing the discontinuities and the rock mass structures (Figure 2.3) (Yertutanol, 2015).

The GSI identifies the constant parameters for rock mass strength ratings i.e. σ_{ci} and m_i. The m_i is a material constant and σ_{ci} is uniaxial compressive strength of the rock material as they were defined by Hoek et al. (2002). The correlation between the principal stresses at failure for a particular rock is described by these two constants, σ_{ci} and m_i.


GEOLOGICAL STRENGTH INDEX (GSI) FOR JOINTED BLOCKY ROCK MASSES

From the lithology, structure and observed discontinuity surface conditions, estimate the average GSI based on the descriptions in the row and column headings. Alternatively, from logged RQD values and Joint Condition ratings (from Bieniawski, 1989), estimate GSI = 1.5 $JCond_{89}$+ RQD/2 based on the scales attached to the chart axes.

For intact or massive rock with GSI > 75, check for brittle spalling potential. For sparsely jointed rock with GSI > 75, failure will be controlled by structurally defined blocks or wedges. The Hoek-Brown criterion should not be used for either of these conditions.

This chart applies to tunnels of about 10 m span and slopes < 20 m high. For larger caverns and slopes consider reducing GSI to account for decreasing block interlocking.

VERY GOOD Very rough, fresh unweathered surfaces

GOOD Rough, slightly weathered, iron-stained surfaces

FAIR Smooth, moderately weathered and altered surfaces

POOR Slickensided, highly weathered surfaces with compact coatings or fillings of angular fragments

VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or infillings

BLOCKY - well interlocked undisturbed rock mass made up of cubical blocks formed by three sets of intersecting joints

VERY BLOCKY - interlocked, partially disturbed rock mass, multi-faceted angular blocks formed by 4 or more joint sets

BLOCKY, DISTURBED/SEAMY - folded with angular blocks formed by many intersecting joint sets. Persistence of bedding planes or schistosity

DISINTEGRATED - poorly inter-locked, heavily broken rock mass with mixture of angular and rounded rock pieces

Figure 2.3. Quantification of GSI classification system by joint conditions and RQD ($JCond_{89}$ values taken from RMR) (Hoek et al., 2013)

2.2.3. The m_i Value

The m_i value varies with rock type, and it is suggested that this value be specified from a series of triaxial tests (Hoek and Brown 1980a). Some m_i values put forward by Hoek (2007) are given in Table 2.4.

Table 2.4. Updated values of the constant m_i for intact rock, by rock group (Values in parenthesis are estimates) (Adapted from Hoek, 2007)

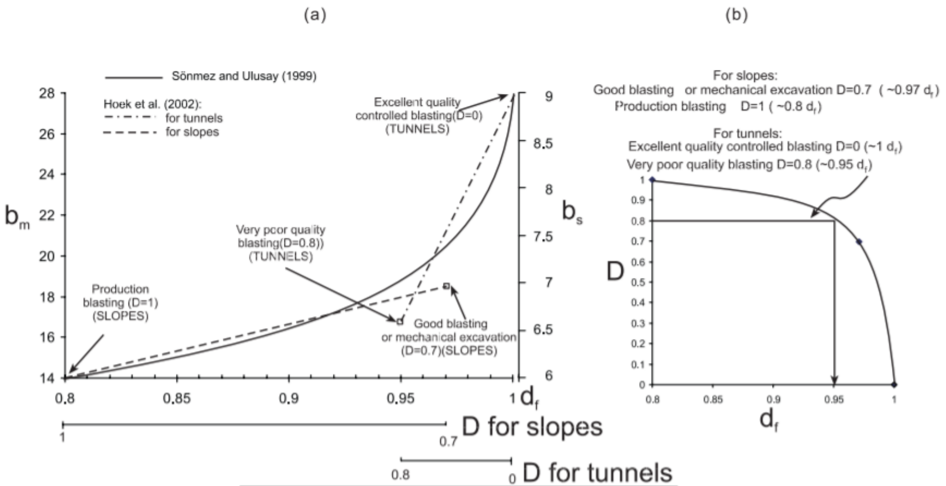
Rock Type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (22±3) Breccias (19±5)	Sandstone 17±4	Siltstone (7±2) Graywacke (18±3)	Claystone 4±2 Shales (6±2) Marls (7±2)
	Non-clastic	Organic				Chalk (7±2)
		Carbonates	Crystalline Limestone (12±3)	Sparitic Limestone (10±2)	Micritic Limestone (9±2)	Dolomites (9±3)
		Evaporites		Gypsum (8±2)	Anhydrite 12±2	
METAMORPHIC	Non-foliated		Marble 9±3	Hornfels (19±4) Metasandstones 26±6	Quartzites 20±3	
	Slightly foliated		Migmatite (29±3)	Amphibolites 26±6		
	Foliated		Gneiss 28±5	Schists 12±3	Phyllites (7±3)	Slates 7±4
IGNEOUS	Plutonic	Light	Granite 32±3	Diorite 25±5		
			Granodiorite (29±3)			
		Dark	Gabbro 27±3	Dolerite (16±5)		
			Norite 20±5			
	Hypabyssal		Porphyries (20±5)		Diabase (16±5)	Peridotite (25±5)
	Volcanic	Lava		Rhyolite (25±5) Andesite 25±5	Dacite (25±3) Basalt (25±5)	Obsidian (19±3)
		Pyroclastic	Agglomerate (19±3)	Breccia (19±5)	Tuff (13±5)	

It is appearently seen in the table that there is a trend where m_i values are relatively high for coarse grained rocks, average for moderate grained rocks, and low for fine grained rocks (Cai, 2010). The m_i value has a span between 4 and 32 for some frequently encountered rocks in engineering applications and an understanding that m_i is only related to rock type can be procured from the table which is not true. m_i value is contingent upon many factors like mineral content, texture and foliation. The estimations are given in Table 2.4 for intact rocks that were tested normal to bedding or foliation in the event such characteristics are available. The Hoek–Brown failure criterion for intact rocks is given in Equation 2.3 (Hoek and Brown 1980a) where σ'_1 and σ'_3 represent the major and minor principal stresses at failure, respectively.

$$\sigma_1^{'} = \sigma_3^{'} + \sigma_{ci}\left(m_i \frac{\sigma_3^{'}}{\sigma_{ci}} + 1 \right)^{0.5} \tag{2.4}$$

2.2.4. The disturbance factor (D)

The disturbance factor (D) is a determinant specified based on the extent of degradation to which the rock mass was experienced by blowing damage and stress relaxation. This determinant is comparable with b_m and b_s, which varies between 0 and 1 for undisturbed in-situ rock and very disturbed rock masses respectively, as previously suggested by Sönmez and Ulusay (1999). Hoek et al. (2002) suggested guidelines for selection of the disturbance factor. The relationships between b_m-d_f and b_s-d_f provided by Sönmez and Ulusay (1999) with the disturbance factor suggested by Hoek et al. (2002) are given in Figure 2.4. It is obvious from Figure 2.4 (a) that the disturbance factor (D) is described by two straight lines and expresses an approximation to d_f -b_m-b_s curve drawn by implementing the equations proposed by Sönmez and Ulusay (1999). Also, the disturbance factor d_f provided by Sönmez and Ulusay in 1999 reviews the effects of different types of disturbance with a continuous relationship between d_f and b_m-b_s and is depending on the explanations for different disturbance effects by Kendorski et al. (1983) for this goal. The relationship between D and d_f is given in Figure 2.4 (b) (Sönmez and Ulusay, 2002).



Sönmez and Ulusay (1999)	
$m_b = m_i \exp\left(\frac{GSI-100}{b_m}\right)$	$s = \exp\left(\frac{GSI-100}{b_s}\right)$

Sönmez and Ulusay (1999)	Hoek et al. (2002)
$b_m = 3.14\ln\left(\frac{d_f}{d_f + 340(1-d_f)}\right) + 28$	$b_m = 28 - 14D$
$b_s = 0.67\ln\left(\frac{d_f}{d_f + 340(1-d_f)}\right) + 9$	$b_s = 9 - 3D$

Figure 2.4. (a) Comparison of the disturbance factors suggested by Sönmez and Ulusay (1999) and Hoek et al. (2002), and (b) d_f -D relationships

2.3. Slope Stability Analysis by Limit Equilibrium Method

Methods based on limit equilibrium have for a long time been the dominating choice to use for determination of the stability and factor of safety (FOS) for slopes. While newer methods to determine slope stability such as the Discontinuity Layout Optimisation (DLO) (Smith and Gilbert, 2007) and finite element-based limit analysis (Sloan, 2012) have been developed, they still do not have the same widespread usage as methods and programs based on the limit equilibrium method (Hernwall, 2017).

Self weights and stabilities against failure under applied forces of all natural or cut slopes are analyzed by limit equilibrium methods based on theory of elasticity. In spite of the differences between these methods in practice, investigation of stability of sliding mass on a known or predicted critical slip surface is common in all. By these methods with an assumption of potential circular or wedge failure, safety factor of the slope is determined by using the connection between the sliding and resistant stresses or forces (Duncan and Wright, 2005). Stability of a slope is dominantly studied by methods of limit equilibrium, and the factor of safety of through which the critical slip surface is designated. Factor of safety is defined as the proportion of the shear strength over the shear stress

$$\text{Factor of Safety} = \frac{\text{Shear strength}}{\text{Shear stress required for equilibrium}}$$

which can be expressed as

$$F = \frac{c + \sigma \tan\phi}{\tau_{eq}} \tag{2.5}$$

where F= factor of safety, c= cohesion, φ = internal friction angle, **s**=normal stress on the slip surface and $\mathbf{t}_{eq}$=shear stress satisfying the equilibrium of the slope.

In limit equilibrium analysis, the potential sliding material in soil or rock mass is divided into a number of slices, and a common limit equilibrium formula (Fredlund et al. 1981; Chugh 1986) is operated for evaluation of safety factor. The equations generated include

- Summation of vertical forces for each slice, in which the outcoming equations are worked out for the normal forces at on the butt of slices,
- Summation of horizontal forces for each slice is utilized to compute the normal forces between slices, where the acquired equations are applied in an integration manner throughout the sliding mass,
- Summation of moments around a common spot for all slices, where the acquired equations can be reorganized and figured out for deriving the moment equilibrium factor of safety (F_m),

- Summation of horizontal forces for all slices, for deriving a force equilibrium factor of safety (F_f).

Despite having the static equations mentioned above, the operation is still uncertain, some more assumptions are required concerning the direction of the resultant interslice forces. The interslice force function is sustained to clarify the direction of the resultant interslice forces. The safety factors for each slices can be evaluated depending on moment equilibrium (F_m) and force equilibrium (F_f). The safety factor may vary based on the proportion of the interslice force function put upon the calculation (Yang, 2005).

2.4. Back Analysis

Back analysis is a practice widely performed in geotechnical engineering to assess the properties of a rock or soil mass. In slope engineering applications, it can be used to analyze a visibly stable slope in order to estimate the minimum operating shear strengths (Brown et al., 2016). Sakurai (1981) defines back analysis as a technique of finding the governing parameters of a system by analyzing the system output behavior. In back analysis of rock structures, strength parameters such as modulus of elasticity, cohesion and internal friction angle are determined from displacement, strain and failure measured during or after construction. Back analysis which is also referred as "reverse" method (Sakurai, 1981), is a method where the force conditions and strength properties are the input for determining displacement, stress and strain, and stability of a structure. The opposite approach to back analysis is the "forward" or "ordinary" analysis (Figure 2.5) (Calderon, 2000).

According to Calderon (2000), in most cases back analysis is the most realistic and representative way of obtaining shear strength parameters, especially if the slope failure parameters are identified reasonably realistically. These parameters are mechanism of failure, slope and slide geometry, groundwater conditions, acting forces at slope failure, displacement, and strains. A series of steps are suggested by Denis da Gama (1981) to perform a back analysis:

1. Input data
    - Define slope and slide geometry
    - Groundwater conditions
    - Acting forces at slope failure
2. Formulation of slope failure model, including its mechanism
3. Stability analysis (limit equilibrium methods, finite element, etc.)
4. Determination of shear strength parameters (Calderon, 2000).

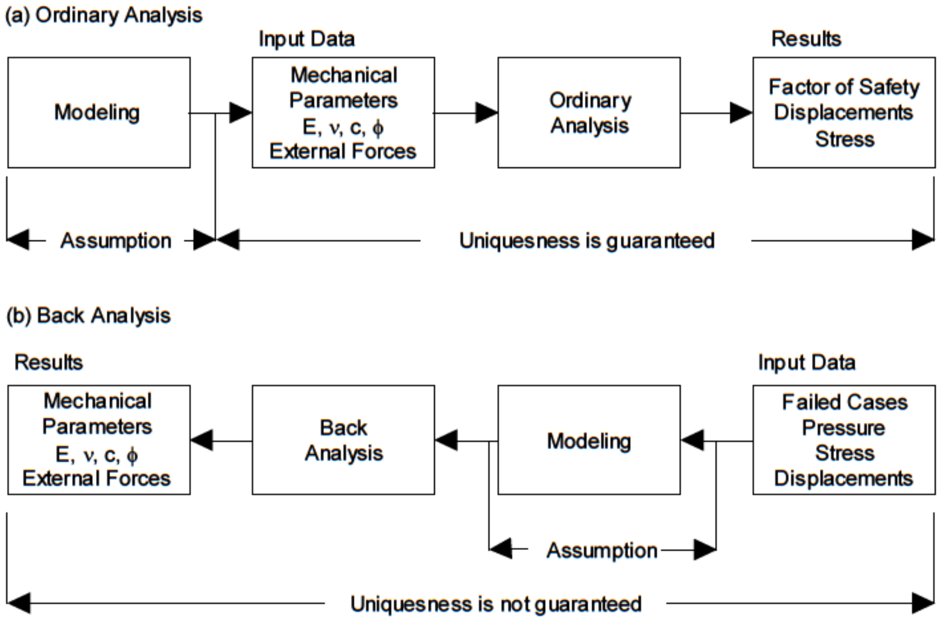


Figure 2.5. Relationship between ordinary analysis and back analysis (after Sakurai, 1981)

The slope stability principles recognize the back analysis as the most reliable method for shear strength determination depending on the actual natural or induced failure events. The back analysis provides practically valuable geotechnical information for the actual failure events, especially that on cohesion and friction angles along the joints. The shear strength parameters (cohesion, and internal friction angle) of the failed material obtained from laboratory and in-situ tests can be deceptive. Back analysis is a more reliable and appropriate procedure to evaluate the mobilized parameters in-situ.

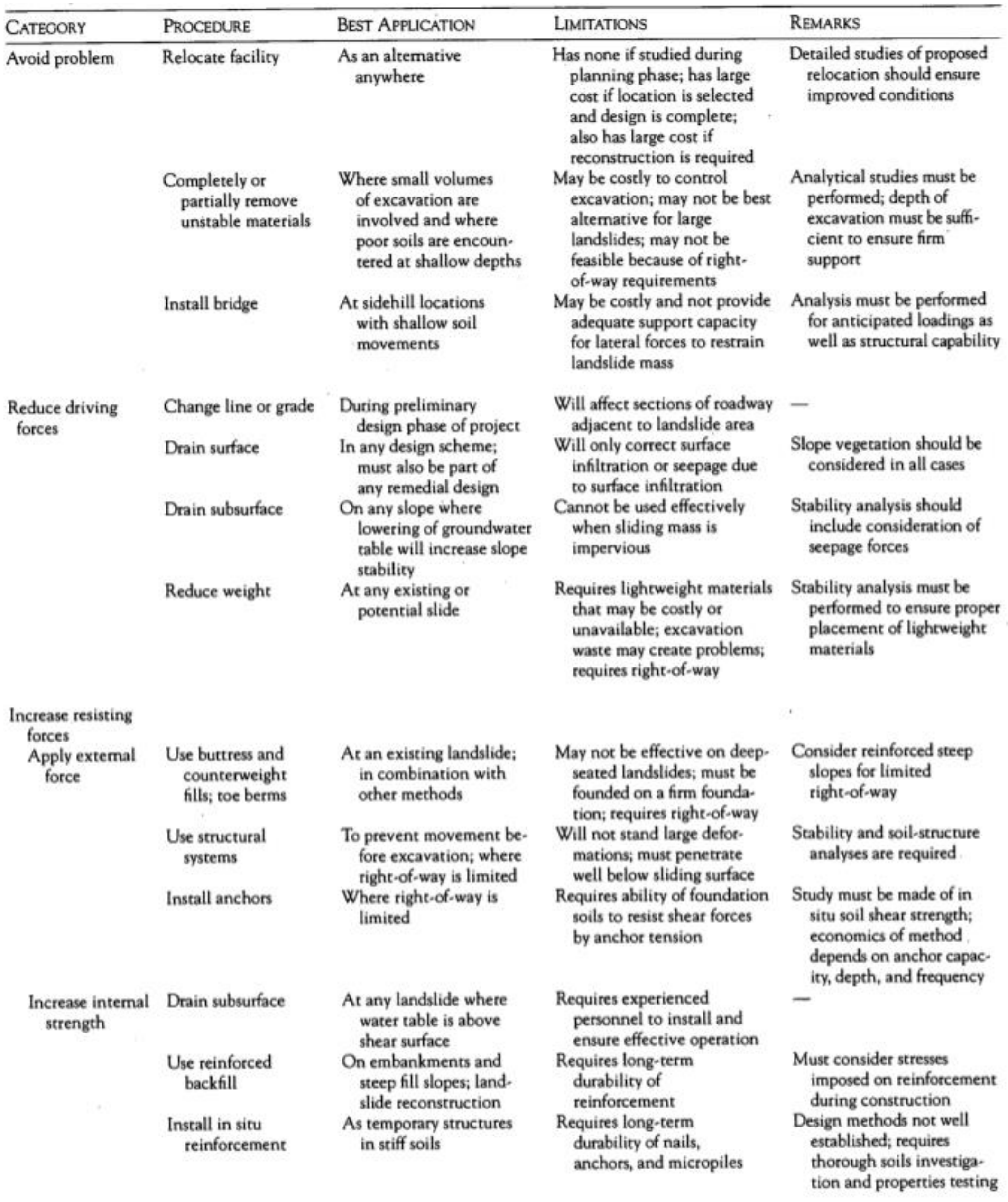
Even if back analysis mostly provides a better shear strength estimation than laboratory tests, still there are some uncertainties. Some of them were explained by Leroueil and Tevenas (1981), Duncan and Stark (1992), Stark and Eid (1998), Gilbert et al. (1998), Tang et al. (1999) and Deschamps and Yankey (2006). Engineering properties of the included materials in the cross-section; slope's geometry, phreatic surface and porewater pressures at the time of failure; effect of rainfall; position of failure surface and existence of tension cracks are some of the uncertainties that affect the back-calculated shear strength (Hussain et al., 2010).

2.5. Remedial Measures for Slope Stability

Remediation of an existing landslide or the repression of a suspensive landslide is a function of a mitigation in the driving forces or an extension in the available resisting forces. All remedial measures will be taken has to include one or both of these parameters (Table 2.5) (Popescu and Sasahara, 2009). Deciding on an appropriate remedial measure turns on:

- engineering feasibility,
- economic feasibility,
- legal or regulatory conformity,
- social acceptability, and
- environmental acceptability.

Table 2.5. Summary of Approaches to Potential Slope Stability Problems (modified from Gedney and Weber 1978)



Category	Procedure	Best Application	Limitations	Remarks
Avoid problem	Relocate facility	As an alternative anywhere	Has none if studied during planning phase; has large cost if location is selected and design is complete; also has large cost if reconstruction is required	Detailed studies of proposed relocation should ensure improved conditions
	Completely or partially remove unstable materials	Where small volumes of excavation are involved and where poor soils are encountered at shallow depths	May be costly to control excavation; may not be best alternative for large landslides; may not be feasible because of right-of-way requirements	Analytical studies must be performed; depth of excavation must be sufficient to ensure firm support
	Install bridge	At sidehill locations with shallow soil movements	May be costly and not provide adequate support capacity for lateral forces to restrain landslide mass	Analysis must be performed for anticipated loadings as well as structural capability
Reduce driving forces	Change line or grade	During preliminary design phase of project	Will affect sections of roadway adjacent to landslide area	—
	Drain surface	In any design scheme; must also be part of any remedial design	Will only correct surface infiltration or seepage due to surface infiltration	Slope vegetation should be considered in all cases
	Drain subsurface	On any slope where lowering of groundwater table will increase slope stability	Cannot be used effectively when sliding mass is impervious	Stability analysis should include consideration of seepage forces
	Reduce weight	At any existing or potential slide	Requires lightweight materials that may be costly or unavailable; excavation waste may create problems; requires right-of-way	Stability analysis must be performed to ensure proper placement of lightweight materials
Increase resisting forces				
Apply external force	Use buttress and counterweight fills; toe berms	At an existing landslide; in combination with other methods	May not be effective on deep-seated landslides; must be founded on a firm foundation; requires right-of-way	Consider reinforced steep slopes for limited right-of-way
	Use structural systems	To prevent movement before excavation; where right-of-way is limited	Will not stand large deformations; must penetrate well below sliding surface	Stability and soil-structure analyses are required
	Install anchors	Where right-of-way is limited	Requires ability of foundation soils to resist shear forces by anchor tension	Study must be made of in situ soil shear strength; economics of method depends on anchor capacity, depth, and frequency
Increase internal strength	Drain subsurface	At any landslide where water table is above shear surface	Requires experienced personnel to install and ensure effective operation	—
	Use reinforced backfill	On embankments and steep fill slopes; landslide reconstruction	Requires long-term durability of reinforcement	Must consider stresses imposed on reinforcement during construction
	Install in situ reinforcement	As temporary structures in stiff soils	Requires long-term durability of nails, anchors, and micropiles	Design methods not well established; requires thorough soils investigation and properties testing

Engineering feasibility includes examination of geological and hydrological conditions at the site to assure the physical sufficiency of the remedial measure. A generally missed out aspect is making sure the system will not misguide the problem in somewhere else. Economic feasibility takes the cost of the remedial action into account considering the advantages it provides. Those advantages include postponed maintenance, absistence from damage including loss of life, and other substantial and moral advantages. Legal-regulatory conformity serves for the measure encountering local building rules, avoiding annoying other property owners, and other concerned factors. Social acceptability is an extent to which a remedial measure is reasonable for the society and neighbours. Some measures can prohibit further damage but can be an eyesore to neighbours. Environmental acceptability refers to the necessity of the remedial measure not to affect the environment adversely (Popescu and Sasahara, 2009).

2.5.1. Drainage

Drainage is important remedial measure since it plays an important role in reducing shear strength by means of pore-water pressure. Drainage of surface water and groundwater is the widely applied, and usually the most accomplished stabilization method due its high stabilization efficiency compared to its cost. Still, in the long run, it is on the rack because the drains necessitate maintainance in order to continue to function (Bromhead, 1992).

Surface waters are canalized from unstable slopes via ditches and pipes (Figure 2.6) while groundwater drainage is usually provided by networks of trench drains. On the other hand, drainage of the failure surfaces is ensured by counterfort or deep drains that are trenches embedded into the ground in a way that will intersect the shear surface and go below it. For deep landslides, an efficient way of dropping groundwater level is driving drainage tunnels into the formation below the landslide. On these tunnels, numerous drainage holes directed upwards can be drilled to drain the landslide (Popescu, 2002).

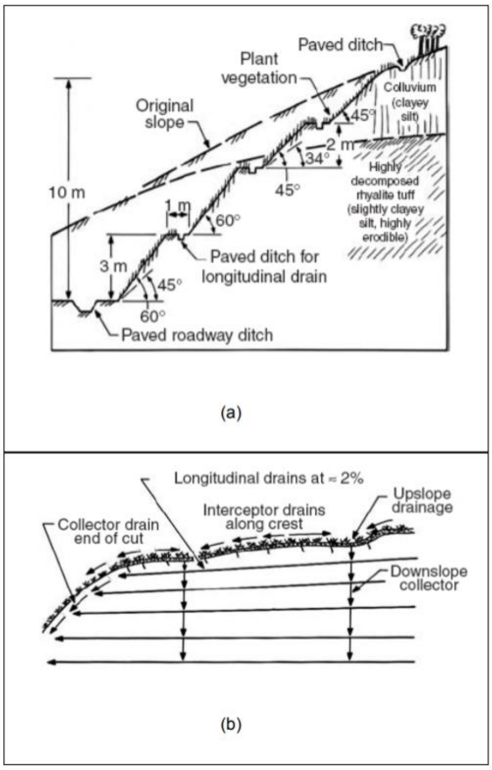


Figure 2.6. a. Benching scheme for cut (low benches permit maximum inclination to reduce the effect of runoff erosion), b. Longitudinal and downslope drains (Hunt, 2005)

2.5.2. Modification of Slope Geometry

Changing the geometry of a slope is the most influential method especially in deep seated landslides (Popescu, 2002). Slope geometry can be changed by methods like excavation, filling, or both. Stability of the slope can be enhanced by lowering the height or inclination of a slope (Ataş, 2017). The major construction work involved in modification of slope stability is excavation and disposal of material from the slope. In order to improve stability by excavation, it is necessary to sacrifice some of material from top of the slope, provide a site that is suitable for the entry of essential equipment and provide available area for accumulation of the excavated material (Duncan and Wright, 2005) (Figure 2.7).

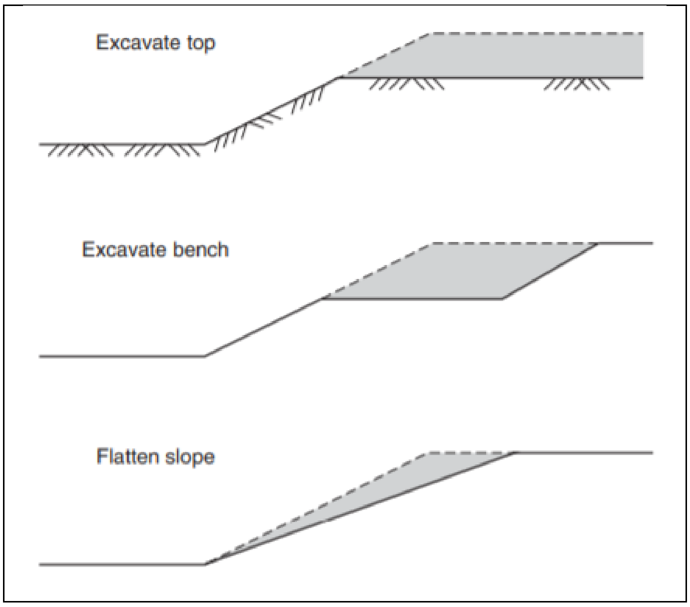


Figure 2.7. Slope stabilization by excavation (Duncan and Wright, 2005)

The most common method of enhancing the stability of a soil cut slope was by cutting the slope to a constitute a gentler profile before 1990 (Koirala and Tang, 1988). Another way to provide a stable cut slope is changing slope geometry by means of forming benches on the slope. Other than decreasing the slope angle of the whole slope, a number of small steps can be formed, thus enhancing the safety factor, thereby minimizing the effect of erosion. This method is mainly impressive in reducing the occurence of relatively small failures but are not very successful in improving the slope stability of larger failures where other methods are recommended. Forming benches are effective in protecting structures beneath rockfall-prone cliffs, governing surface drainage and for providing a construction site for building other structures (Highland and Bobrowsky, 2008). According to the Technical Specification of the

General Directorate of Highways, a slope with a height of 10 m with barrels 3 to 5 m width are accepted as a standard practice (Ataş, 2017).

Since special engineering techniques are not required, the construction cost for modification of a slope is economical and easy to perform. Still, there are cases in which this application is not easy to adjust. Those are wide landslides where there is no appearent toe and/or crest, circumstances in which the geometry is specified by engineering restrictions, presence of critical conditions of failure and an abrupt change in topography (Popescu, 2002).

2.5.3. In-situ Systems

The reinforcement of existing soil masses are provided by in-situ methods. These methods are soil nailing, soil anchors, root piles, micropiles and pin piles (Holtz and Schuster, 1996).

Consisting of highly durable steel reinforcing bars dipped into the ground in the form of drilling and grouting, soil nailing was firstly offered in Hong Kong in 1980s (Figure 2.8). The process of soil nailing is not favorable for all weather conditions and there is necessity to place the soil nails avoiding from the trees. The method is easy and practical compared to the other structural solutions, making it adaptable to most cases widely encountered in slope sites. Additionally, since soil nails can be installed closely, they have capability to reduce the fragility of the slope bearing unforeseen geological zones and undesirable joints by holding the soil together to occupy an integral mass. Thereby, the design of cut slopes via soil nailing is less delicate against adverse ground conditions. After the designing and the construction by Watkins and Powell (1992), the soil nailing method was accepted as a solid and economical engineering solution for the improvement of the cut slopes’ stability in Hong Kong. In addition, upon theoretical studies and field observations, soil nailing is still accepted to be a more robust and reliable application than cutting back since it is more competible to local geological incidents (Choi and Cheung, 2013).

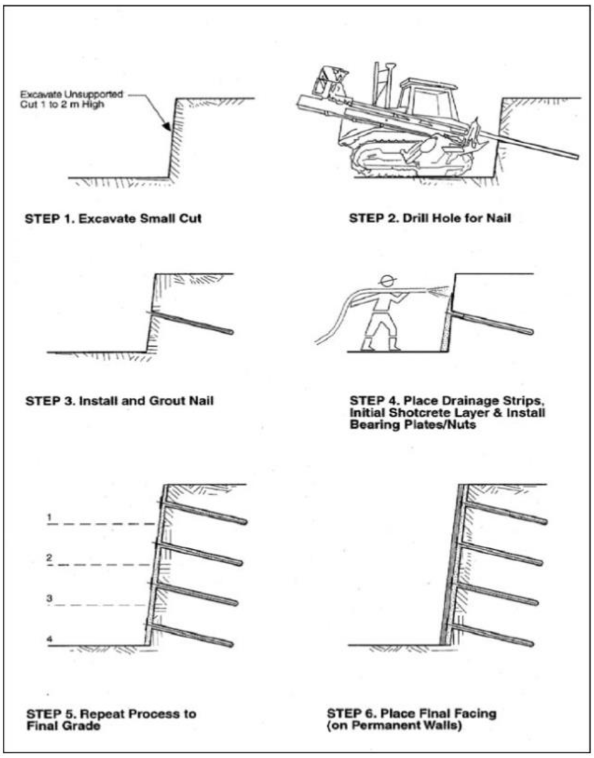


Figure 2.8. Construction sequence of soil nailing (Byrne et al., 1998)

Anchoring in rock or soil is a construction procedure where prestressed components are embedded in the ground (Figure 2.9). The anchors are dipped into boreholes drilled, and are fixed at the end. Following stabilization, the anchors are typically prestressed and their outer ends are fixed to heads. Anchor heads are attached to a structure such as a plate, slab, bar, grid or another structural component that distributes the stress induced by heads onto the wider surface of soil or rock. Anchoring process into the ground fulfills three basic procedures such that:

- It constitutes forces acting on the element in a direction towards the point of contact with the soil or rock.

- It constitutes a reinforcement of soil or rock medium through which the anchor passes wherever non-prestressed anchorage is used.
- It constitutes a prestressing on the anchored structure, while the anchors are passing through the structure.

Anchoring is always accompanied by prestressing of the rock. In this process the ground is consolidated, strengthened and its mechanical characteristics improved. Anchorage, is a process of integrating the structure together with ground mass, which makes it possible to choose with comperative ease based on load centre of the anchoring forces, static analysis, magnitude and direction. The forces, implicated into the entire system acting on the structure, provide the stability of the system with the economy and efficiency. In this way, anchorage protects the structure against vertical displacement by virtue of uplift, turning over, tangential displacement along the toe, shear failure along the critical surface within the underlying strata and also against seismic effect. The continued effectiveness of anchors can be checked easily, and the static mechanics of anchoring forces is straightforward. Anchorage can therefore be regarded as an efficient construction method (Hobst and Zajic, 1983).

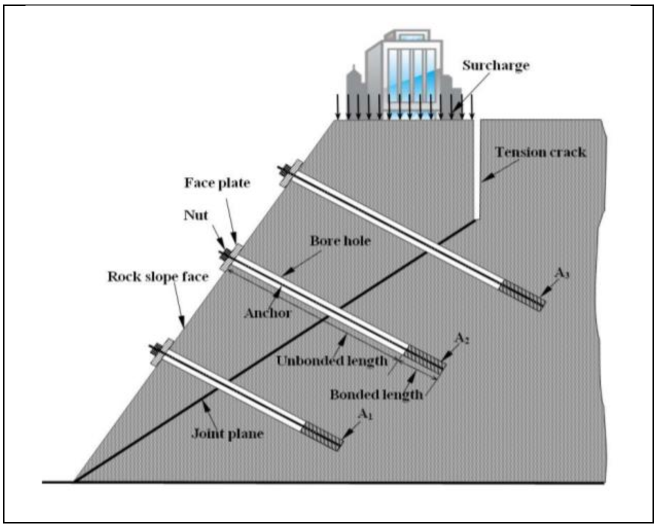


Figure 2.9. A single-directional anchored rock slope (Hossain, 2011)

Another method of accommodation of soils and soft rocks is application of root piles, known as pin piles or micropiles. Root piles are reinforced concrete piles built in-situ with a diameter of 7.5 to 30 cm. For smaller diametered piles, the insertions are applied with a central reinforcing rod or steel pipe, while the ones having larger diameters are applied by a reinforcing bar cage surrounded with spiral reinforcement. Root-pile system is an integrated block of reinforced soil which extends below the critical failure surface. Root piles are strongly influenced by their three-dimensional, rootlike geometric layout in contrast to soil nailing (Holtz and Schuster, 1996).

2.5.4. Application of External Force

Application of external forces in order to enhance the resistance of a slope to potential movements raises the slope stability (Figure 2.10). These resisting forces are usually applied on the toe of a mass having a potential to mobilize via various methods, including:

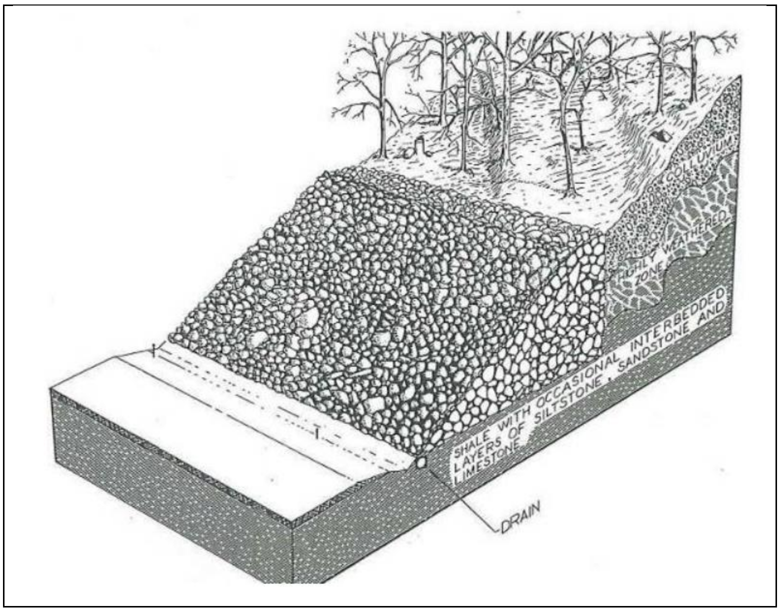


Figure 2.10. Coarse grained or rock buttress for slope stabilization (Gedney and Weber, 1978)

- Buttresses, counterweight fills and toe berms,
- Structural sealing systems like cantilever and gravity retaining walls, externally braced walls or walls supported by anchors/tiebacks, soil nailing, root piles, conventional piles and drilled shafts
- Reinforced soil systems.

Throughout the opening part of the post-war, landslides were generally accepted to be engineering problems necessitating engineering solutions including remediation by using of structural methods. The structural approach originally concentrated on construction of retaining walls but later is varied to comprise a number of more elaborate techniques i.e. passive piles and piers, reinforced concrete walls and reinforced earth retaining structures (Popescu and Sasahara, 2009). If designed and constructed neatly, these structural solutions are remarkably prospering, specifically in areas having a high failure potential or in restricted sites. Yet, remediation by means of structural solutions mostly lead to taking on so pricy precautions that are not appropriate compared to some other solutions such as modification of slope geometry or drainage (DOE, 1994).

Recently, there is a visible tendency towards "soft engineering" where classical non-structural remediations including drainage and slope geometry modification, lime and/or cement stabilization, grouting and soil nailing are applied widely. Also, the non-structural remedial measures is notably economical compared to that of the structural solutions. In other respects, structural solutions like retaining walls includes the risk of digging the slope for construction and mostly necessitate to form steep cuts temporarily. All such operations trigger increment of infiltration after rainfall and the risk of failure and/or over-stepping during construction. Although, being a non-structural solution, the use of soil nailing does not require to open or modify the slope from its present geometry (Popescu and Sasahara, 2009).

CHAPTER 3

GEOLOGY

3.1. Geology of Study Area

The geology of the study area is discussed in terms of the formations observed around the project line. They are Orta Anadolu (Central Anatolian) granitoid (Kog), İncik formation (Toi), İç Anadolu group (Ti) and alluvium (Qal) in chronological order. The generalized columnar section and geological map of the study area is given in Figures 3.1 and 3.2.

3.1.1. Orta Anadolu (Central Anatolian) Granitoid (Kog)

The unit was studied under the name of Yozgat Magmatics by Erdoğan et al. (1996) and Central Anatolian Crystalline Massive by Akçe (2003). Granitoids in Central Anatolia mainly consist of granite, porphyry granite, granodiorite, porphyry granodiorite, quartzdiorite and porphyry quartzdiorite (Dönmez, et al., 2008). According to Güleç and Kadıoğlu (1998) and Kadıoğlu and Güleç (1999), K-feldspar megacrysts, mafic microgranular enclaves and abundance of mafic minerals are common features in H-type central Anatolian granitoids which are dominantly interpreted to result from magma mixing processes resembling Barbarin (1990)'s H-type granitoid series. H-type granitoids are accepted to be illustrative for Central Anatolia's late stage Alpine magmatism (Göncüoğlu et al., 1997; Köksal et al., 2001). H-type granitoids are outputs of the crustal thickening generated from arc to arc and/or arc to continent collision (Göncüoğlu et al., 1992 and 1993, Kaymakçı et al., 2009).

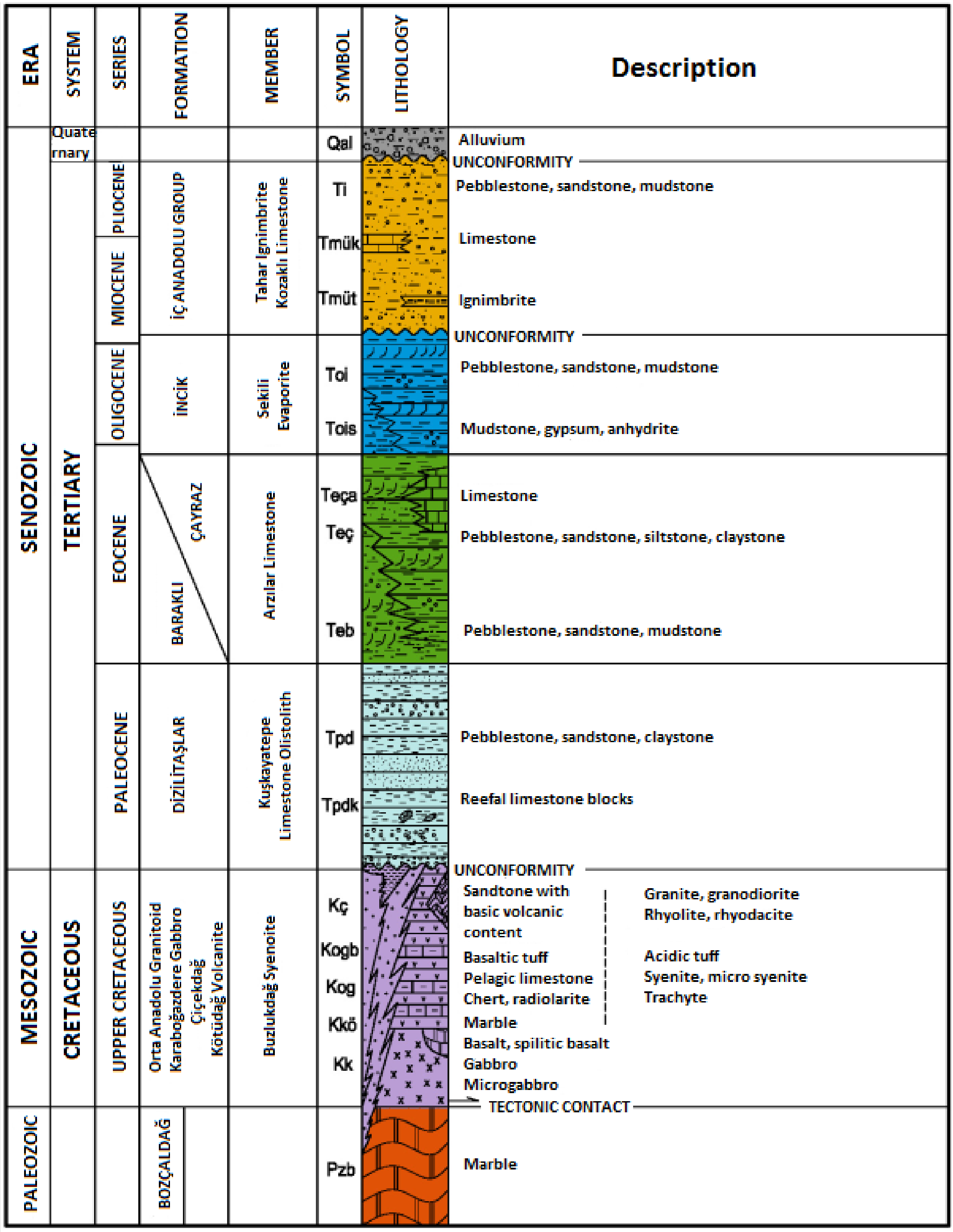


Figure 3.1. Columnar section of the study area (Source: 1/100 000 Scaled Kırşehir - İ 31 Map Section of MTA)

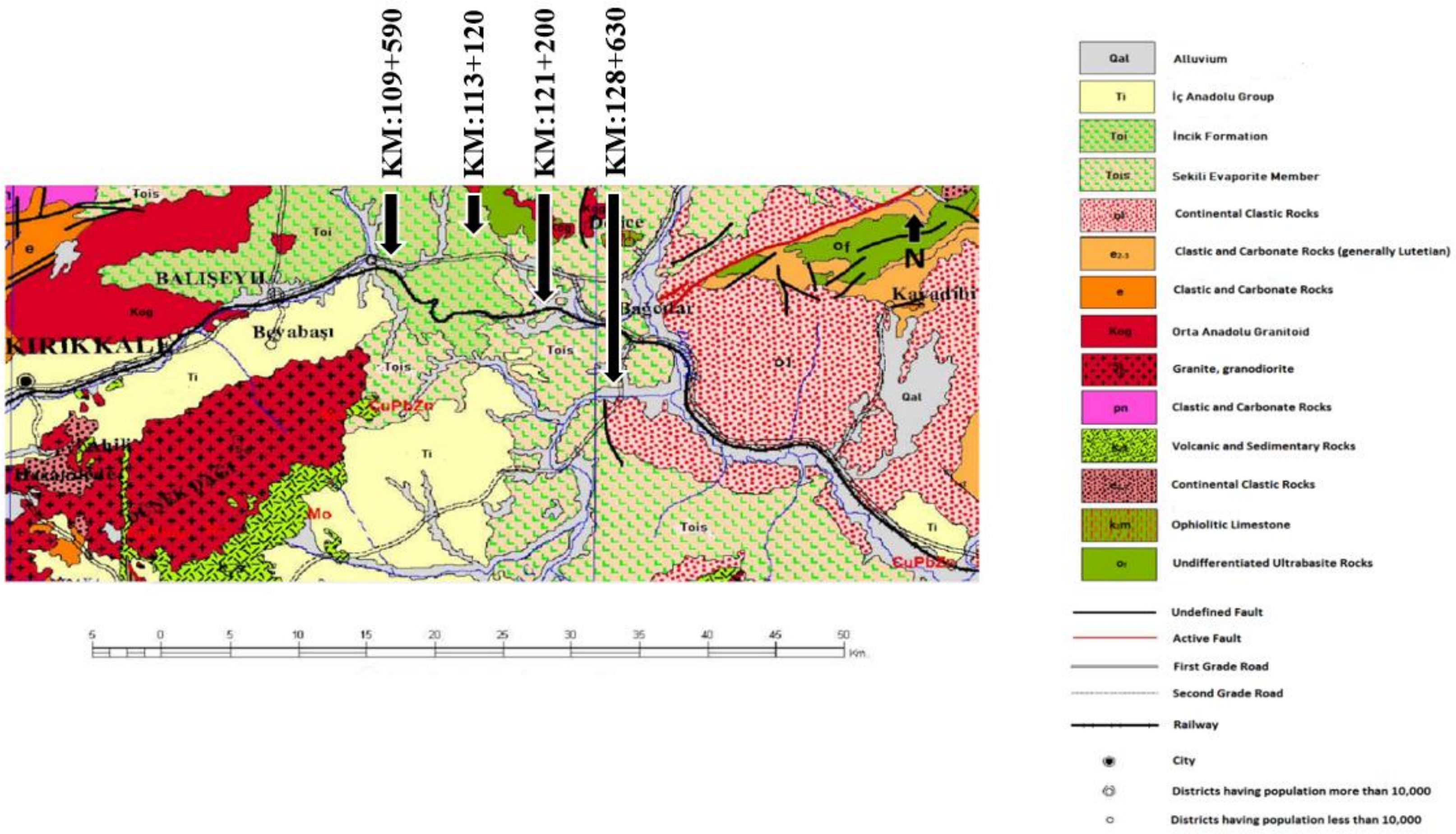


Figure 3.2. Geological map of the study area (Adapted from http://www.mta.gov.tr)

By radiometric aging technique, age of the formation was found to be 54 my (Eocene) (Ayan, 1963) and 71 my (Upper Cretaceous) (Ataman, 1972). The granitoids of Central Anatolian Crystalline Complex (CACC) fall into the metamorphic and ophiolitic rocks of the CACC and are nonconformably overlaid by the Upper Paleocene to Oligocene basin and Neogene-Quaternary units (Gülyüz, 2009).

Orta Anadolu Granitoid is observed at the KM:85+000-88+100 segment of the foreseen high speed railway line (Figure 3.3). At this beginning part of the line, the formation is monitored to be fine-grained and altered. At these parts, feldspars and kaolinized metallic minerals (i.e. pyrite, manganese) are monitored. Towards the end of the railway line (KM:153+725), advanced arenization is monitored locally in the granitic rocks.

Figure 3.3. General view of Orta Anadolu Granitoid

3.1.2. İncik Formation (Toi)

İncik formation is one of the most widespread units monitored in the study area. The formation was firstly named by Birgili et al. (1974). İncik formation is composed of continental red clastics with a thickness of more than 2000 meters (Figure 3.4).

The exact age of İncik formation is unknown due to lack of fossils (Kaymakçı et al., 2009). Age of the formation is accepted to be Upper Eocene-Oligocene based on its stratigraphic position (Dönmez, et al., 2008). Lower part of İncik formation is composed of normal/well-sorted fine/medium/coarse bedded sandstone and alternating gypsum and mudstone. Middle and upper parts of the formation are composed of cross-bedded conglomerate and sandstone alternating with mudstone (Yüksel Proje, 2011a). İncik formation unconformably overlies Orta Anadolu Granitoid and conformably overlies Lutesian aged Kocaçay formation (Birgili et al., 1975) and Çadırlıhacıyusuf formation that is the counterpart of Çayraz formation (Kara and Dönmez, 1990). İncik formation is unconformably covered by Late Miocene-Pliocene aged İç Anadolu Group's deposits (Savaş and Korkanç, 2010) (Figure 3.1).

In the study area (around Balışeyh), the İncik formation outcrops at places that are not covered by alluvium. In the interval of KM:115+090-115+202 of the railway line, Sekili evaporite (Tois) member of İncik formation is also observed (Figure 3.2 and 3.5). Sekili evaporite (Tois) member was firstly defined by Kara (1991). The member is composed of intercalations of red, brown, gray, white and green colored medium-thick bedded gypsum and mudstone (Kürçer, 2012).

Figure 3.4. General view of the İncik formation around the railway route

Figure 3.5. Gypsum specimen exposed in the study area at KM: 109+590 cut slope

Gypsum and anhydrates wedge and fine away while mudstone thickens laterally. Mudstone is thin-medium bedded and multicolored i.e. shades of red, light green and green (Güler, 2011). Sekili evaporite (Tois) member conformably overlies the Çayraz formation (Kara and Dönmez, 1990) and unconformably overlain by İç Anadolu Group deposits. Thickness of the member reaches up to 600-700 meters around Çankırı-Çorum Basin (Birgili et al., 1975). Deposited at the initiative regression of evaporitic environment, Sekili evaporite member contains no fossil and the age of the formation is accepted to be Upper Eocene-Oligocene based on its stratigraphic position (MERS, 2017) (Figure 3.1).

3.1.3. İç Anadolu (Central Anatolia) Group (Ti)

Terrestrial facies in Central Anatolia i.e. Middle Miocene-Pliocene aged stream, alluvial fan and lacustrine deposits are grouped under İç Anadolu Group (Kara and Dönmez, 1990). Being deposited under terrestrial environment, the formation consists of conglomerate, sandstone, siltstone, marl and claystone (Figure 3.6). The formation is dominated by conglomerates at the base and continues with sandstone, marl and claystone sequence towards the top. The formation is generally greenish gray colored and thin-medium bedded with a thickness around 200 meters (Evcimen, 2011). The parts that form fluvial facies are made up of reddish brown, cross-bedded conglomerate, sandstone and mudstone bands and lenses. The parts that are represented by lacustrine facies of mid-basin forming the uppermost part of the unit are composed of either unconsolidated to semi-consolidated sandstone, mudstone, gypsum and anhydrite or conglomerate, sandstone, mudstone, limestone and ignimbrite sublevels. Deposits belonging to the İç Anadolu Group overlies pre-Miocene aged rocks with unconformity. The group is covered by Quaternary clastic deposits by angular unconformity (Akın and Çiftçi, 2011).

Figure 3.6. General view of İç Anadolu Group deposits

According to investigations done by researchers (i.e. Kara and Dönmez, 1990; Métais, et al., 2016) on the basis of fossils within it, age of the group is estimated to be Upper Miocene-Pliocene. Kömişini formation (Uğuz, et al., 1999), Kızılbayırtepe formation (Umut et al., 1990), Kuşça formation (Uygun, 1981), Cihanbeyli formation (Akarsu, 1971) and Bozkır formation (Yılmaz, 1973) can be stated as equivalent of İç Anadolu Group (Dönmez et al., 2005).

3.1.4. Alluvium (Qal)

Quaternary aged alluvium is the youngest deposit in the study area. The alluvium is composed of loose gravel, sand, silt and clay formed by deposition of the material that is carried through recent stream beds. Outcropping at old and contemporary stream beds, alluvium is reddish-green, dirty white, dirty yellow colored, griseous, poorly-sorted, narrowcasting, locally blocky, pebbly, sandy and silty. The blocks and the conglomerates in alluvium consist of gabbro, diabase, serpantinite, andesite, bazalt, pebblestone, sandstone, claystone and mudstone originating from formations with different age and characteristics (MGS, 2016).

Dominantly composed of conglomerate, sand, silt and clay, Quaternary deposits are widely observed around Kızılırmak and Çoruhözü Valley. Conglomerate inclusion in the deposit is dominant around Çoruhözü and especially Kırıkkale-Aşağı Mahmutlar area. Clayey and silty units predominate around Kızılırmak valley and Balışeyh where the deposits are gray in color, porous and have a thickness of 25-30 cm (Sönmezer, 2016).

3.2. Structural Geology of the Study Area

Within the project area including the study area, the marbles belonging to Bozçaldağ formation forming the uppermost part of the Kırşehir Massive occupies the basement (Figure 3.1). This formation is overlain by Santonian aged basic magmatic-volcanic-volcanoclastic rocks and extrinsic pelagic deposits of Karaboğazdere gabbro and Çiçekdağı formation tectonically in the southern part of the study area. Marbles belonging to Bozçaldağ formation and Çiçekdağı formation are cut by Santonian aged deep marine environment rocks like granite, granitoid, syenite and Orta Anadolu Granitiods. These rock units are overlain by Tertiary deposits. Tertiary deposits are represented by turbiditic Paleocene-Early Eocene aged Dizilitaşlar deposits formed in deep marine environment. Eocene aged Baraklı and Çayraz formations deposited in terrestrial and shallow deep marine environment, and Upper Eocene-Lower Miocene aged İncik and Miocene-Lower Pliocene aged İç Anadolu Group deposits (Figure 3.1).

No tectonic lines are observed in the study area and in the vicinity at regional scale. Still, the frequently observed folded structures formed in the İncik formation proves that it is under compressional regime, where compression trend is almost E-W (Yüksel Proje, 2011a). Also, the faults in İncik formation are usually in the form of narrow zones bearing no large scaled damage zones, formed just as minor faults, cleavages, or tension gashes are sparse. On the other hand, crossbedding is encountered within red conglomerate-sandstone alternation of the İncik Formation (Tokay, 2015).

3.3.Seismicity of Study Area

According to the latest Seismic Earthquake Hazard Map of Turkey, study area falls into a range between low and high hazard zone. Seismic load coefficient can be figured out by an interactive map of General Directorate of Natural Disasters (AFAD) (Figure 3.7). The seismic load coefficients came up to be around 0.2g at the beginning (1), center (2) and end (3) points of the railway route in study area (Figure 3.8). Pseudostatic method is used throughout this study which is an approach used to evaluate the seismic stability of earth structures (Kramer, 1996). Commonly, the vertical seismic force is assumed to be zero ($k_v = 0$) and only the horizontal force is considered in the analysis (Ghobrial et al., 2015). According to Hynes-Griffin and Franklin (1984), the horizontal coefficient (k_h) equals to 0.5 PGA (peak ground acceleration) providing a factor of safety greater than 1 and a strength reduction of 20%. Accordingly, horizontal seismic load coefficient was accepted to be 0.1g throughout the analyses.

The ancient and recently measured earthquake records indicate that the eastern section of Central Anatolia is less active when compared to other parts of Anatolia seismically

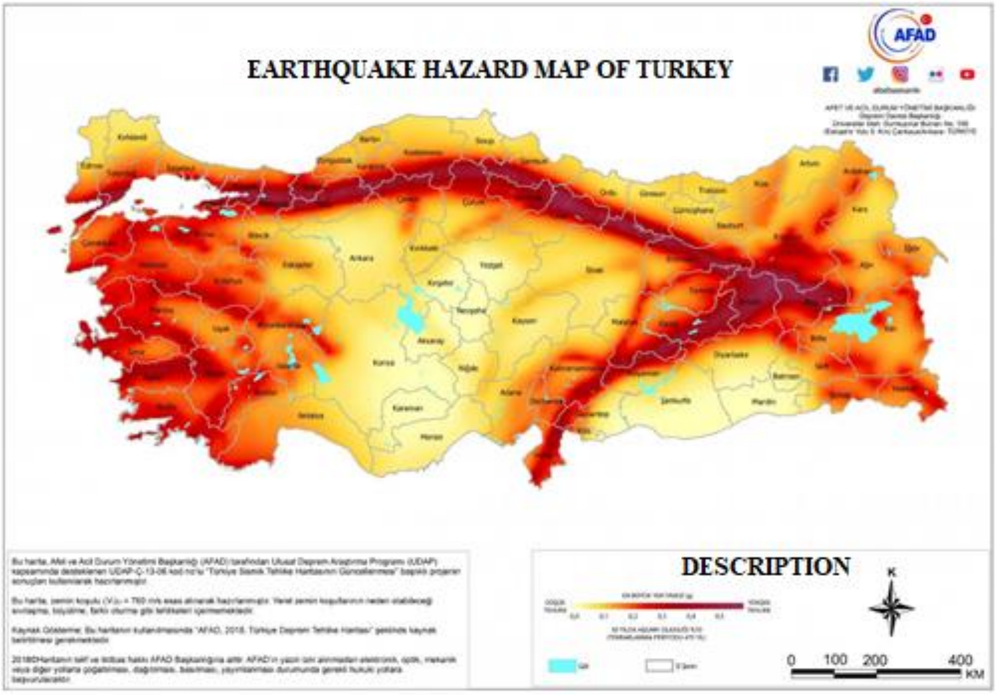
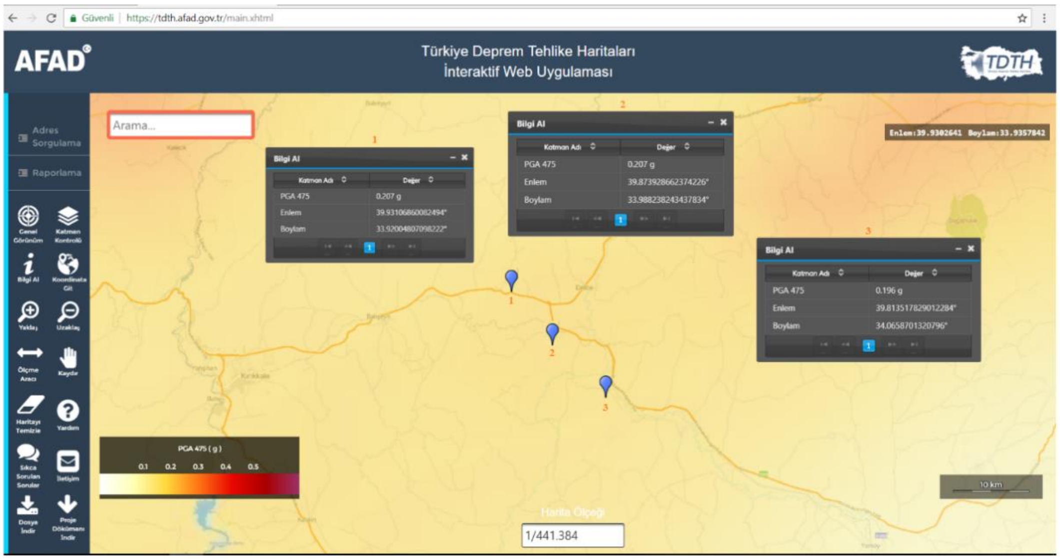


Figure 3.7. Earthquake hazard map of Turkey (Adapted from http://www.afad.gov.tr)



Enlem:39.9302641 Boylam:33.9357842

1 — Bilgi Al

Katman Adı	Değer
PGA 475	0.207 g
Enlem	39.93106860082494°
Boylam	33.92004807098222°

2 — Bilgi Al

Katman Adı	Değer
PGA 475	0.207 g
Enlem	39.873928662374226°
Boylam	33.988238243437834°

3 — Bilgi Al

Katman Adı	Değer
PGA 475	0.196 g
Enlem	39.813517829012284°
Boylam	34.0658701320796°

Figure 3.8. Seismic zone map of the study area (http://www.afad.gov.tr)

(Ambraseys and Jackson, 1998). Contrary to the eastern part, the western part is dominated by a sets of NE-SW and NW-SE trending cross-graben and horst structures enclosed by oblique-slip normal faults with strike-slip components. This area is a zone of transition between the extending western Anatolia and the eastern Central Anatolia where is dominated by a strike-slip faults (Dwivedi and Hayashi, 2010).

Far and wide, the study area is surrounded by North Anatolian Fault System, Salt Lake Fault Zone, Central Anatolian Fault Zone and İnönü-Eskişehir Fault Zone. The area located at northern, northeastern and eastern parts of this system has a tectonic zone (with normal component) characterized by strike-slip faults. However, the area at western, southwestern and southern parts of this system has an extensional neotectonic zone characterized by oblique-slip normal faults (Sönmezer, 2016) (Figure 3.9).

Looking under the hood, the study area is surrounded by Keskin Fault Zone, Karakeçili Fault Zone from southwest, Çankırı Fault at north and Kırıkkale-Sungurlu Fault Zone from northeast (Figure 3.10). The earthquake with a magnitude of Mw=6.6 occured along Akpınar Fault Zone 50 km away in 1938 is one of the earthquakes indicating the seismic activity around the study area.

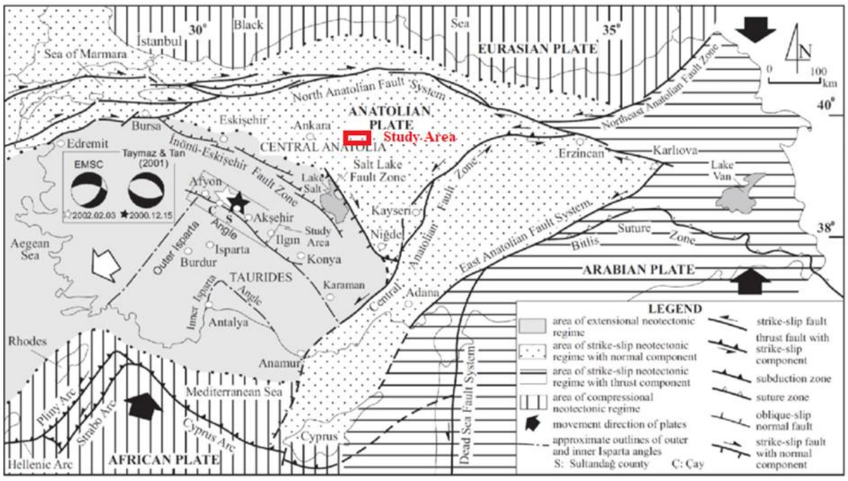


Figure 3.9. Simplified map of neotectonic structure around study area (Adapted from Koçyiğit and Özacar, 2003)

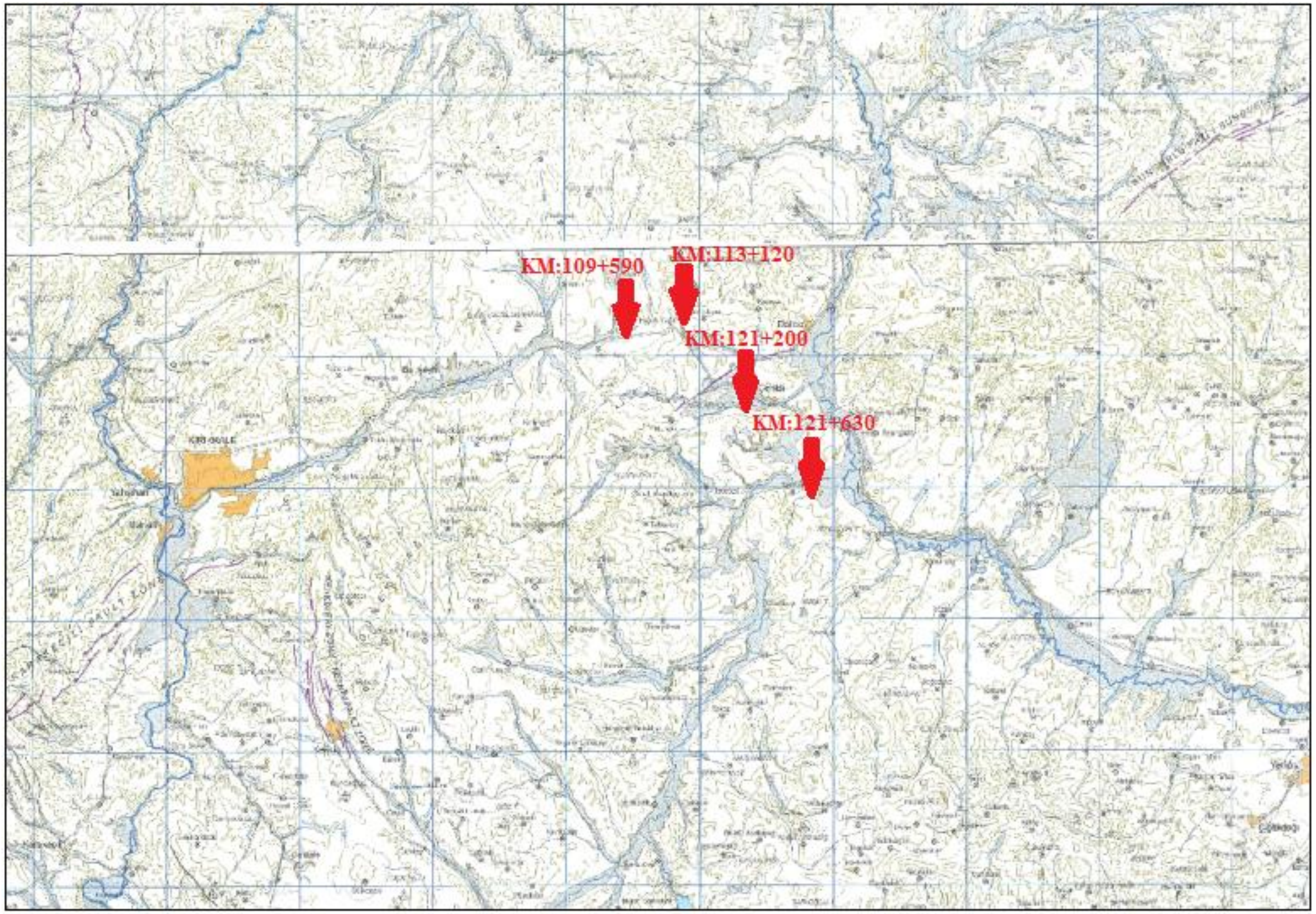


Figure 3.10. Faults in the vicinity of the study area (from http://www.mta.gov.tr)

3.4.Hydrogeology

At time of geological mapping, hydrogeological properties of soil and rock units forming railway route and groundwater conditions were studied broadly and the results obtained were evaluated based on foundation boring and test pit data.

Occupying most of the study area, the İncik Formation (Toi) is dominantly made up of claystone, siltstone and conglomerate. Claystone and siltstone are practically where sandstone and conglomerate are permeable. Eventually, in all the borings carried out in these formations, groundwater is encountered (Table 3.1). The granite which is Orta Anadolu Granitoid’s (Kog) member exhibits a semi-permeable to permeable structure. Alluvial deposits (Qal) observed at valley floor along the high speed railway line include groundwater depending on their structural characteristics. Sandy and gravelly levels of the formations have groundwater bearing capacity.

Table 3.1. Groundwater data for cut slopes with failure

Drilling No	**KM**	**Coordinates (ITRF96-TM33)**			**Depth (m)**	**Groundwater Level (m)**
		X	**Y**	**Z**		
KY-107	109+871	4 421 308.00	575 572.00	905.91	15.45	9.50
KY-115	113+290	4 422 047.00	581 344.00	793.98	21.45	7.30
KY-128	120+659	4 415 771.00	584 534.00	698.60	12	3.5
KY-135	128+457	4 410 611.00	590 361.00	666.26	10.50	0.80

CHAPTER 4

CHARACTERISTICS OF THE FAILED-CUT SLOPES

4.1. Acquisition of Engineering Geological Properties of the Failed Cut Slopes

The failed-cut slopes excavated along the high-speed railway route consist of various rock types as it was explained in Chapter 3 - Geology. The engineering geological characteristics of the cut slopes were specified and quantified based on field observations and laboratory and in-situ tests. Since all the failed-cut slopes within the scope of this thesis were excavated on İncik formation; field observations, the test results on exploratory drillings, test pits and SPT tests conducted on this formation were evaluated within the scope of this study.

4.1.1. Exploratory Drillings and Test Pits

A sum of 97 exploratory drillings with a total length of 2391,94 meters were planned throughout the railway route where four of them were drilled at close vicinity of the failed-cut slopes in the study area (Table 4.1). Boring logs (KY-107, KY-115, KY-128, KY-135) and pictures of drilling samples taken from the failed-cut slopes in the study area are given at Appendices A and B. Also, a total of 88 test pits were planned throughout the railway route where four of them were excavated at close vicinity of the failed-cut slopes. The four test pit logs (KYGÇ-71, KYAÇ-35, KYAÇ-42, KYGÇ-87) taken from the vicinity of the failed-cut slopes are given at Appendix C.

In order to investigate the characteristics of the geological formations in the study area, disturbed (SPT), undisturbed (UD) and core samples were taken from the boreholes. Wet unit weight, liquid limit, plastic limit, plasticity index, water content, density and grain size distribution in addition to point load strength index, uniaxial compressive strength and shear strength parameters (c’ and φ’) were specified according to the results of the tests applied on the samples taken from the drillings.

Hydrogeology of the study area was also examined based on exploratory drilling data. After completion of drilling process, PVC pipes were let down the wells in order to measure the depth of groundwater periodically. Exploratory drillings were conducted in accordance with Technical Specifications in Exploratory Engineering Services of General Directorate of Highways (KGM, 2005). İncik formation was defined to have water in sandy and gravelly levels.

4.1.2. In-situ Testing

4.1.2.1. SPT

In the course of drilling processes, a total of 888 standart penetration tests (SPT) at 91 exploratory drilling points were conducted throughout the project in order to gain insight about in-situ characteristics of the formations. A total of 22 standard penetration tests at four exploratory drilling points (KY-107, KY-115, KY-128, KY-135) were conducted in vicinity of the failed cut-slopes in the study area (Appendix A). At KY-107 10 SPT’s, at KY-115 5 SPT’s, at KY-128 3 SPT’s, at KY-135 4 SPT’s were conducted. The disturbed samples taken with penetrometer and undisturbed samples in samplers were tested in Yüksel Proje Uluslararası A.Ş.’s soil and rock mechanics laboratory. All SPT tests were conducted in accordance with TS 1900-1 and TS 1900-2 standards considering Technical Specifications in Exploratory Engineering Services of General Directorate of Highways (KGM, 2005).

Table 4.1. Exploratory drilling data (Yüksel Proje, 2011a)

Drilling No	KM	Coordinates (ITRF96-TM33)			Depth (m)	GW Level (m)
		X	Y	Z		
KY-66	74+792	4 414 830.00	545 302.00	830.10	21.00	6.00
KY-67	75+380	4 415 026.00	545 861.00	822.05	15.45	5.00
KY-68	76+492	4 415 264.00	546 947.00	840.19	15.07	2.20
KY-69	76+918	4 415 319.00	547 372.00	825.04	30.27	4.30
KY-70	77+090	4 415 355.00	547 540.00	820.39	30.40	11.70
KY-71	77+203	4 415 377.00	547 651.00	824.06	30.45	11.40
KY-72	77+662	4 415 450.00	548 104.00	830.05	10.95	6.20
KY-73	78+041	4 415 537.00	548 473.00	807.45	15.25	4.40
KY-74	79+079	4 415 708.00	549 497.00	817.35	21.45	3.90
KY-75	79+575	4 415 671.00	549 992.00	801.69	15.45	6.10
KY-76	80+869	4 415 670.00	551 289.00	846.97	35.13	13.70
KY-77	81+925	4 415 578.00	552 341.00	825.59	15.90	3.00
KY-78	82+631	4 415 543.00	553 043.00	805.73	15.25	2.50
KY-79	83+978	4 415 874.00	554 341.00	793.94	15.06	9.00
KY-80	84+669	4 416 192.00	554 955.00	794.64	33.00	3.70
KY-80A	84+845	4 416 318.00	555 082.00	796.23	36.00	4.60
KY-81	85+266	4 416 539.00	555 441.00	884.07	85.00	23.90
KY-82	85+885	4 416 921.00	555 929.00	812.93	15.00	5.00
KY-83	87+422	4 418 045.00	556 973.00	888.72	62.00	26.20
KY-84	88+003	4 418 399.00	557 436.00	878.60	50.00	40.50
KY-85	88+118	4 418 421.00	557 561.00	868.62	35.00	15.20
KY-86	88+386	4 418 587.00	557 767.00	858.33	27.00	16.50
KY-87	88+967	4 418 890.00	558 259.00	836.45	16.95	1.20
KY-88	90+000	4 419 132.00	559 260.00	845.56	25.95	3.60
KY-89	91+310	4 419 376.00	560 547.00	857.44	30.00	4.90
KY-90	91+372	4 419 361.00	560 612.00	856.61	30.00	6.70
KY-91	93+047	4 419 819.00	562 220.00	858.76	22.95	0.90
KY-92	95+827	4 420 735.00	564 845.00	871.53	15.45	1.40
KY-92A	97+167	4 421 212.00	566 097.00	878.45	24.00	2.60
KY-93	98+083	4 421 552.00	566 946.00	882.26	19.95	1.30
KY-94	98+687	4 421 854.00	567 468.00	889.53	19.91	3.60
KY-95	99+449	4 422 154.00	567 169.00	907.96	15.00	10.00
KY-96	100+039	4 422 384.00	568 714.00	899.49	21.05	1.80
KY-97	100+440	4 422 440.00	569 112.00	889.97	27.45	4.40
KY-98	101+009	4 422 493.00	569 679.00	904.62	24.00	1.50
KY-99	101+542	4 422 458.00	570 211.00	910.68	25.77	2.90

Table 4.1. Exploratory drilling data (Yüksel Proje, 2011a) (continued)

Drilling No	**KM**	**Coordinates (ITRF96-TM33)**			**Depth (m)**	**GW Level (m)**
		X	**Y**	**Z**		
KY-100	102+675	4 422 115.00	571 282.00	917.74	19.10	1.80
KY-101	103+500	4 421 766.00	572 029.00	946.39	20.00	11.80
KY-102	104+147	4 421 562.00	572 647.00	952.15	23.00	7.70
KY-103	104+526	4 421 396.00	572 988.00	969.21	45.00	21.30
KY-104	104+952	4 421 243.00	573 385.00	980.73	64.00	64.00
KY-105	105+321	4 421 151.00	573 742.00	976.86	65.00	40.40
KY-106	106+567	4 421 180.00	574 968.00	930.68	40.00	40.00
KY-106A	107+197	4 421 308.00	575 572.00	905.91	16.50	23.90
KY-107	109+871	4 422 321.00	578 035.00	841.94	15.45	9.50
KY-108	110+360	4 422 466.00	578 502.00	825.41	19.50	8.50
KY-109	110+613	4 422 510.00	578 751.00	814.34	30.00	11.00
KY-110	110+775	4 422 530.00	578 912.00	808.12	30.00	3.50
KY-111	111+068	4 422 569.00	579 203.00	810.55	30.00	6.90
KY-112	111+466	4 422 564.00	579 602.00	814.16	30.00	11.50
KY-113	111+870	4 422 517.00	580 005.00	799.65	24.00	10.50
KY-114	112+593	4 422 295.00	580 695.00	789.93	15.00	14.90
KY-115	113+290	4 422 047.00	581 344.00	793.98	21.45	7.30
KY-116	113+624	4 421 912.00	581 651.00	789.38	16.50	3.80
KY-117	114+692	4 421 175.00	582 415.00	752.00	15.00	6.90
KY-118	115+251	4 420 755.00	582 778.00	742.72	22.50	6.80
KY-119	115+570	4 420 483.00	582 944.00	718.82	27.00	8.50
KY-120	115+823	4 420 250.00	583 043.00	712.64	27.00	4.50
KY-121	116+071	4 420 018.00	583 129.00	709.03	27.00	3.20
KY-122	116+335	4 419 766.00	583 205.00	705.60	25.50	2.00
KY-123	116+611	4 419 496.00	583 261.00	703.11	27.00	2.20
KY-124	116+849	4 419 260.00	583 293.00	698.39	25.50	3.00
KY-126	117+214	4 418 900.00	583 353.00	696.50	25.50	1.20
KY-127	118+305	4 417 814.00	583 457.00	690.57	19.50	0.90
KY-128	120+659	4 415 771.00	584 534.00	698.60	12.00	3.50
KY-129	123+205	4 414 226.00	586 557.00	669.91	13.50	1.50
KY-130	123+568	4 413 982.00	586 827.00	669.15	13.50	0.90
KY-130A	124+006	4 413 692.00	587 157.00	668.03	19.59	1.50
KY-131	125+508	4 412 711.00	588 290.00	664.00	16.50	1.80
KY-132	126+037	4 412 335.00	588 662.00	663.12	25.50	0.80
KY-133	126+449	4 412 040.00	588 950.00	666.46	16.59	7.50
KY-134	127+674	4 411 171.00	589 813.00	667.25	10.50	2.30

Table 4.1. Exploratory drilling data (Yüksel Proje, 2011a) (continued)

Drilling No	**KM**	**Coordinates (ITRF96-TM33)**			**Depth (m)**	**GW Level (m)**
		X	**Y**	**Z**		
KY-135	128+457	4 410 611.00	590 361.00	666.26	10.50	0.80
KY-136	129+712	4 409 764.00	591 284.00	670.64	13.50	2.35
KY-137	131+370	4 408 974.00	592 721.00	669.02	15.00	5.90
KY-138	132+086	4 408 809.00	593 420.00	665.70	34.50	2.05
KY-139	132+531	4 408 802.00	593 868.00	666.88	29.00	1.90
KY-140	132+823	4 408 821.00	594 160.00	668.96	21.00	4.80
KY-141	132+966	4 408 827.00	594 302.00	667.67	15.00	8.10
KY-142	133+674	4 408 907.00	595 008.00	711.95	30.00	4.60
KY-143	134+228	4 408 876.00	595 563.00	683.95	15.00	11.50
KY-144	135+287	4 408 726.00	596 607.00	733.89	33.00	29.30
KY-144A	136+463	4 408 676.00	597 787.00	706.13	15.05	5.50
KY-145	138+227	4 408 827.00	599 544.00	689.07	19.50	2.50
KY-146	141+723	4 407 251.00	602 501.00	698.04	16.59	6.50
KY-147	143+551	4 405 759.00	603 557.00	693.82	28.50	4.60
KY-148	144+637	4 404 864.00	604 172.00	698.20	30.11	7.30
KY-149	145+666	4 404 029.00	604 775.00	698.04	15.00	1.50
KY-150	146+734	4 403 251.00	605 501.00	703.05	16.95	6.50
KY-151	147+983	4 402 570.00	606 547.00	706.78	25.50	7.90
KY-152	149+511	4 401 785.00	607 858.00	712.78	16.94	4.50
KY-153	150+196	4 401 400.00	608 425.00	733.80	21.00	7.60
KY-154	151+304	4 400 855.00	609 391.00	721.97	15.61	3.20
KY-155	152+486	4 400 243.00	610 402.00	731.78	30.00	14.00
KY-156	152+861	4 400 117.00	610 761.00	729.50	27.00	11.60
KY-157	153+194	4 399 902.00	611 023.00	741.77	25.50	21.40

4.1.3. Laboratory Testing

With the aim of determining geological and geotechnical properties of the formations throughout the railway route, laboratory tests were carried out on the samples taken from exploratory drillings, test pits and SPT samplers. Since all failures took place in the cut slopes excavated on İncik formation in the study area, only the tests conducted on the samples belonging to this formation were taken into account in the scope of this study. The laboratory tests are conducted at Yüksel Proje Uluslararası A.Ş.'s soil and rock mechanics laboratory.

From the boreholes KY-107, KY-115, KY-128, KY-135 in the study area, a total of 23 disturbed samples were taken for laboratory analyses. In addition, 6 undisturbed (UD) samples were taken from different borehole locations in the study area.

4.1.3.1. Soil Mechanics Tests

Numerous soil mechanics tests were practiced on disturbed (SPT) and undisturbed (UD) samples within the scope of obtaining engineering geological and index properties i.e.

- natural moisture content,
- Atterberg limits (liquid limit, plasticity limit, plasticity index),
- grain size distribution,
- triaxial compressive strength,
- non-confined compression configuration,
- unified soil classification

of the formations in the railway route. The test results are given in Appendices D and E.

On three UD samples taken from the study area, unconsolidated-undrained (UU) triaxial compressive tests were conducted. The shear strength values for the İncik formation were found to be c=65 kN/m^2 and φ=1° for the UD sample KY-121, c=57 kN/m^2 and φ=2° for the UD sample KY-123 and c=78 kN/m^2 and φ=2° for the UD sample KY-126.

4.1.3.2. Rock Mechanics Tests

Numerous rock mechanics tests were conducted on the core samples taken by exploratory drilling in order to obtain mechanical and physical properties i.e.;

- uniaxial compressive strength,
- wet unit weight,
- point load index

of the rock units within the study area. A total of 7 uniaxial compressive strength tests were performed on samples obtained from exploratory drillings at 2 different locations and 72 point load tests were performed on samples taken from exploratory drillings at 4 different locations. The test results are given in Appendices F and G.

4.2. Characteristics of the Failed-Cut Slopes

A total of four failed-cut slopes were analyzed in terms of their geological and geotechnical properties within the scope of this study. The studied route was in NW-SE direction, 19 kilometers long, with a maximum height of 33 meters and an inclination of H/V = 3/2 (Figure 4.1). All cut slopes were excavated by mechanical excavation.

Excavation of the cut slopes dates back to January 2015. While no failure was met at time of excavation, traces of groundwater outlets and intensive erosion came up during field work around July 2015. The failures took place at right slope of the route except for KM:109+590 cut slope in direction of increasing kilometers. Having lower SPT values in the upper 5-7 meters, these parts of the cut slopes are supposed to be weathered (Appendix A). Nevertheless, these differences are assumed to be negligable, not taken into consideration and the formation is supposed to be homogenous throughout the study area. Disturbance due to excavation was estimated to be the main reason for the landslides. On the other hand, effects of groundwater and surface waters were the triggering factors for failures. Rill erosion was also observed at all sections due to surface runoff.

Figure 4.1. Locations of the failed-cut slopes in the study area

4.2.1. Characteristics of KM:109+590 Cut Slope

KM: 109+590 cut slope is located around 109th kilometer of the railway, at 2,7 km southwest of Büyükyağlı District. The cut slope is composed of the İncik formation with a maximum height of 16 meters and length of 322 meters (Figure 4.2).

The slope has an inclination of H/V=3/2. The cut slope is composed of reddish-brown, gray, parallel and cross-bedded, poorly-sorted, partially uncemented continental conglomerate, sandstone, mudstone alternation with evaporates (Figure 4.3). Rill erosion was commonly observed due to surface runoff at durable parts of the cut slope.

Figure 4.2. Google Earth view of KM:109+590 cut slope

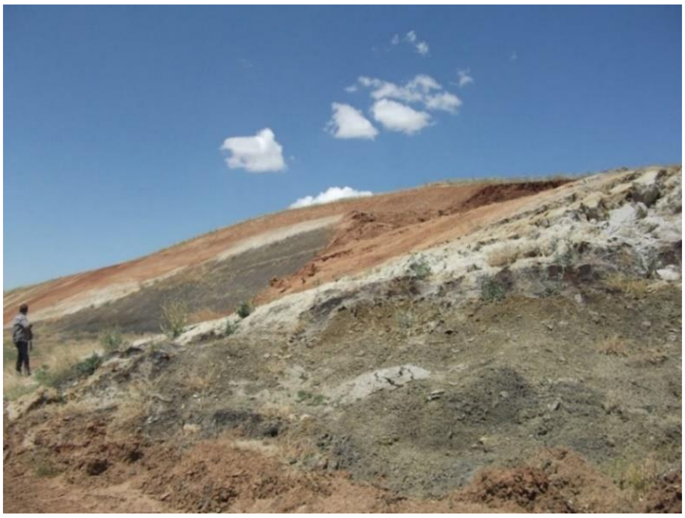


Figure 4.3. General view of KM:109+590 cut slope

4.2.2. Characteristics of KM:113+120 Cut Slope

KM: 113+120 cut slope is located around 113th kilometer of the railway, at 2,7 km southeast of Fadılobası Village. The cut slope is composed of the İncik formation with a maximum height of 33 meters and length of 1456 meters (Figure 4.4). The slope has an inclination of H/V=3/2. The cut slope is composed of reddish-brown, gray, parallel and cross-bedded, poorly-sorted, partially uncemented continental conglomerate, sandstone, mudstone alternation with evaporates (Figure 4.5). Rill erosion was also observed due to surface runoff at durable parts of the cut slope (Figure 4.6).

Figure 4.4. Google Earth view of KM:113+120 cut slope

Figure 4.5: General view of KM:113+120 cut slope

Figure 4.6. Seepage of surface waters and rill erosion on KM:113+120 cut slope

4.2.3. Characteristics of KM:121+200 Cut Slope

KM: 121+200 cut slope is located around 121st kilometer of the railway, at 3,2 km southeast of Fadılobası Village. The cut slope is composed of the İncik formation with a maximum height of 21m and length of 1043 meters (Figure 4.7). The slope has an inclination of H/V=3/2. The cut slope is composed of reddish-brown, gray, parallel and cross-bedded, poorly-sorted, partially uncemented continental conglomerate, sandstone, mudstone alternation with evaporates (Figure 4.8). Rill erosion was also observed due to surface runoff at durable parts of the cut slope.

Figure 4.7. Google Earth view of KM:121+200 cut slope

Figure 4.8: General view of KM:121+200 cut slope

4.2.4. Characteristics of KM:128+630 Cut Slope

KM: 128+630 cut slope is located around 128th kilometer of the railway, at 3,2 km south of Çerikli Village. The cut slope is composed of the İncik formation with a maximum height of 15 meters and length of 1030 meters (Figure 4.9). The slope has an inclination of H/V=3/2. The cut slope is composed of reddish-brown, gray, parallel and cross-bedded, poorly-sorted, partially uncemented continental conglomerate, sandstone, mudstone alternation with evaporates, formation is characterized by conglomerate dominated bands that can be observed at top levels of this cut slope (Figure 4.10). Sheet erosion was also observed due to surface runoff at durable parts of the cut slope.

Figure 4.9. Google Earth view of KM:128+630 cut slope

Figure 4.10: General view of KM:128+630 cut slope

CHAPTER 5

BACK ANALYSES, LIMIT EQUILIBRIUM ANALYSES AND POSSIBLE REMEDIAL MEASURES FOR THE FAILED CUT SLOPES

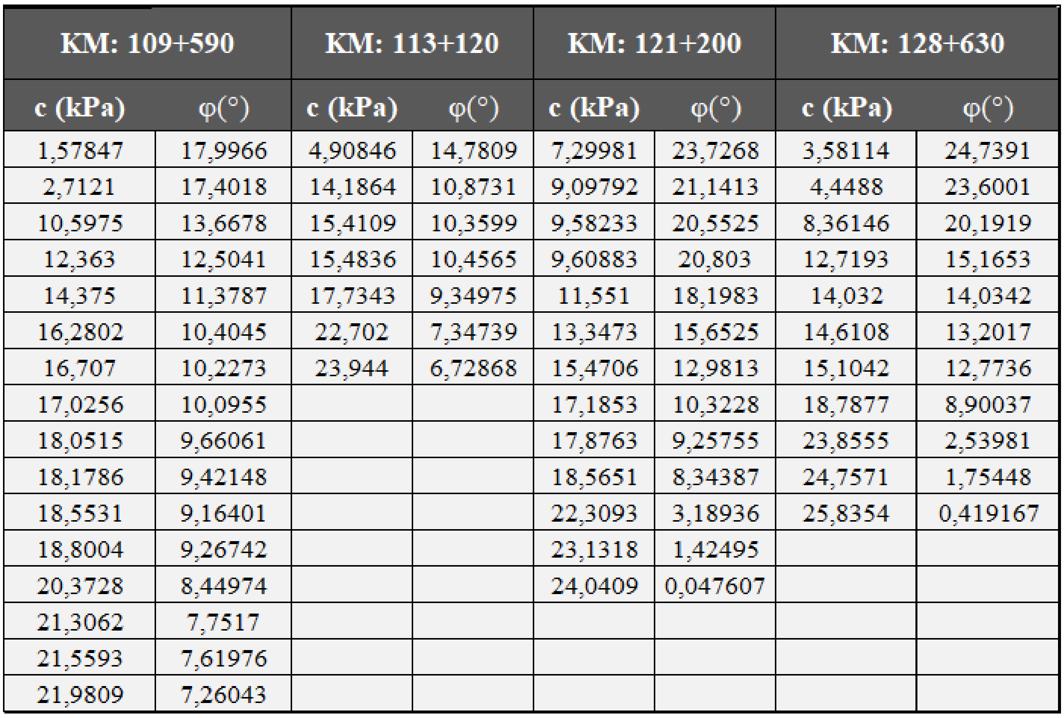
This section of study is about back analyses, limit equilibrium analyses and possible stability solutions of the failures seen at four cut slopes along the railway alignment. For these cut slopes, firstly the present failure conditions were analyzed by back and limit equilibrium analyses. Then, various methods for probable remedial measures were considered for each cut slope.

5.1. Back Analyses of the Failed Cut Slopes

For the scope of obtaining the shear strength parameters of the İncik formation at the time of failure, back analysis was conducted in order to specify remedial slope stability measures. The back analysis is the method in which shear strength parameters are estimated based on given slope geometry and/or some other material properties. This is accomplished by changing cohesion (c) and internal friction angle (φ) values of the material that will produce a factor of safety of 1 along two or more sections (Sancio, 1981; Duncan and Wright, 2005; Topsakal, 2012).

With the scope of attaining the shear strength parameters of the units at time of failure, back analysis was carried out using the Rocscience SLIDE software (Version 6.0) for non-circular slide. For the back analysis, groundwater geometry was defined according to Yüksel Proje’s study (2011a) and slip surfaces were defined according to the field surveys. Using the slope geometry and failure surface of the cut slopes, back-analysis was performed. Pairs of random shear strength parameters (c and φ) of the formation were selected by trail and error for obtaining the limit equilibrium condition where factor of safety (FS) is 1. Table 5.1 presents the c-φ pairs satisfying FS=1 conditions and Figure 5.1 shows the scatter diagram of c – φ pairs obtained from the back analysis for each cut slope.

Table 5.1. c- φ pairs of each cut slope satisfying FS=1 condition based on the back analyses



KM: 109+590		KM: 113+120		KM: 121+200		KM: 128+630	
c (kPa)	**φ(°)**	**c (kPa)**	**φ(°)**	**c (kPa)**	**φ(°)**	**c (kPa)**	**φ(°)**
1,57847	17,9966	4,90846	14,7809	7,29981	23,7268	3,58114	24,7391
2,7121	17,4018	14,1864	10,8731	9,09792	21,1413	4,4488	23,6001
10,5975	13,6678	15,4109	10,3599	9,58233	20,5525	8,36146	20,1919
12,363	12,5041	15,4836	10,4565	9,60883	20,803	12,7193	15,1653
14,375	11,3787	17,7343	9,34975	11,551	18,1983	14,032	14,0342
16,2802	10,4045	22,702	7,34739	13,3473	15,6525	14,6108	13,2017
16,707	10,2273	23,944	6,72868	15,4706	12,9813	15,1042	12,7736
17,0256	10,0955			17,1853	10,3228	18,7877	8,90037
18,0515	9,66061			17,8763	9,25755	23,8555	2,53981
18,1786	9,42148			18,5651	8,34387	24,7571	1,75448
18,5531	9,16401			22,3093	3,18936	25,8354	0,419167
18,8004	9,26742			23,1318	1,42495		
20,3728	8,44974			24,0409	0,047607		
21,3062	7,7517						
21,5593	7,61976						
21,9809	7,26043						

Each pair of c and φ for the corresponding cut slopes which satisfy the condition of FS equal to 1, is nearly located along a single line. Appearently, intersection of these lines concentrate around a small triangular area indicating c=17.5 kPa and φ=9° (Figure 5.1). These values are considered to be representative for the İncik formation.

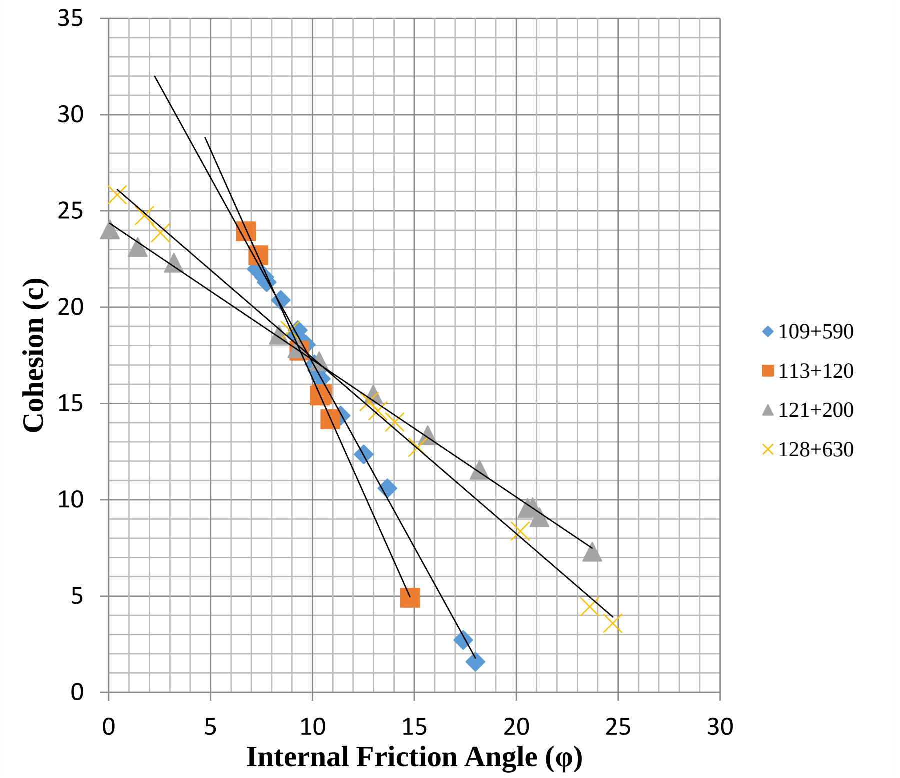


Figure 5.1. c-φ graph of the cut slopes derived from the back analysis

5.2. Limit Equilibrium Analyses and Possible Remedial Measures for the Failed Cut Slopes

For the limit equilibrium analyses, a cohesion (c’) of 17.5 kPa and an internal friction angle (φ’) of 9° based on the back analyses results were used as shear strength parameters of the unit. Saturated unit weight of the formation (γ) was considered to be 20 kN/m^3 with groundwater condition. Stability methods satisfying different equilibrium conditions (horizontal/vertical force or moment equilibrium i.e. Bishop (1955), Janbu (1968), Morgenstern and Price (1965) and Spencer (1967)) are considered to give a better estimation of the safety factor of the slopes.

By using Bishop Simplified, Janbu Corrected, Morgenstern and Price and Spencer methods, number of slices and maximum iterations were 50 and tolerance was 0.005

for a non-circular failure for the analysis. Pseudostatic analysis is conducted throughout all analyses which is the simplest approach used in earthquake engineering to analyze the seismic response of soil embankments and slopes (Melo and Sharma, 2004). Thus, peak horizontal seismic acceleration was accepted to be 0.1g in accordance with General Directorate of Natural Disasters' (AFAD) data for Kırıkkale Municipality.

For long term stability of the cut slopes, minimum safety factor of 1.1 was aimed in accordance with Technical Specifications of General Directorate of Highways (KGM, 2005) because KGM's acceptions are dominantly used in TCDD's high speed railway projects.

Methods for stabilizing slope stability mostly operate by reducing driving forces, increasing resisting forces or both. Removal of material from the essential part of the unstable structure and drainage of water to repress the hydrostatic pressures can be stated as the methods for reducing driving forces. Increasing the shear strength of the ground by drainage, elimination of weak strata or other potential failure zones, building of retaining structures or other supports, provision of in-situ reinforcement of the ground, chemical treatment to increase the shear strength of the ground can be stated as the methods for increasing resisting forces (Abramson et al., 2002).

The probable solutions for long term stability of the cut slopes can be piling, bench and/or toe buttressing, slope flattening, removal of sliding material and filling with rock. Micropiling solution is a method where soil is stuck by digging a retaining support in it. By this way, frictional resistance within the slope is improved. For application of micropiling solution in SLIDE, force application is chosen to be active, force direction is parallel to surface, out of plane spacing is 1 m and pile shear strength is changing from 150 to 1550 kNs.

Forming a bench and removal of sliding material are methods for reducing driving forces by excavating some overlying material by means of forming a step-like structure or retaining the original geometry on slope. For the case of bench solution in

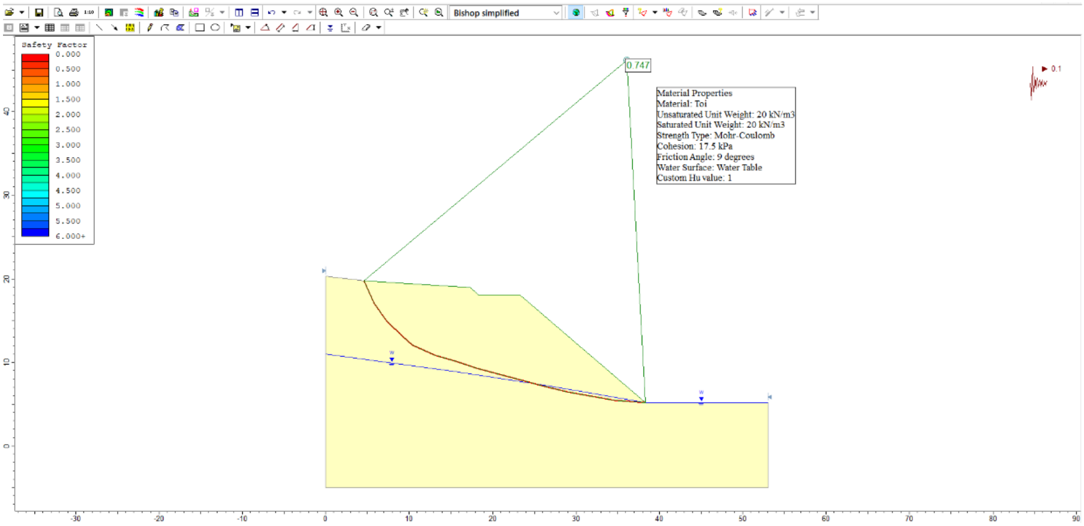
SLIDE, a step-like structure with a width of 5 m was formed per 10 m slope height in accordance with Technical Specifications of General Directorate of Highways (KGM, 1989).

Toe buttressing or filling with rock are methods where soil is stuck by accumulating an amount of rock on slope face in order to increase the frictional resistance within slope. For the case of toe buttressing solution, unit weight of rockfill is accepted to be 22 kN/m^3 where c’ and φ’ are equal to 1 kPa and 35° as rockfill’s shear strength parameters, respectively.

Slope flattening is a method for reducing driving forces by excavating some overlying material and forming a low-pitched slope. Flattening of the slopes with H/V=3/1 was tried for all failed cut slopes as another alternative.

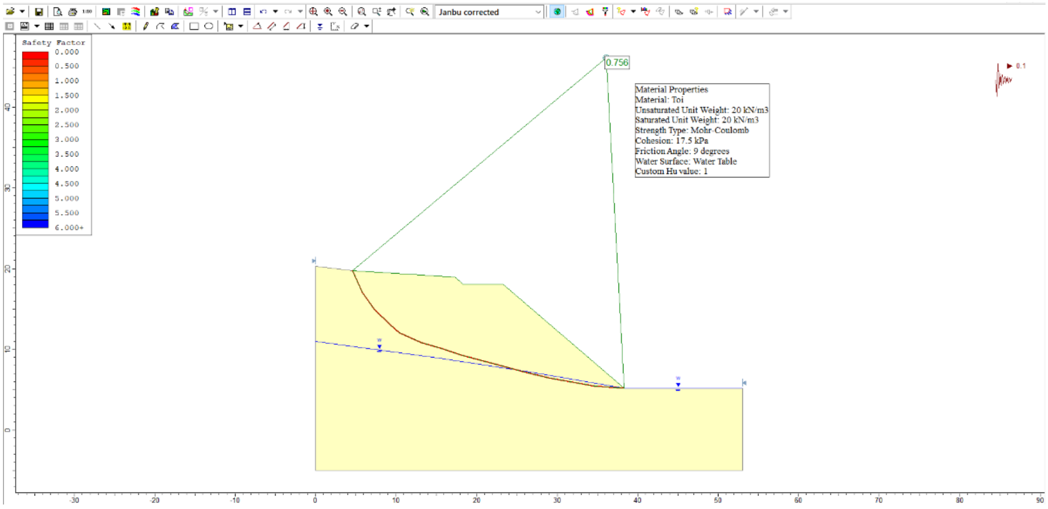
5.2.1. KM:109+590 Cut Slope

In order to assess the stability of KM:109+590 cut slope, slope stability analyses of the KM:109+590 cut slope were performed in accordance with Bishop (1955), Janbu (1968), Morgenstern and Price (1965) and Spencer (1967) methods (Figures 5.2-5.5).



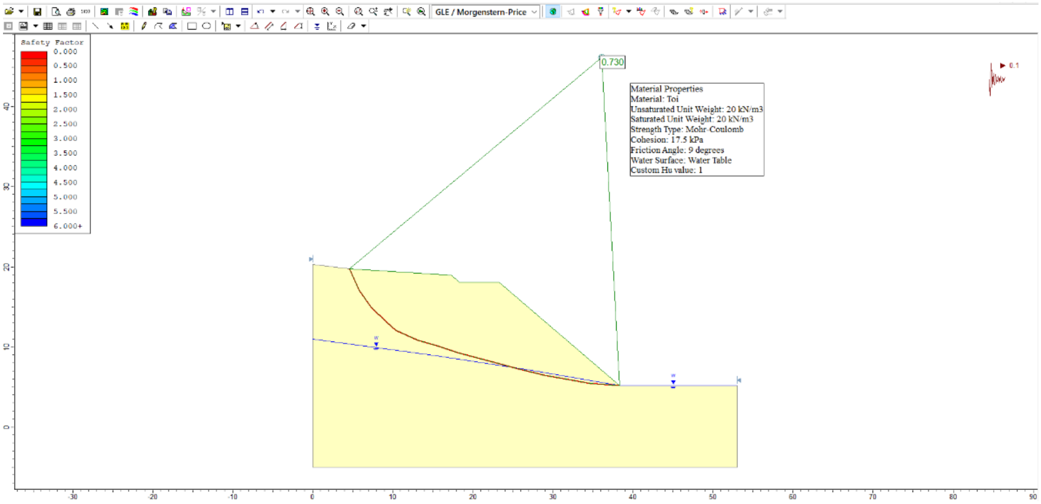
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.2. Stability analysis of KM: 109+590 cut slope based on the back analysis data via Bishop simplified method



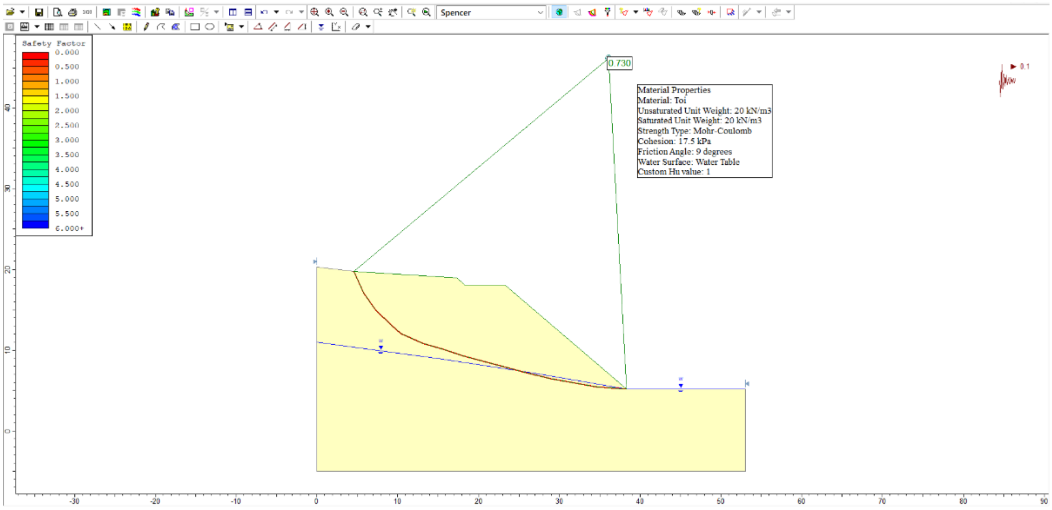
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.3. Stability analysis of KM: 109+590 cut slope based on the back analysis data via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

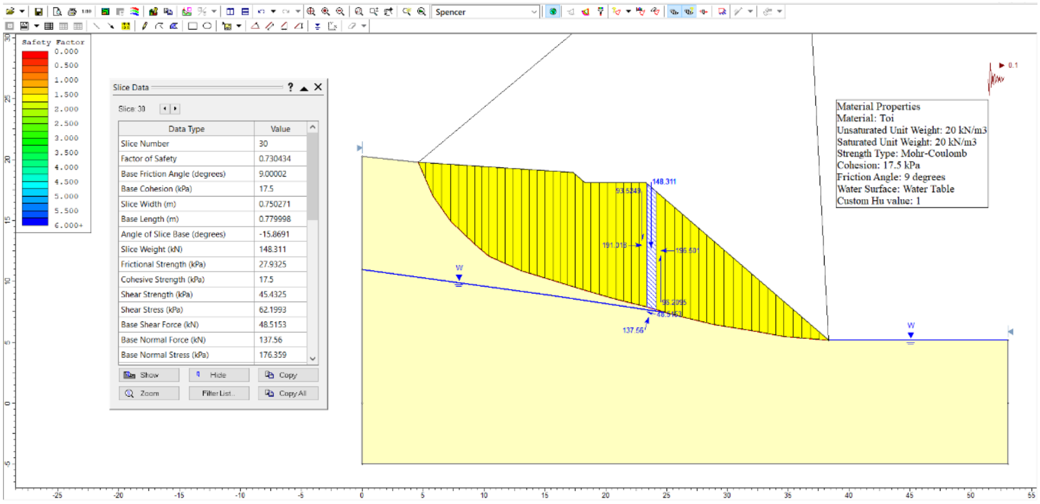
Figure 5.4. Stability analysis of KM: 109+590 cut slope based on the back analysis data via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.5. Stability analysis of KM: 109+590 cut slope based on the back analysis data via Spencer method

Similar safety factors of the slope mainly ranging between 0.73 and 0.76 were obtained. As a solution suggestion, firstly piling solution was tried. In order to decide where to drive the pile, slice with the highest forces (Slice # 30 in this case) was considered and the analysis was performed accordingly (Figure 5.6).



Slice Data

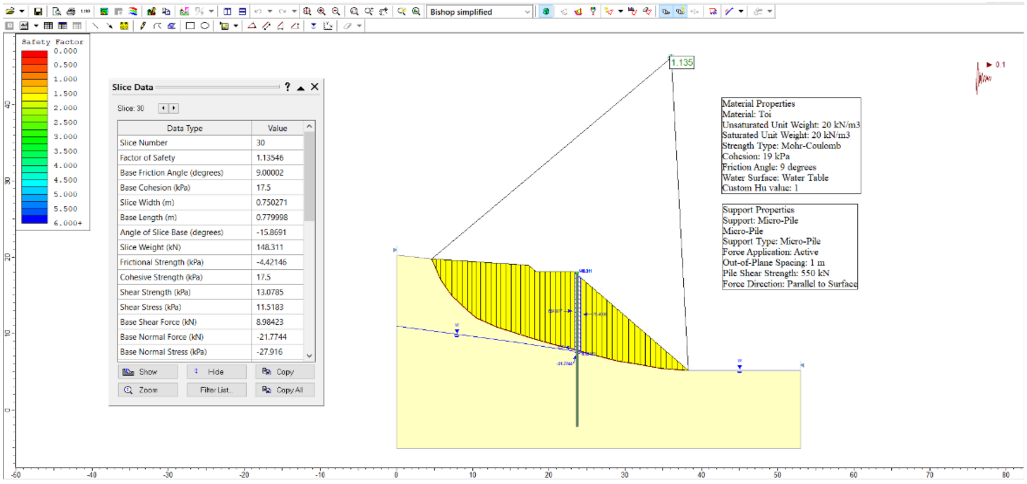
Slice: 30

Data Type	Value
Slice Number	30
Factor of Safety	0.730434
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.750271
Base Length (m)	0.779998
Angle of Slice Base (degrees)	-15.8691
Slice Weight (kN)	148.311
Frictional Strength (kPa)	27.9325
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	45.4325
Shear Stress (kPa)	62.1993
Base Shear Force (kN)	48.5153
Base Normal Force (kN)	137.56
Base Normal Stress (kPa)	176.359

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.6. Forces acting on slice of highest forces (Slice #30) at KM:109+590 cut slope

After a few trials, a pile with a shear strength of 550 kN and length of 20 m satisfied the value of factor of safety 1.1 for KM:109+590 cut slope by all analysis methods where specifically FS was equal to 1.105 via Spencer method (Figures 5.7-5.10). Piling turned out to be a solution for KM:109+590 cut slope.



Slice Data

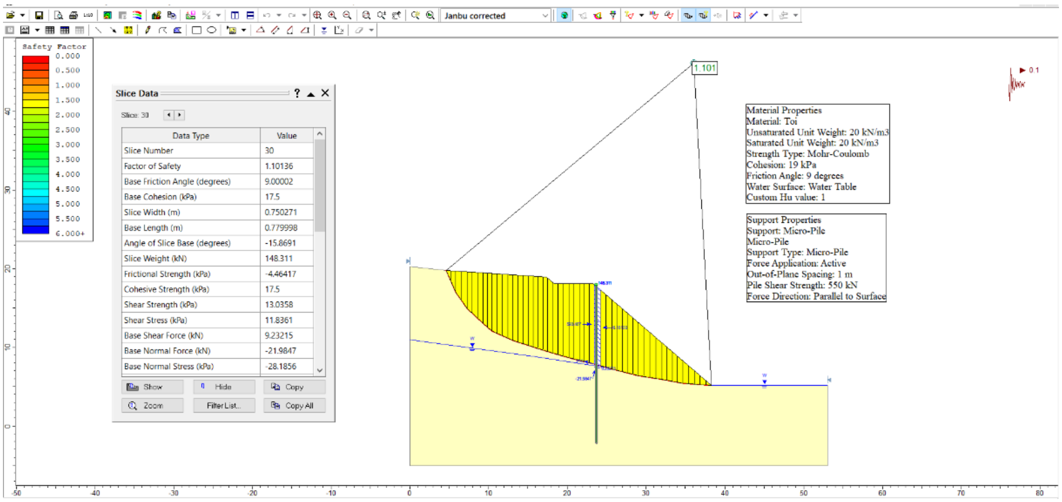
Slice: 30

Data Type	Value
Slice Number	30
Factor of Safety	1.13546
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.750271
Base Length (m)	0.779998
Angle of Slice Base (degrees)	-15.8691
Slice Weight (kN)	148.311
Frictional Strength (kPa)	-4.42146
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	13.0785
Shear Stress (kPa)	11.5183
Base Shear Force (kN)	8.98423
Base Normal Force (kN)	-21.7744
Base Normal Stress (kPa)	-27.916

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 19 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 550 kN
Force Direction: Parallel to Surface

Figure 5.7. Stability analysis of KM: 109+590 cut slope with driven pile solution via Bishop simplified method

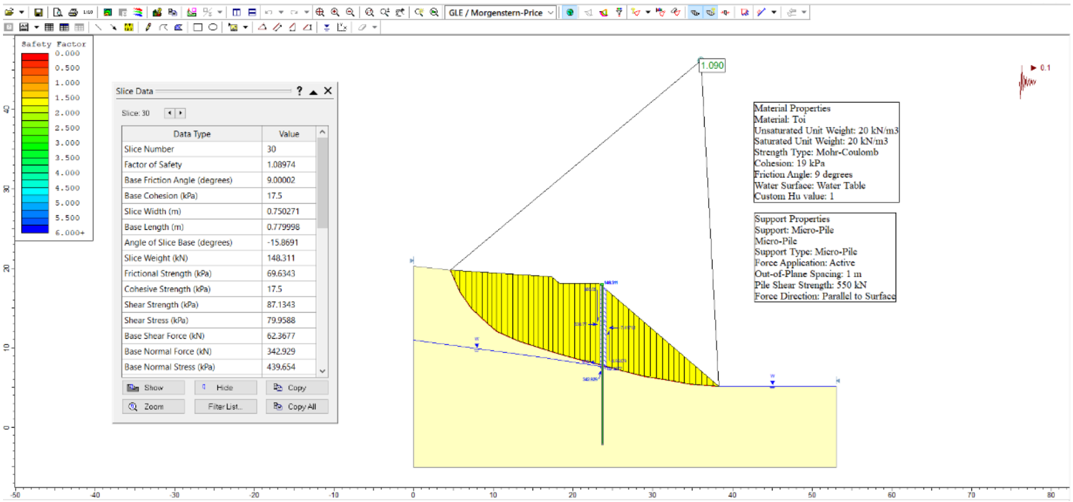


Data Type	Value
Slice Number	30
Factor of Safety	1.10136
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.750271
Base Length (m)	0.779998
Angle of Slice Base (degrees)	-15.8691
Slice Weight (kN)	148.311
Frictional Strength (kPa)	-4.46417
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	13.0358
Shear Stress (kPa)	11.8361
Base Shear Force (kN)	9.23215
Base Normal Force (kN)	-21.9847
Base Normal Stress (kPa)	-28.1856

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 19 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 550 kN
Force Direction: Parallel to Surface

Figure 5.8. Stability analysis of KM: 109+590 cut slope with driven pile solution via Janbu corrected method

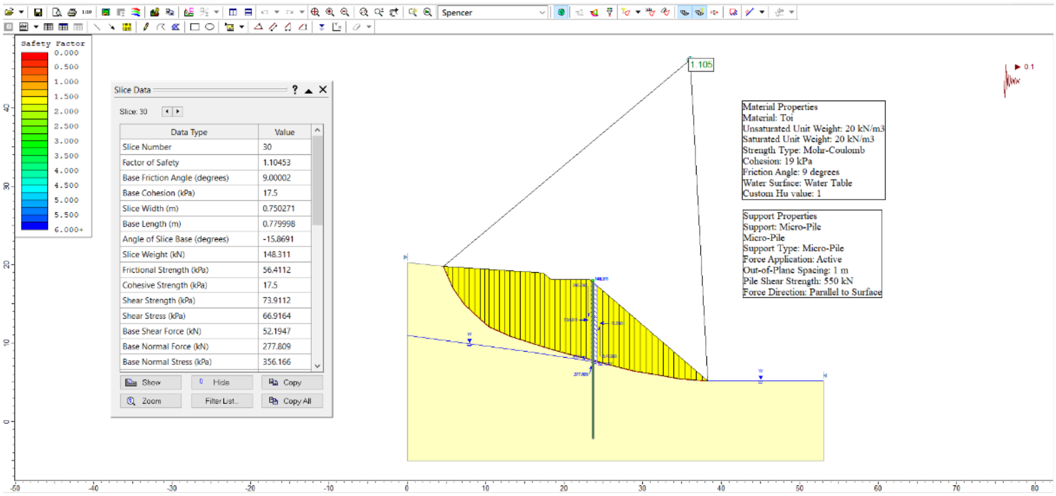


Data Type	Value
Slice Number	30
Factor of Safety	1.08974
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.750271
Base Length (m)	0.779998
Angle of Slice Base (degrees)	-15.8691
Slice Weight (kN)	148.311
Frictional Strength (kPa)	69.6343
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	87.1343
Shear Stress (kPa)	79.9588
Base Shear Force (kN)	62.3677
Base Normal Force (kN)	342.929
Base Normal Stress (kPa)	439.654

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 19 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 550 kN
Force Direction: Parallel to Surface

Figure 5.9. Stability analysis of KM: 109+590 cut slope with driven pile solution via GLE/Morgenstern-Price method



Slice Data

Slice: 30

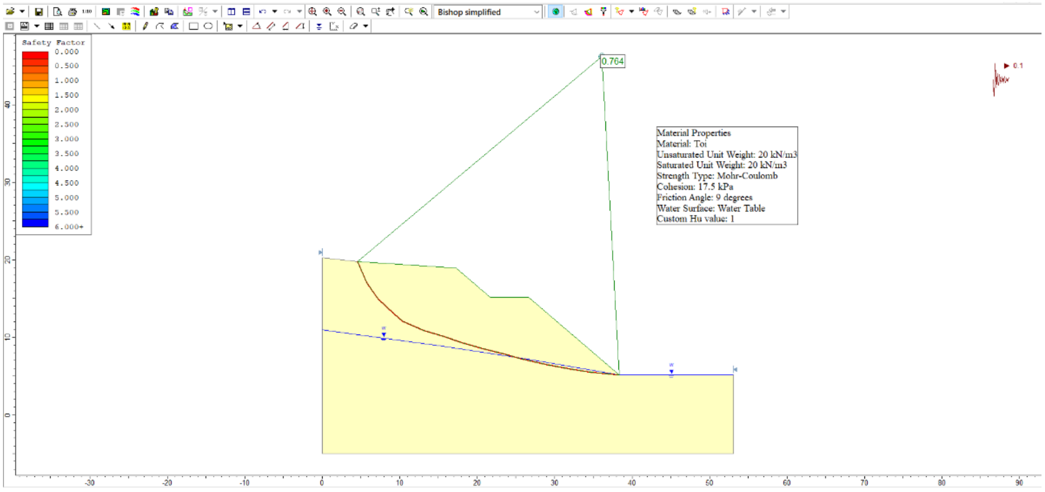
Data Type	Value
Slice Number	30
Factor of Safety	1.10453
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.750271
Base Length (m)	0.779998
Angle of Slice Base (degrees)	-15.8691
Slice Weight (kN)	148.311
Frictional Strength (kPa)	56.4112
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	73.9112
Shear Stress (kPa)	66.9164
Base Shear Force (kN)	52.1947
Base Normal Force (kN)	277.809
Base Normal Stress (kPa)	356.166

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 19 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 550 kN
Force Direction: Parallel to Surface

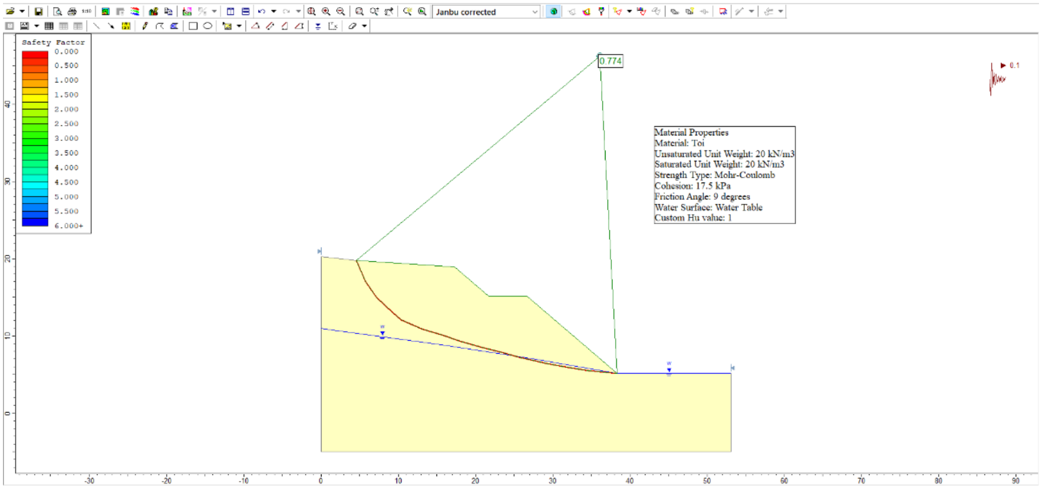
Figure 5.10. Stability analysis of KM: 109+590 cut slope with driven pile solution via Spencer method

Secondly, benching solution with 5 m bench width and 10 m slope height was tried via SLIDE software. Low safety factors of the slope were obtained (Figures 5.11-5.14). Therefore, benching was not considered to be a solution for KM: 109+590 cut slope.



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

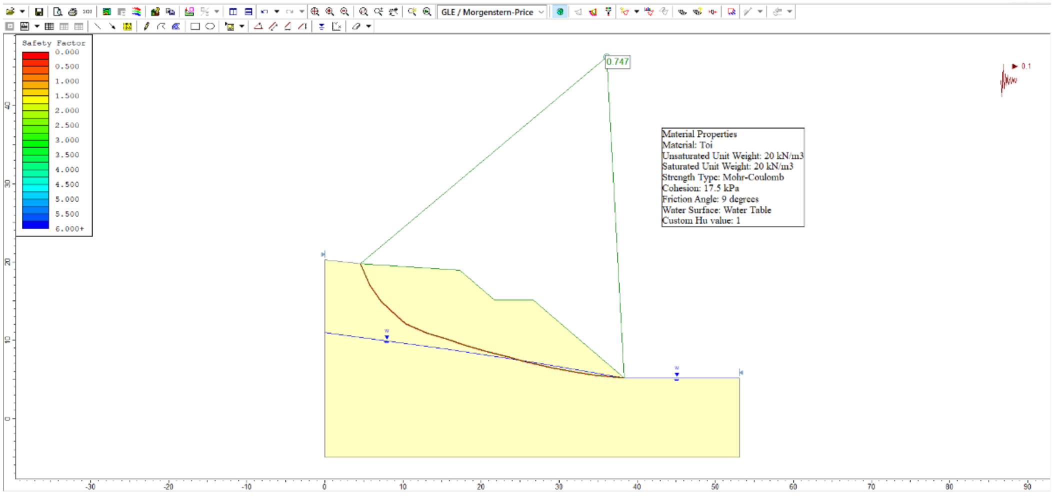
Figure 5.11. Stability analysis of KM: 109+590 cut slope with benching solution via Bishop simplified method



Janbu corrected

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

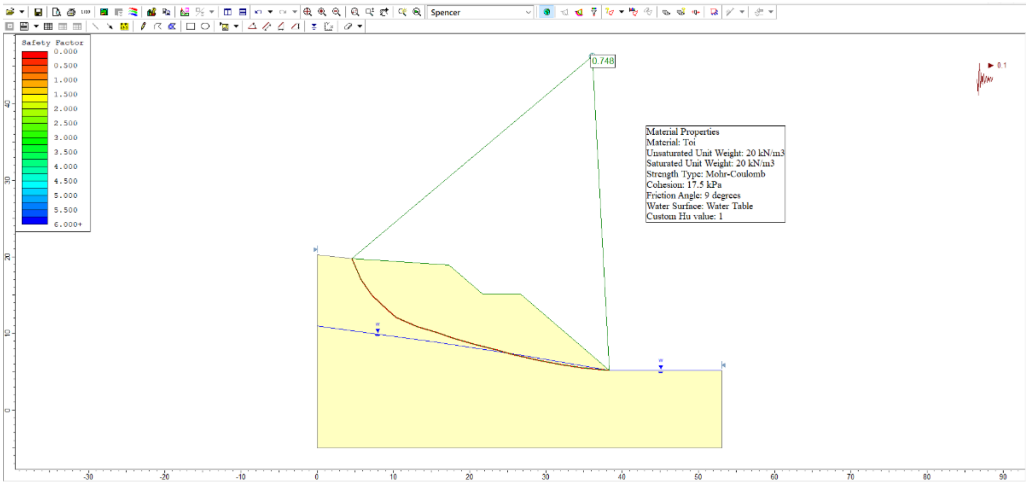
Figure 5.12. Stability analysis of KM: 109+590 cut slope with benching solution via Janbu corrected method



GLE / Morgenstern-Price

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

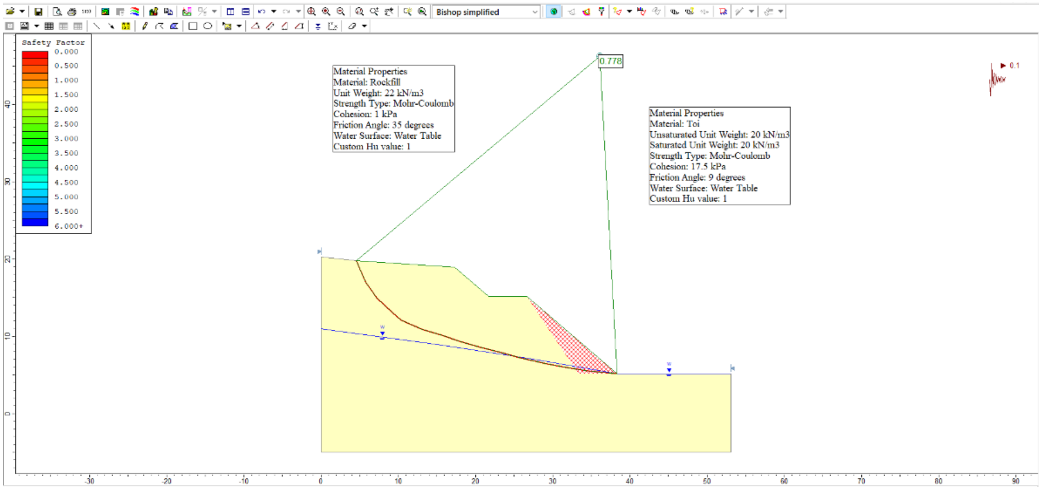
Figure 5.13. Stability analysis of KM: 109+590 cut slope with benching solution via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.14. Stability analysis of KM: 109+590 cut slope with benching solution via Spencer method

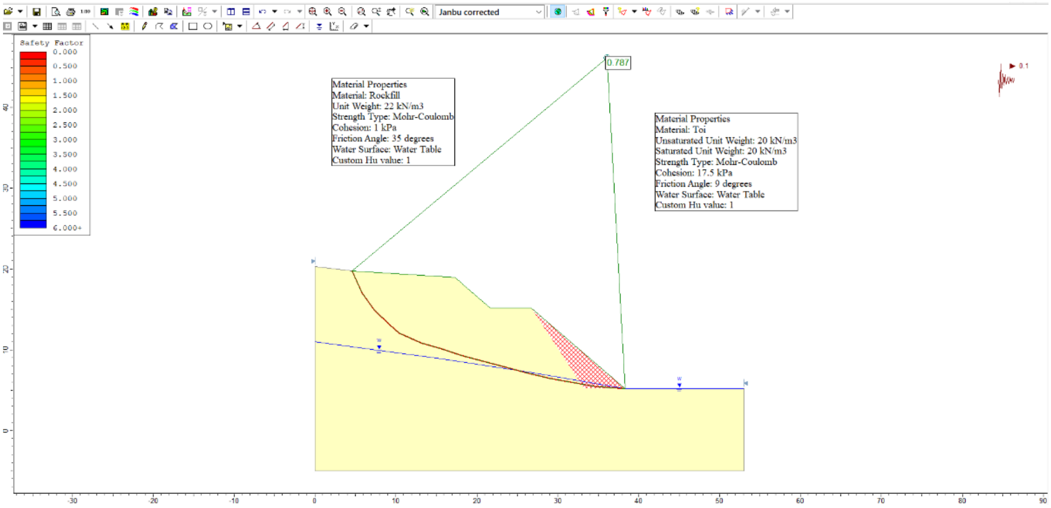
In addition to benching solution, toe buttressing option as toe support was also put on for this slope. Along the lower part of the slope, a strip of 5 m-wide soil at toe was removed and refilled with rock. Relatively low safety factors of the slopes with toe buttressing are obtained for this slope (Figures 5.15-5.18). Therefore, toe buttressing and benching options were not considered to be a solution for KM:109+590 cut slope.



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

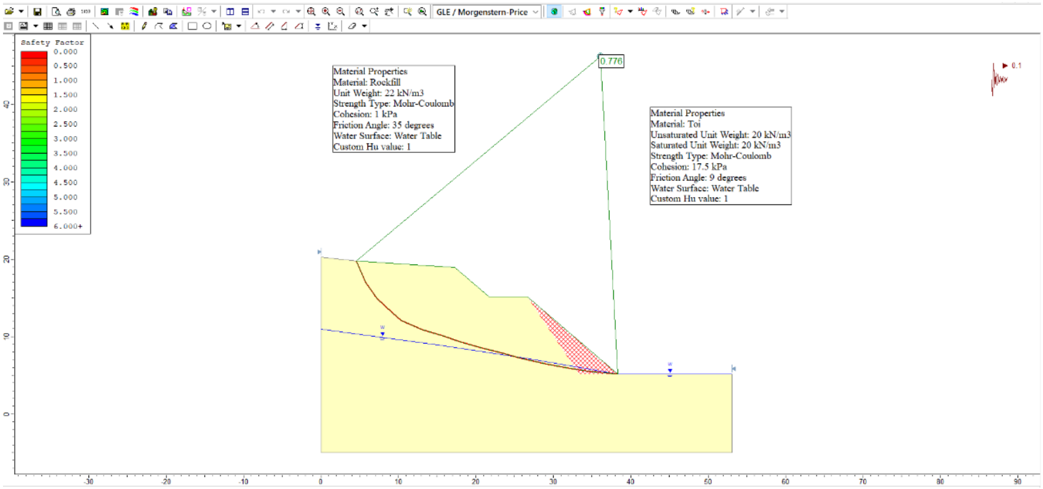
Figure 5.15. Stability analysis of KM: 109+590 cut slope with benching+toe buttressing solution via Bishop simplified method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

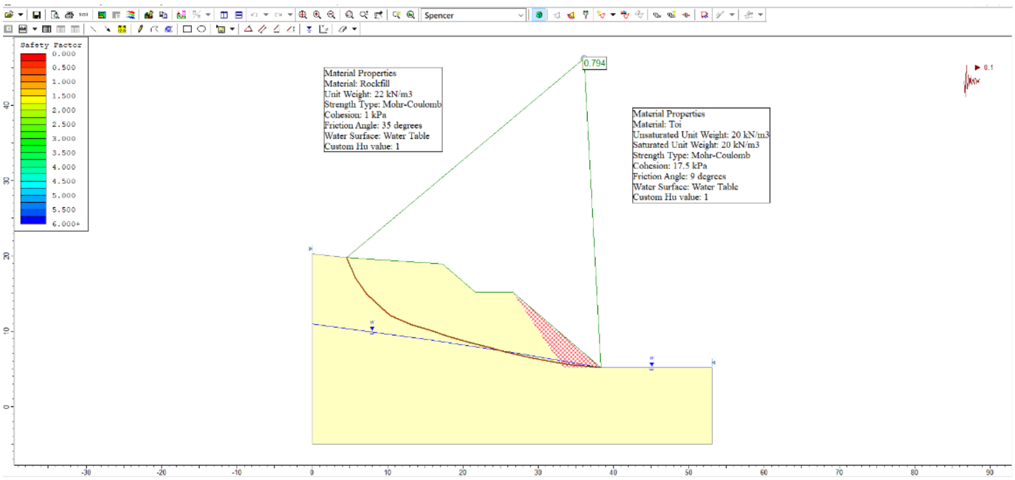
Figure 5.16. Stability analysis of KM: 109+590 cut slope with benching+toe buttressing solution via Janbu corrected method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.17. Stability analysis of KM: 109+590 cut slope with benching+toe buttressing solution via GLE/Morgenstern-Price method

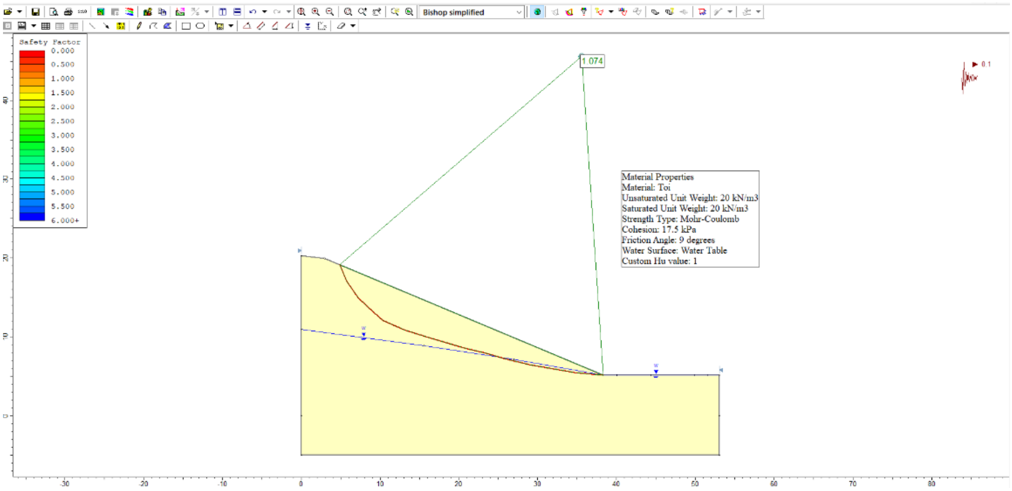


Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

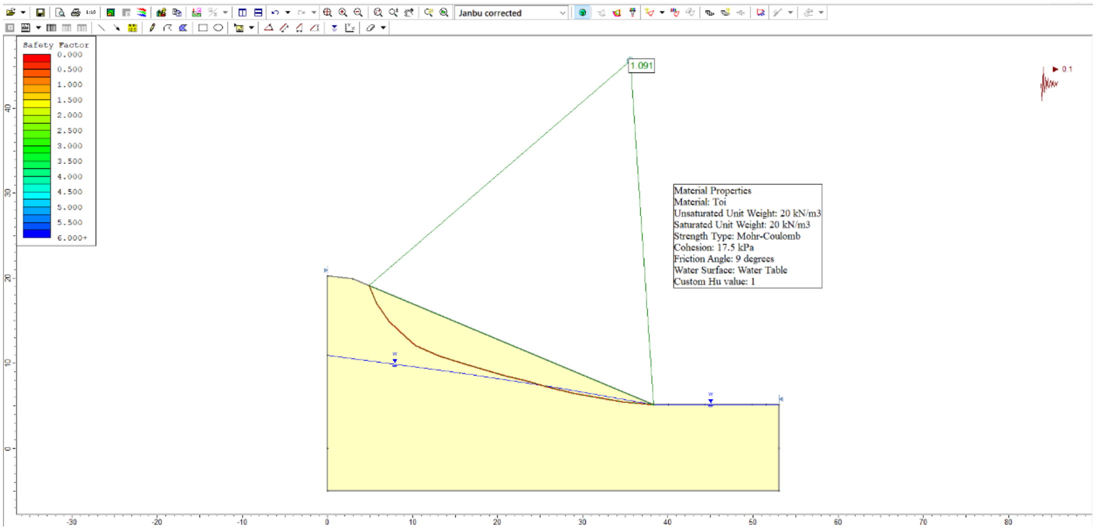
Figure 5.18. Stability analysis of KM: 109+590 cut slope with benching+toe buttressing solution via Spencer method

Fourthly, slope flattening solution was tried. The original slope was flattened by excavating some material throughout the slope so that new slope has an inclination of H/V=3/1. Relatively low safety factors of the flattened slope were obtained using different method of analyses (Figures 5.19-5.22). Therefore, slope flattening was not considered to be an alternative solution for KM:109+590 cut slope.



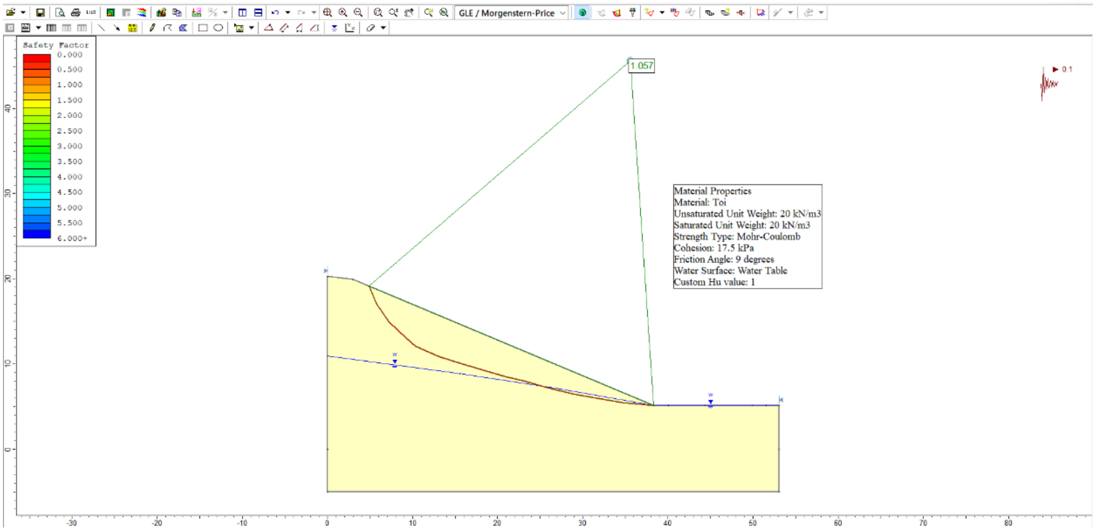
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.19. Stability analysis of KM: 109+590 cut slope with slope flattening solution via Bishop simplified method



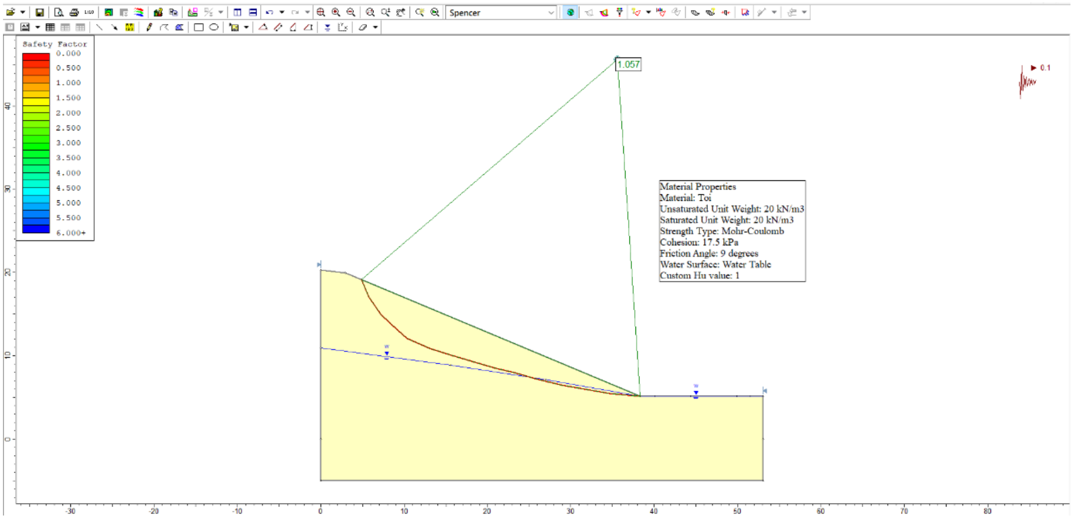
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.20. Stability analysis of KM: 109+590 cut slope with slope flattening solution via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

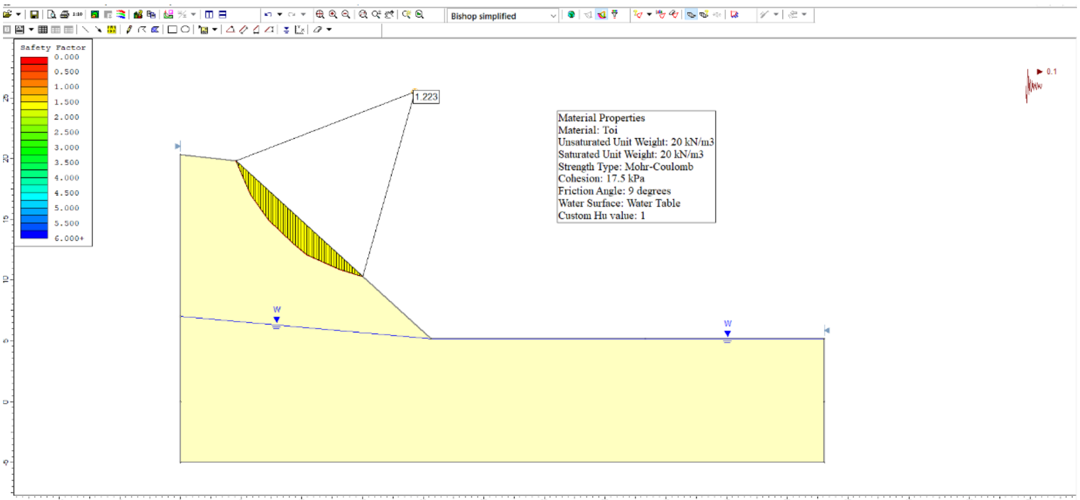
Figure 5.21. Stability analysis of KM: 109+590 cut slope with slope flattening solution via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.22. Stability analysis of KM: 109+590 cut slope with slope flattening solution via Spencer method

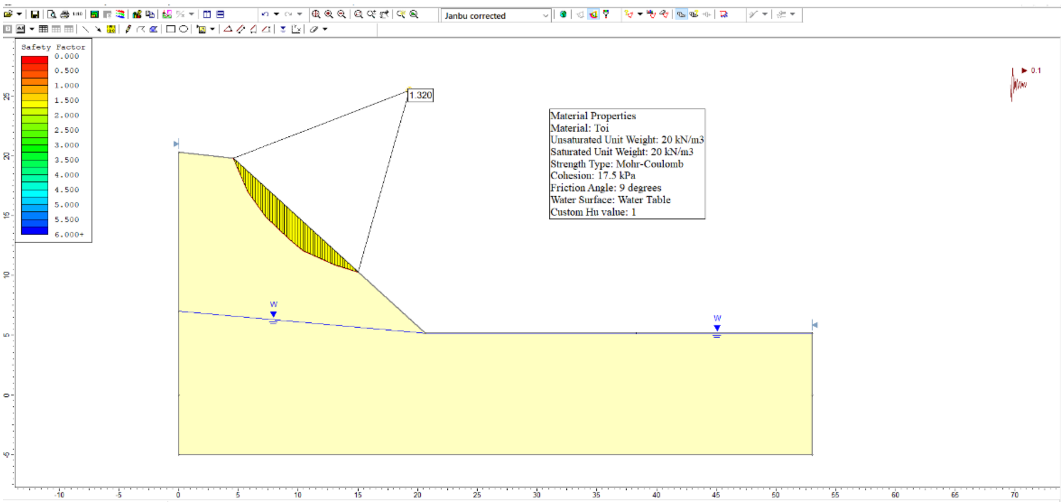
Fifthly, solution of removal of sliding material was tried. The original slope was excavated 10 m in width and reshaped in the way that the same inclination of H/V = 3/2 was satisfied after removal. Relatively high safety factors for the reshaped slopes were obtained using different method of analyses (Figures 5.23-5.26). Therefore, removal of sliding material turned out to be an alternative solution for KM:109+590 cut slope.



Bishop simplified

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

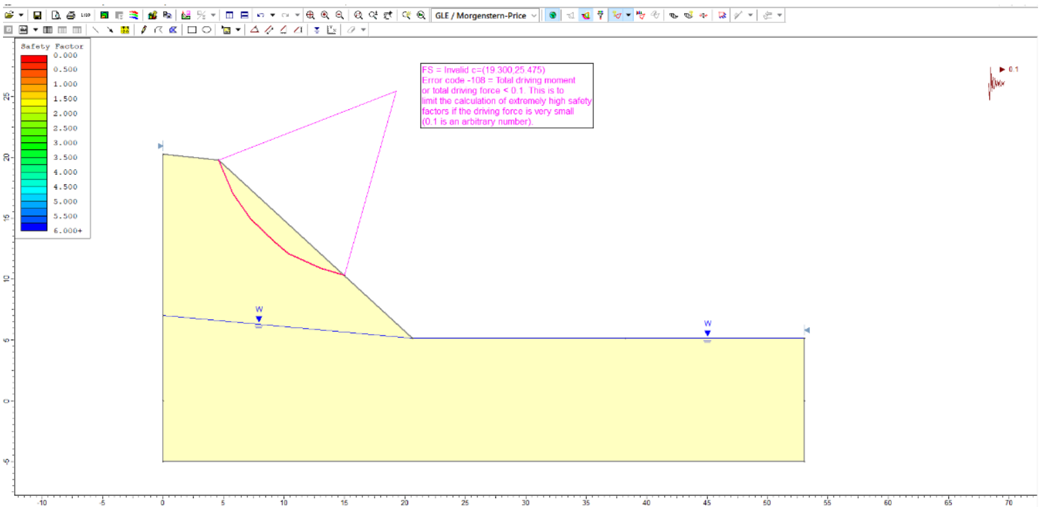
Figure 5.23. Stability analysis of KM: 109+590 cut slope with removal of sliding material solution via Bishop simplified method



Janbu corrected

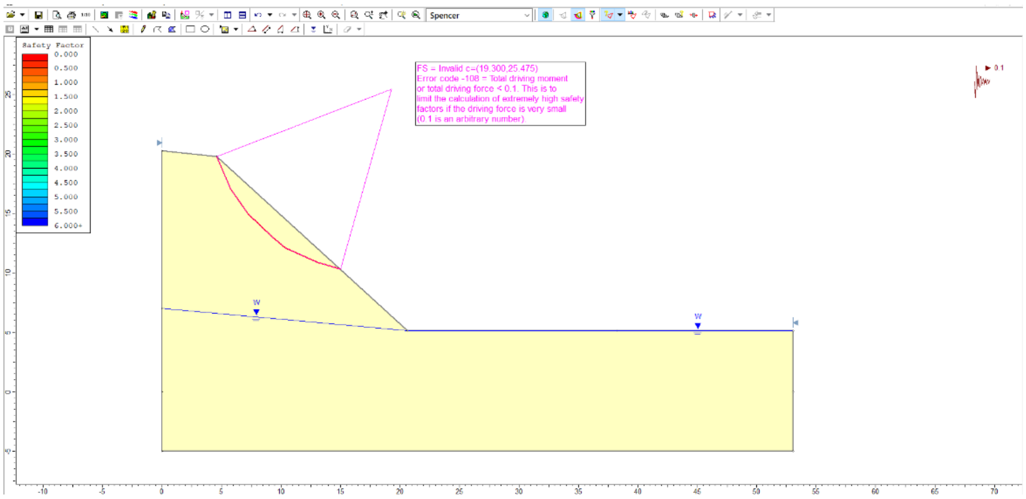
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.24. Stability analysis of KM: 109+590 cut slope with removal of sliding material solution via Janbu corrected method



FS = Invalid c=(19.300,25.475)
Error code -108 = Total driving moment
or total driving force < 0.1. This is to
limit the calculation of extremely high safety
factors if the driving force is very small
(0.1 is an arbitrary number).

Figure 5.25. Stability analysis of KM: 109+590 cut slope with removal of sliding material solution via GLE/Morgenstern-Price method

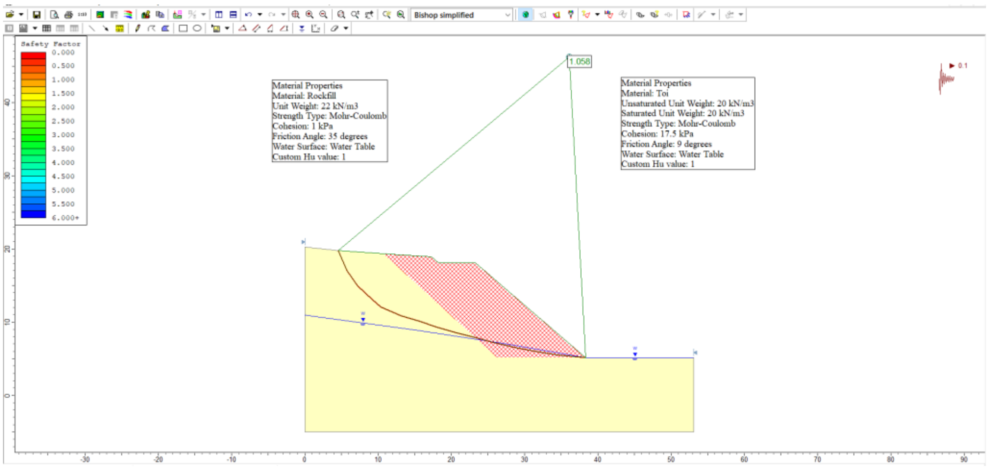


FS = Invalid c=(19.300,25.475)
Error code -108 = Total driving moment
or total driving force < 0.1. This is to
limit the calculation of extremely high safety
factors if the driving force is very small
(0.1 is an arbitrary number).

Figure 5.26. Stability analysis of KM: 109+590 cut slope with removal of sliding material solution via Spencer method

Lastly, solution of removal of sliding material and filling with rock was tried. The original slope was excavated 20 m in width and filled with rock in the way that the same inclination of H/V = 3/2 was satisfied after removal and filling with rock.

Relatively low safety factors of the flattened slope were obtained using Bishop simplified and Janbu corrected methods while relatively high safety factors were obtained using GLE/Morgenstern-Price and Spencer methods of analyses (Figures 5.27-5.30). Therefore, removal of sliding material and filling with rock accepted to be an alternative solution for KM:109+590 cut slope.



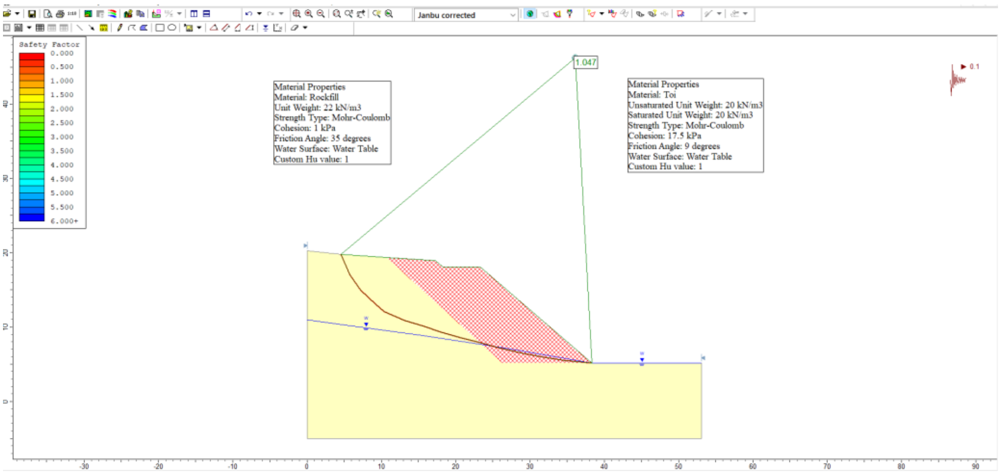
Bishop simplified

1.058

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.27. Stability analysis of KM: 109+590 cut slope with removal of sliding material+filling with rock solution via Bishop simplified method



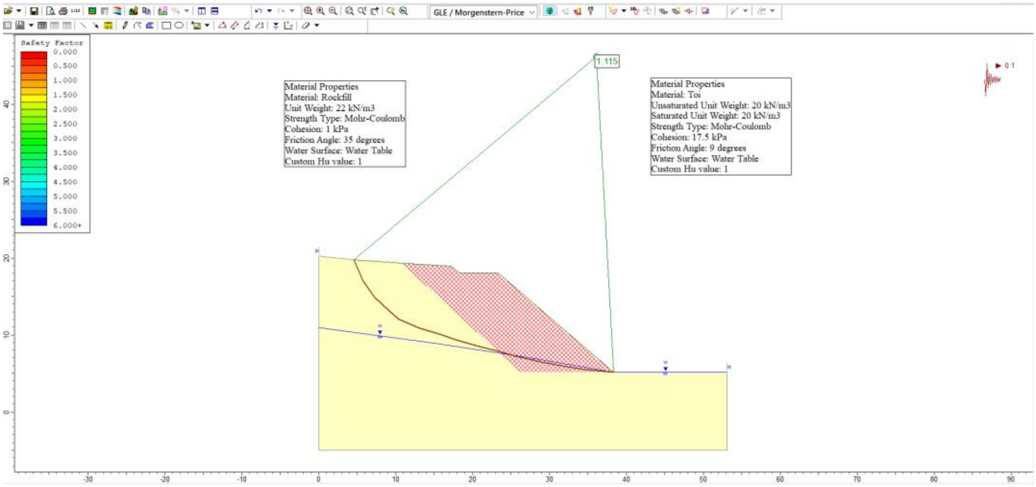
Janbu corrected

1.047

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

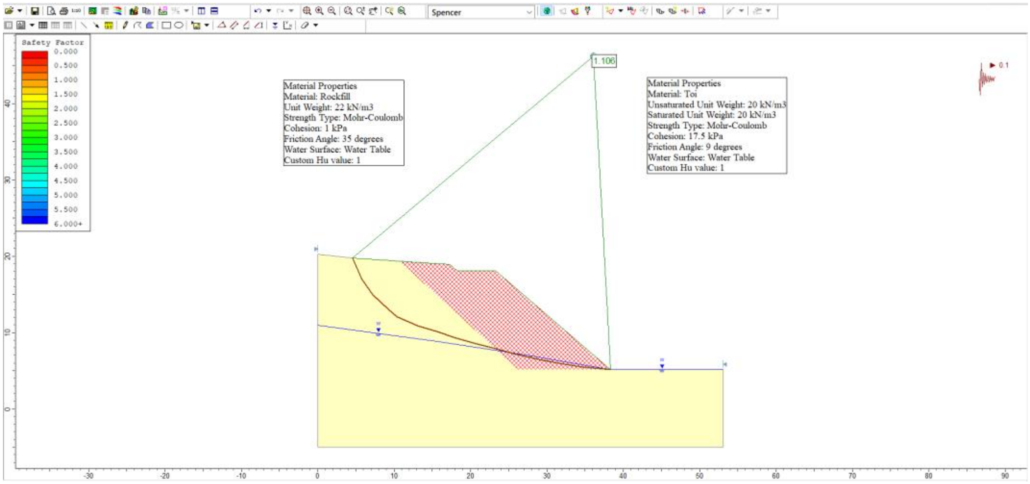
Figure 5.28. Stability analysis of KM: 109+590 cut slope with removal of sliding material+filling with rock solution via Janbu corrected method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.29. Stability analysis of KM: 109+590 cut slope with removal of sliding material+filling with rock solution via GLE/Morgenstern-Price method



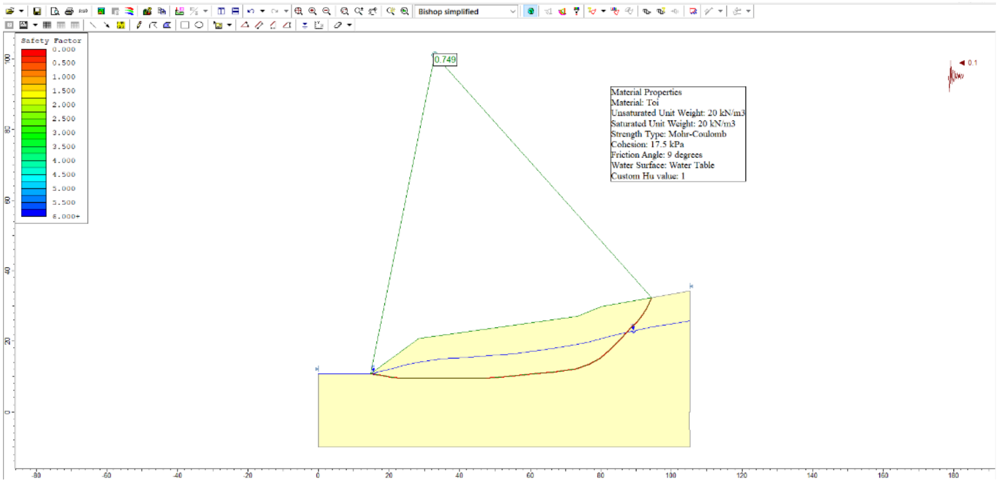
Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.30. Stability analysis of KM: 109+590 cut slope with removal of sliding material+filling with rock solution via Spencer method

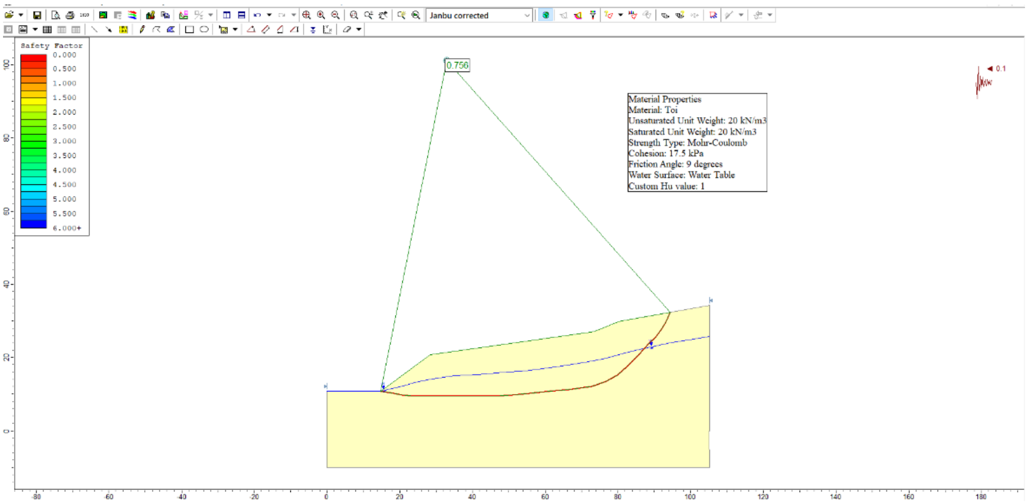
5.2.2. KM:113+120 Cut Slope

In order to survey the stability of KM:113+120 cut slope, slope stability analyses were performed in accordance with Bishop (1955), Janbu (1968), Morgenstern and Price (1965) and Spencer (1967) methods (Figures 5.31-5.34).



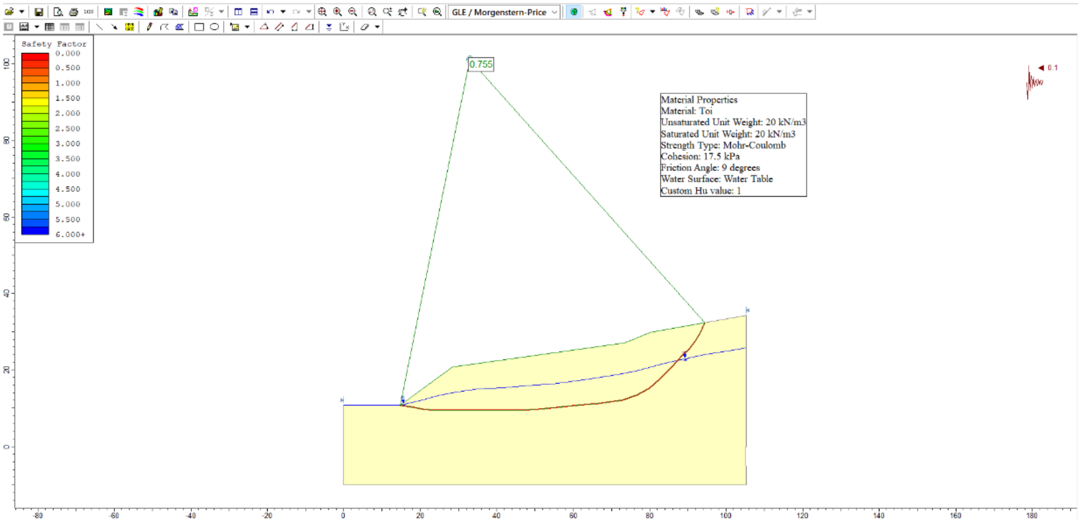
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.31. Stability analysis of KM: 113+120 cut slope based on back analysis data via Bishop simplified method



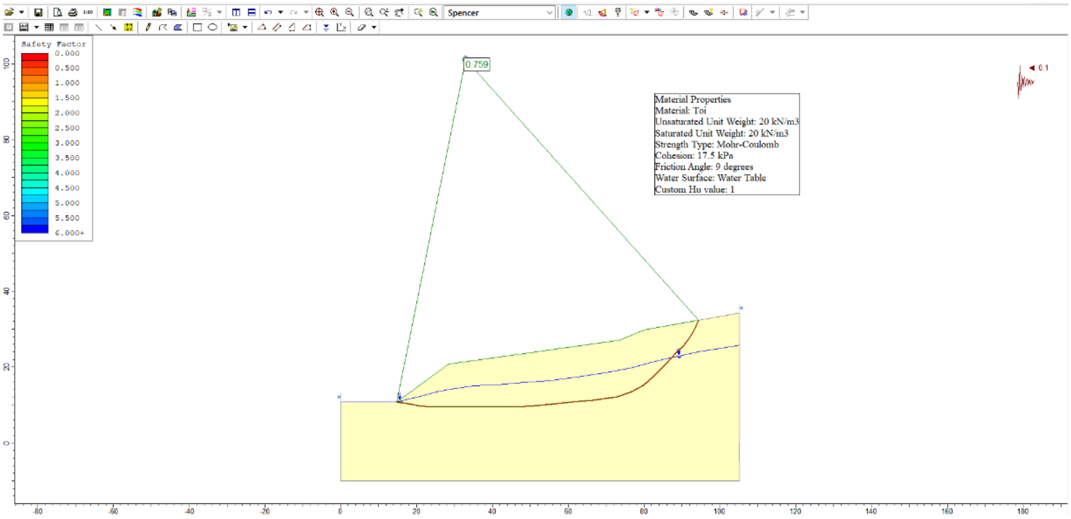
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.32. Stability analysis of KM: 113+120 cut slope based on back analysis data via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

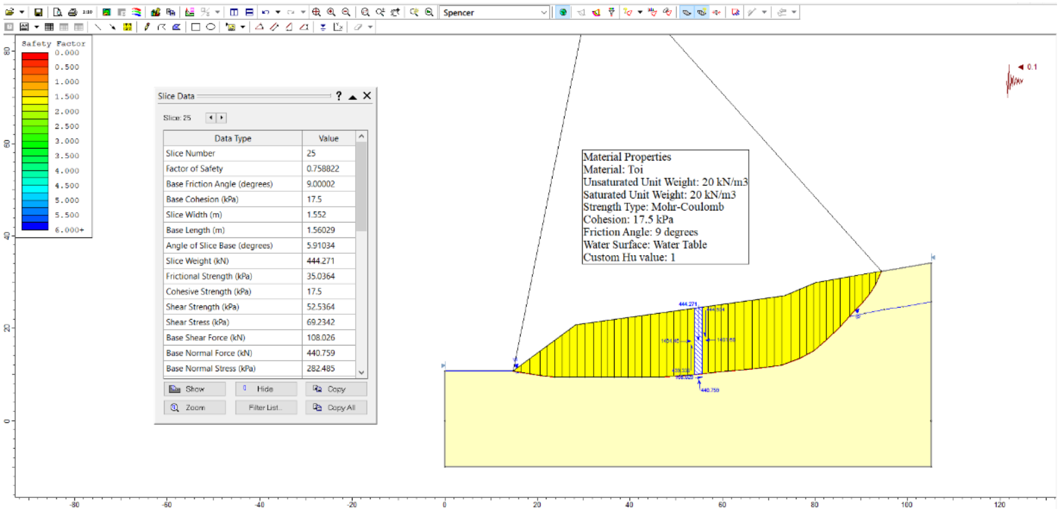
Figure 5.33. Stability analysis of KM: 113+120 cut slope based on back analysis data via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.34. Stability analysis of KM: 113+120 cut slope based on back analysis data via Spencer method

As a slope remedial measure, firstly piling solution was tried. In order to decide where to drive the pile, slice of highest forces was found. SLIDE software was run, the slices were analyzed and slice number of 25 found to be the slice of highest forces (Figure 5.35).

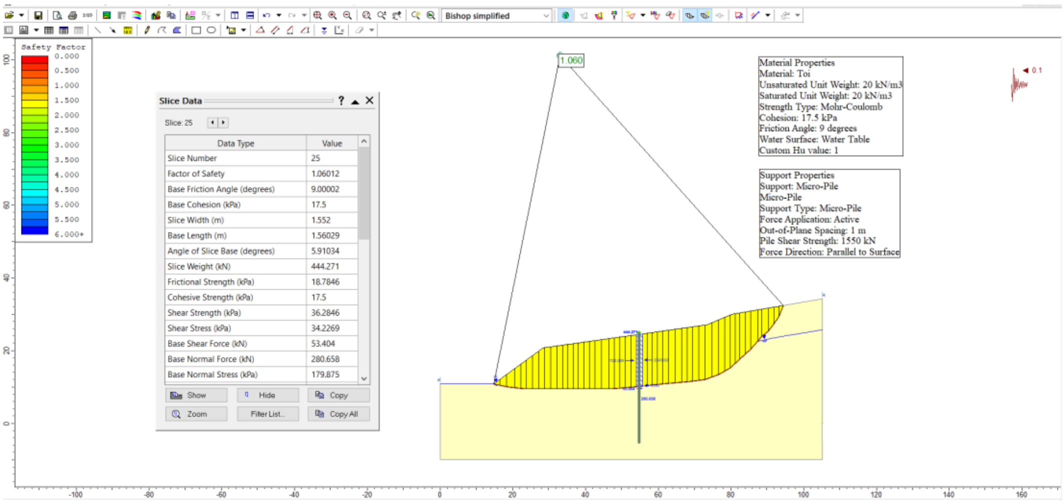


Data Type	Value
Slice Number	25
Factor of Safety	0.758822
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	1.552
Base Length (m)	1.56029
Angle of Slice Base (degrees)	5.91034
Slice Weight (kN)	444.271
Frictional Strength (kPa)	35.0364
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	52.5364
Shear Stress (kPa)	69.2342
Base Shear Force (kN)	108.026
Base Normal Force (kN)	440.759
Base Normal Stress (kPa)	282.485

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.35. Forces acting on slice of highest forces (Slice #25) at KM:113+120 cut slope

After a few trials, a pile with a shear strength of 1550 kN and length of 30 m satisfied the value of factor of safety 1.1 for KM:113+120 cut slope by all analysis methods where specifically FS is equal to 1.105 via Spencer method (Figures 5.36-5.39). Therefore, piling turned out to be a solution for KM:113+120 cut slope.

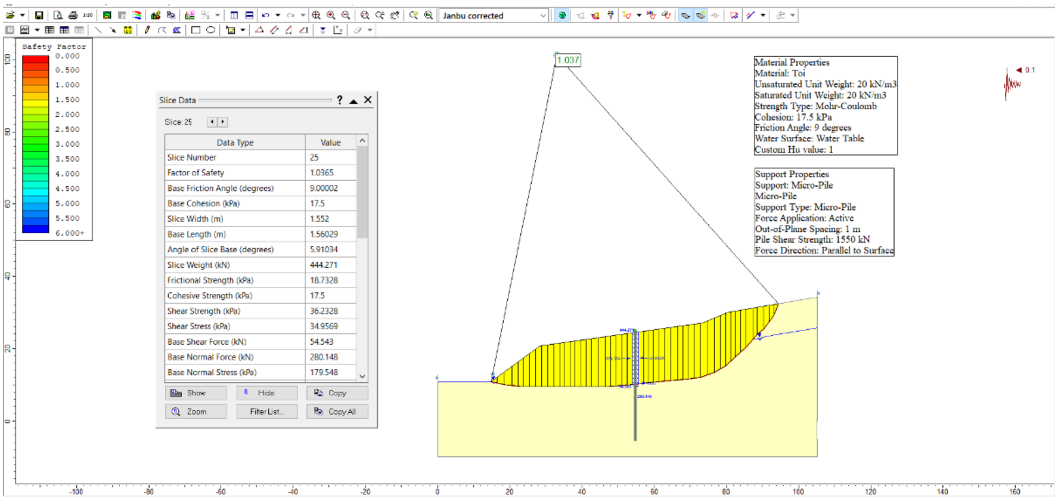


Data Type	Value
Slice Number	25
Factor of Safety	1.06012
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	1.552
Base Length (m)	1.56029
Angle of Slice Base (degrees)	5.91034
Slice Weight (kN)	444.271
Frictional Strength (kPa)	18.7846
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	36.2846
Shear Stress (kPa)	34.2269
Base Shear Force (kN)	53.404
Base Normal Force (kN)	280.658
Base Normal Stress (kPa)	179.875

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 1550 kN
Force Direction: Parallel to Surface

Figure 5.36. Stability analysis of KM: 113+120 cut slope with driven pile solution via Bishop simplified method



Slice Data

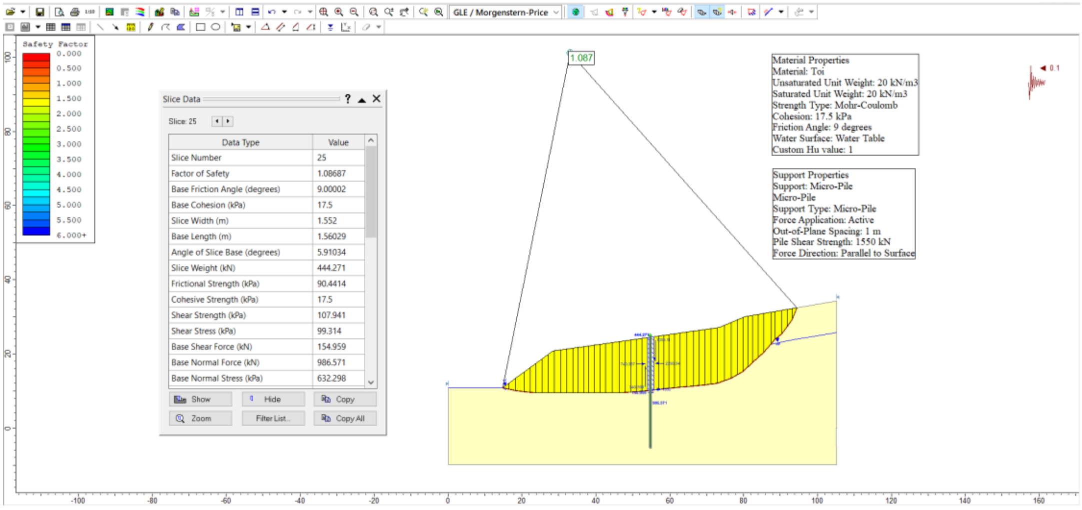
Slice: 25

Data Type	Value
Slice Number	25
Factor of Safety	1.0365
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	1.552
Base Length (m)	1.56029
Angle of Slice Base (degrees)	5.91034
Slice Weight (kN)	444.271
Frictional Strength (kPa)	18.7328
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	36.2328
Shear Stress (kPa)	34.9569
Base Shear Force (kN)	54.543
Base Normal Force (kN)	280.148
Base Normal Stress (kPa)	179.548

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 1550 kN
Force Direction: Parallel to Surface

Figure 5.37. Stability analysis of KM: 113+120 cut slope with driven pile solution via Janbu corrected method



Slice Data

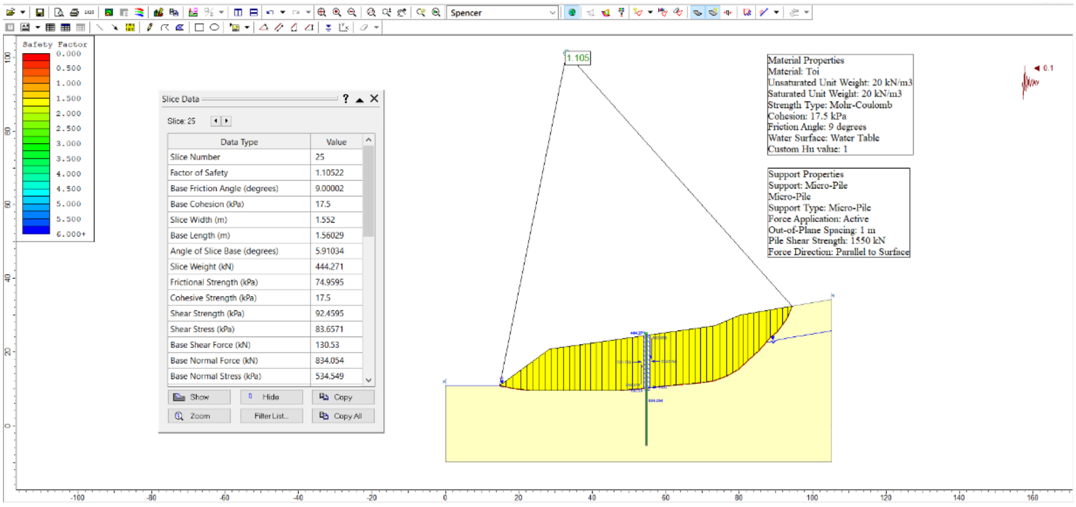
Slice: 25

Data Type	Value
Slice Number	25
Factor of Safety	1.08687
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	1.552
Base Length (m)	1.56029
Angle of Slice Base (degrees)	5.91034
Slice Weight (kN)	444.271
Frictional Strength (kPa)	90.4414
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	107.941
Shear Stress (kPa)	99.314
Base Shear Force (kN)	154.959
Base Normal Force (kN)	986.571
Base Normal Stress (kPa)	632.298

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 1550 kN
Force Direction: Parallel to Surface

Figure 5.38. Stability analysis of KM: 113+120 cut slope with driven pile solution via GLE/Morgenstern-Price method



Slice Data

Slice: 25

Data Type	Value
Slice Number	25
Factor of Safety	1.10522
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	1.552
Base Length (m)	1.56029
Angle of Slice Base (degrees)	5.91034
Slice Weight (kN)	444.271
Frictional Strength (kPa)	74.9595
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	92.4595
Shear Stress (kPa)	83.6571
Base Shear Force (kN)	130.53
Base Normal Force (kN)	834.054
Base Normal Stress (kPa)	534.549

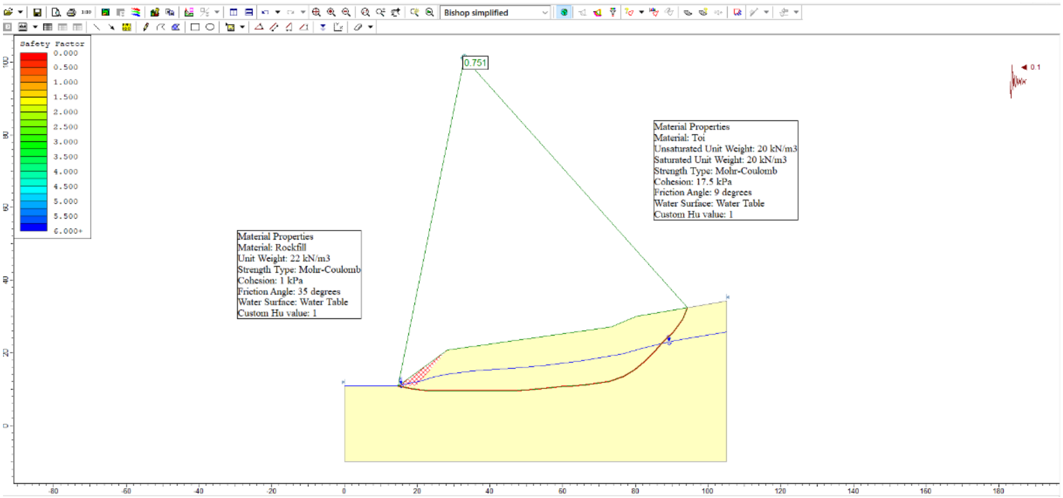
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 1550 kN
Force Direction: Parallel to Surface

Figure 5.39. Stability analysis of KM: 113+120 cut slope with driven pile solution via Spencer method

Benching did not turn out to be a solution for KM:113+120 cut slope since a maximum 5m bench per 10m slope height could not be formed within the slope due to its geometry. For this reason, no benching trials were experienced for KM:113+120 cut slope.

As an alternetive solution, toe buttressing option was considered in this study. Along the lower part of the slope, 5 m wide strip of soil at toe was removed and refilled with rock. The stability analyses reveal that factor of safeties of the slope are lower than the acceptable limit (Figures 5.40-5.43). Therefore, toe buttressing option did not turn out to be a solution for KM:113+120 cut slope.

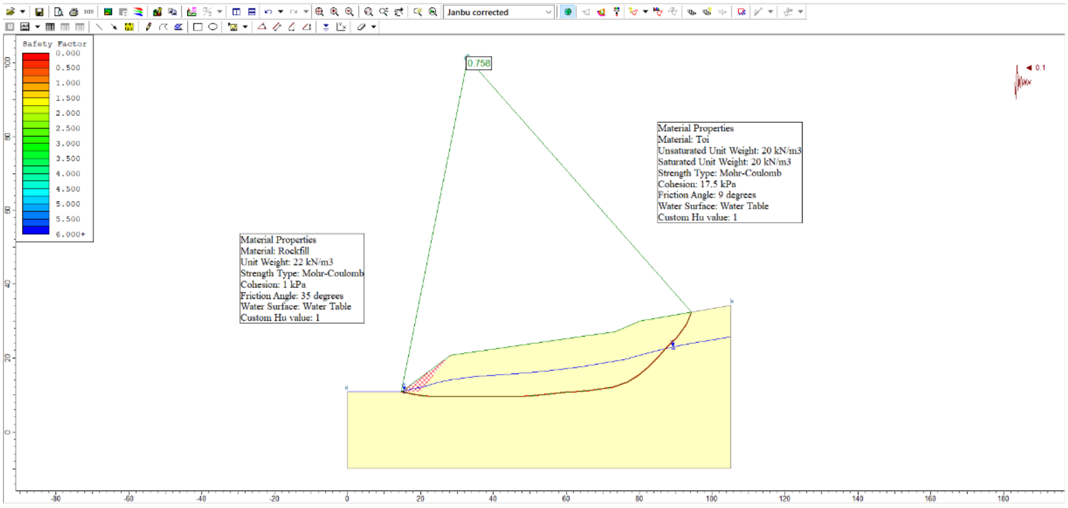


Bishop simplified

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.40. Stability analysis of KM: 113+120 cut slope with toe buttressing solution via Bishop simplified method

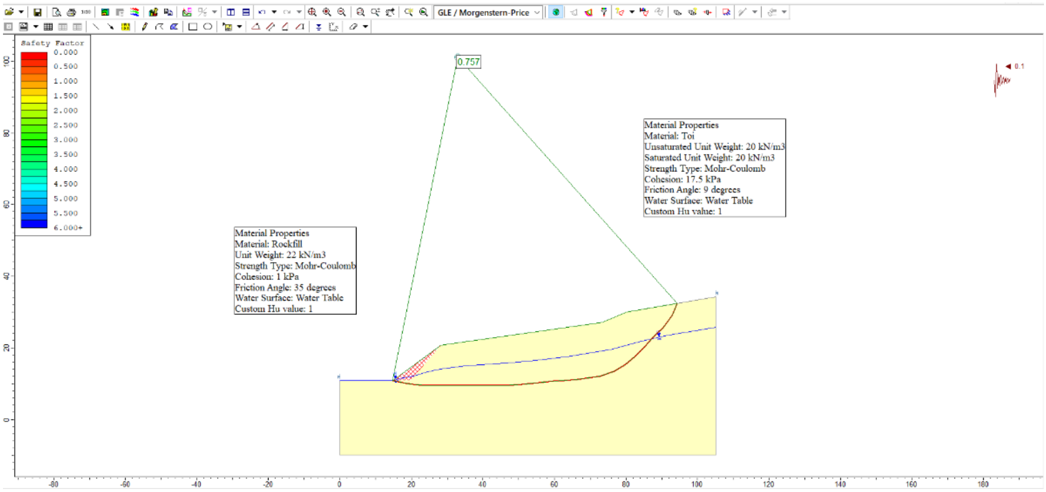


Janbu corrected

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

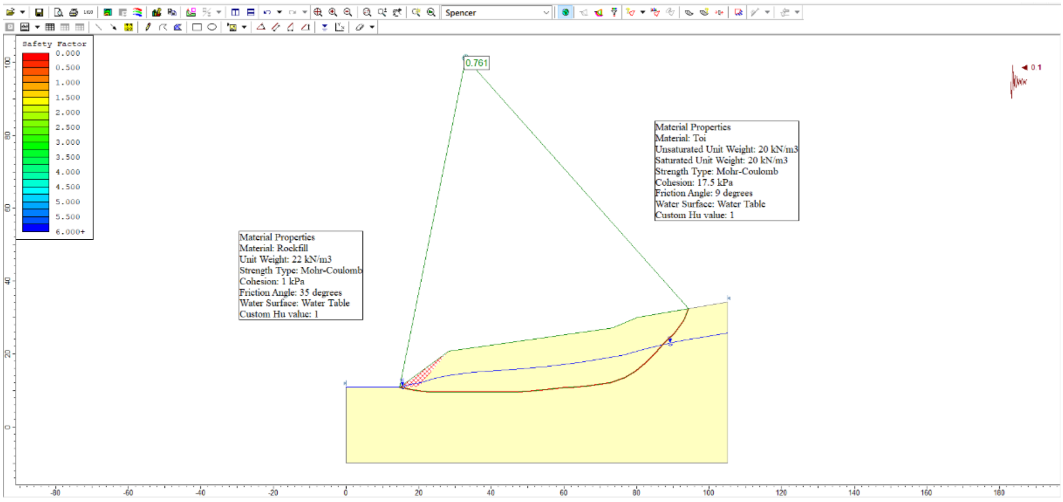
Figure 5.41. Stability analysis of KM: 113+120 cut slope with toe buttressing solution via Janbu corrected method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.42. Stability analysis of KM: 113+120 cut slope with toe buttressing solution via GLE/Morgenstern-Price method

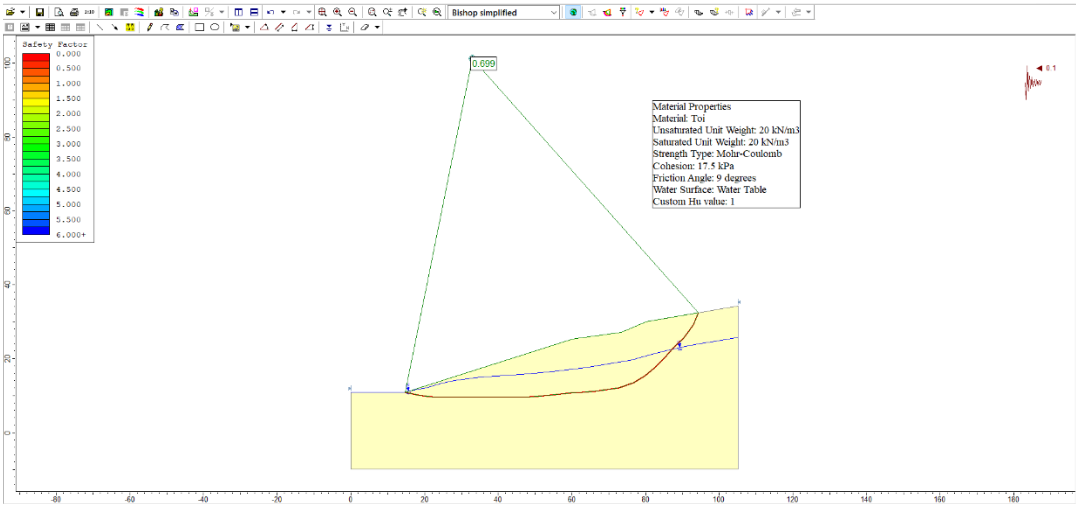


Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

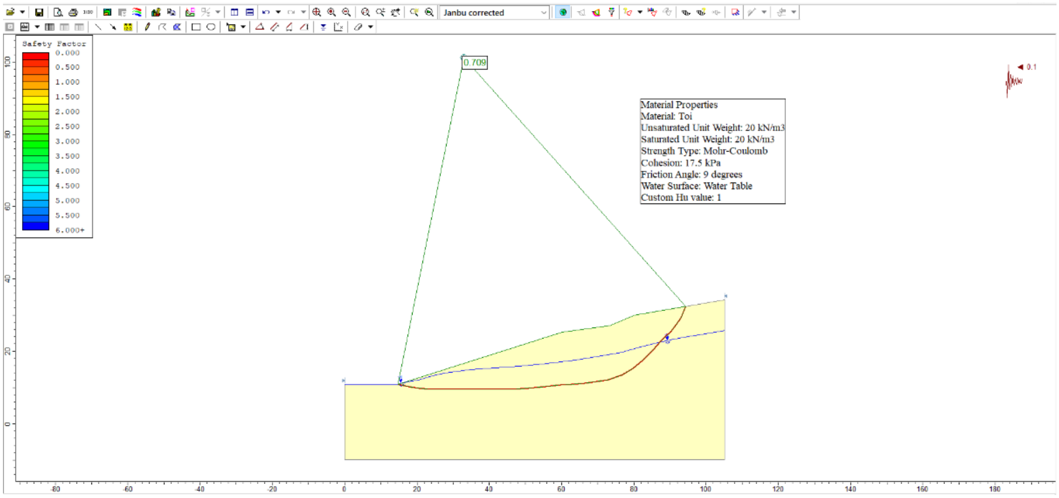
Figure 5.43. Stability analysis of KM: 113+120 cut slope with toe buttressing solution via Spencer method

Fourthly, slope flattening solution was tried. The original slope was flattened by excavating some material throughout the slope so that new slope has an inclination of H/V=3/1. The flattening alternative neither satisfed the value of factor of safety 1.1 nor turned out to be a solution for KM:113+120 cut slope. Worse still, the factor of safety fell further down far more than the initial value due to enlarging of the slope forming material susceptible to landsliding (Figures 5.44-5.47).



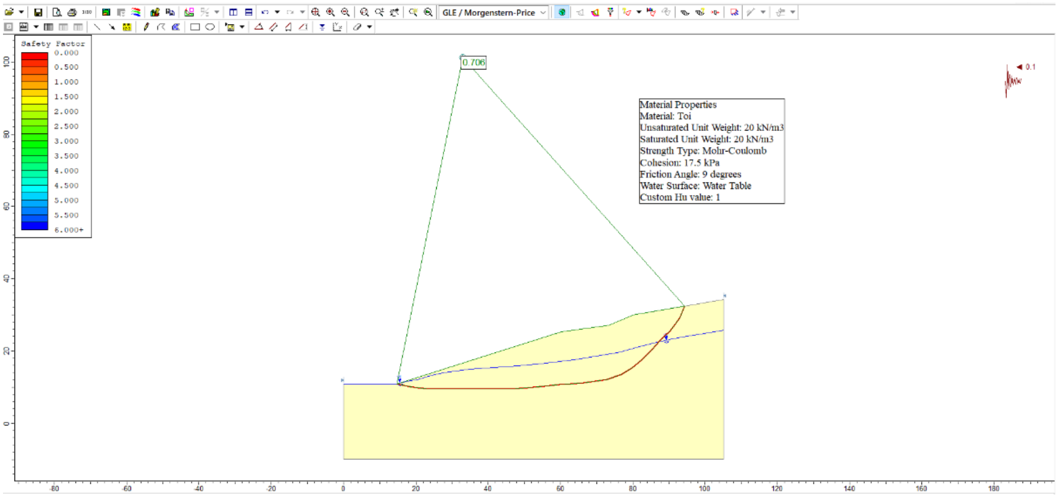
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.44. Stability analysis of KM: 113+120 cut slope with flattening solution via Bishop simplified method



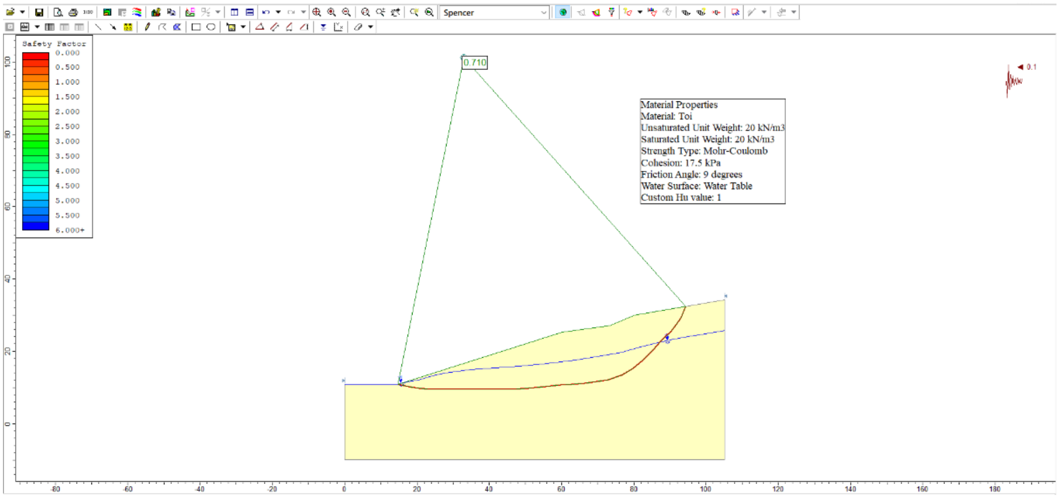
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.45. Stability analysis of KM: 113+120 cut slope with flattening solution via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

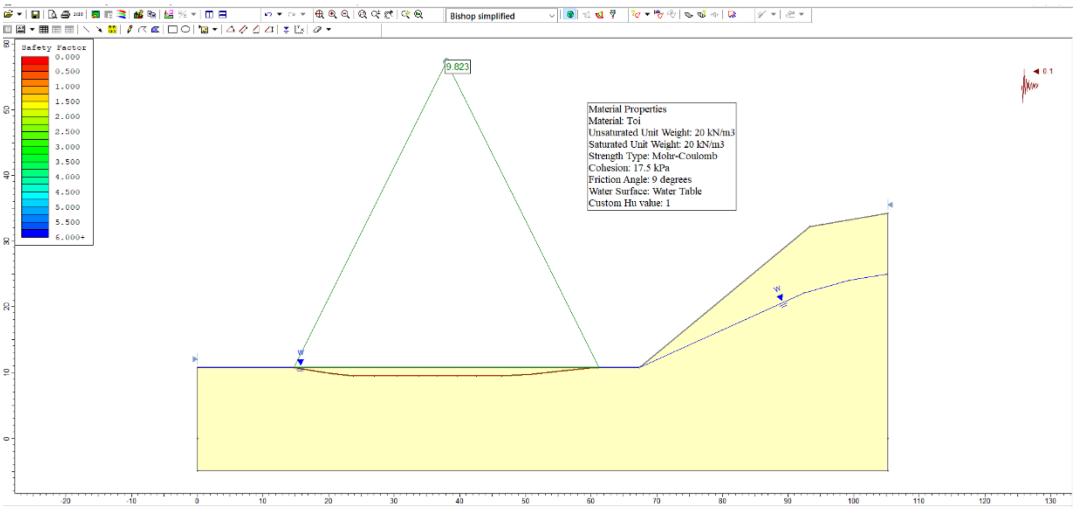
Figure 5.46. Stability analysis of KM: 113+120 cut slope with flattening solution via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

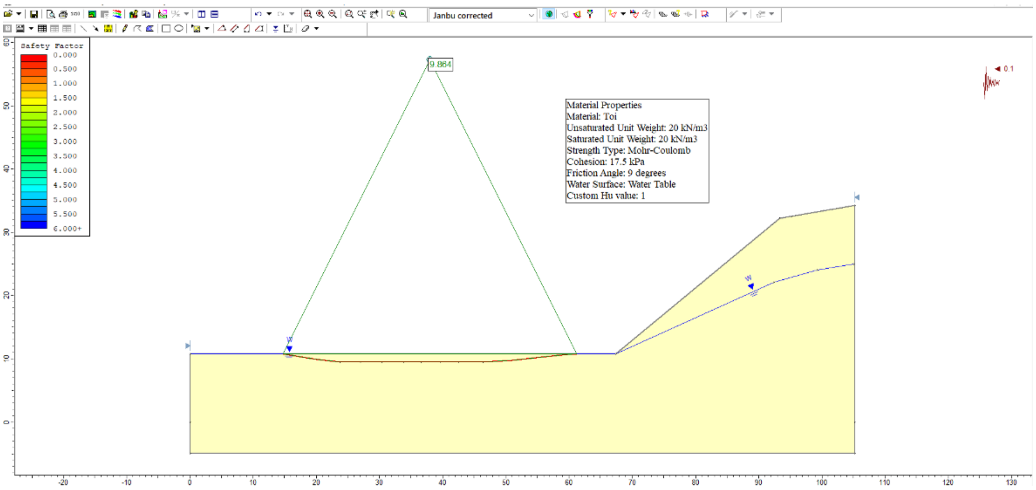
Figure 5.47. Stability analysis of KM: 113+120 cut slope with flattening solution via Spencer method

Fifthly, solution of removal of sliding material was tried. The original slope was excavated up till the failure surface in the way that the same inclination of H/V = 3/2 was satisfied after removal. Relatively high safety factors for the reshaped slopes were obtained using different method of analyses (Figures 5.48-5.51). Therefore, removal of sliding material turned out to be an alternative solution for KM:113+120 cut slope.



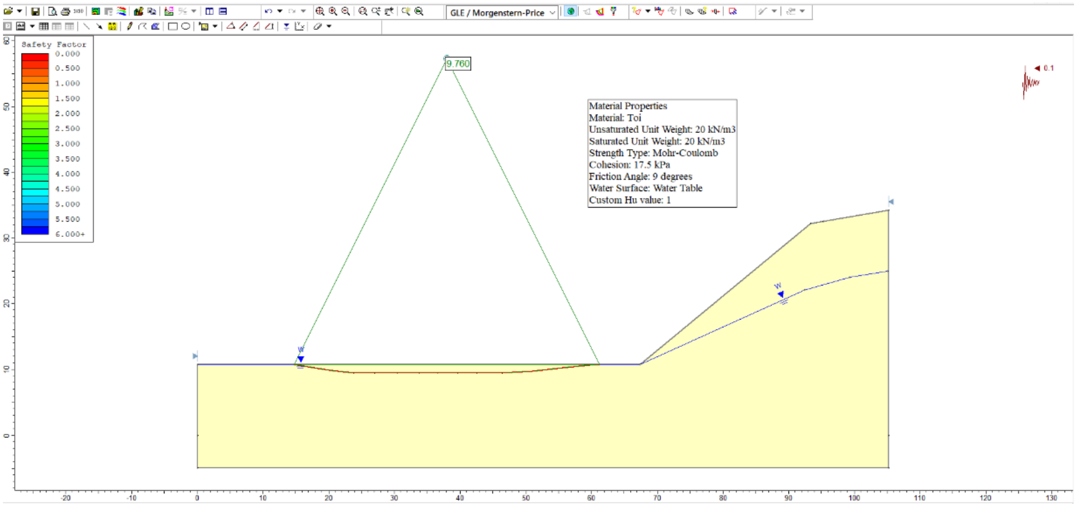
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.48. Stability analysis of KM: 113+120 cut slope with removal of sliding material solution via Bishop simplified method



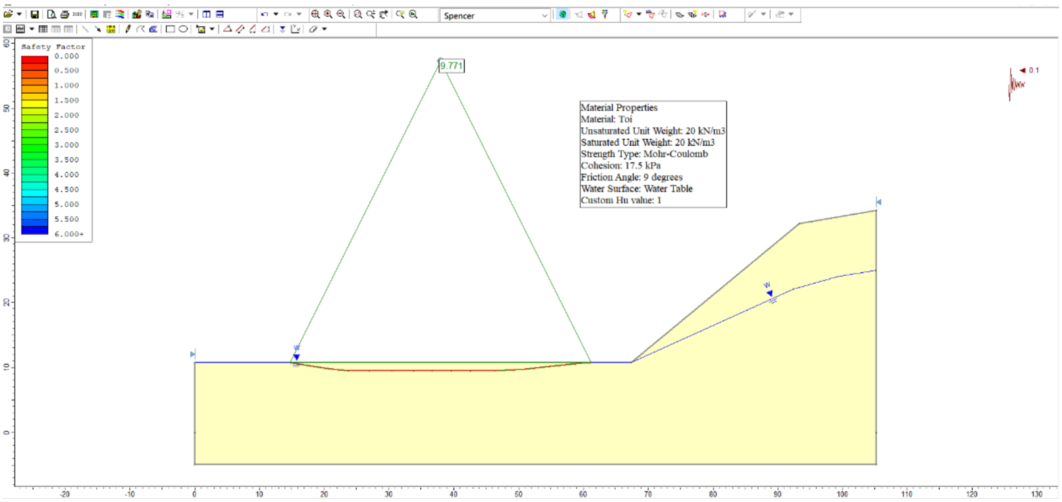
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.49. Stability analysis of KM: 113+120 cut slope with removal of sliding material solution via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

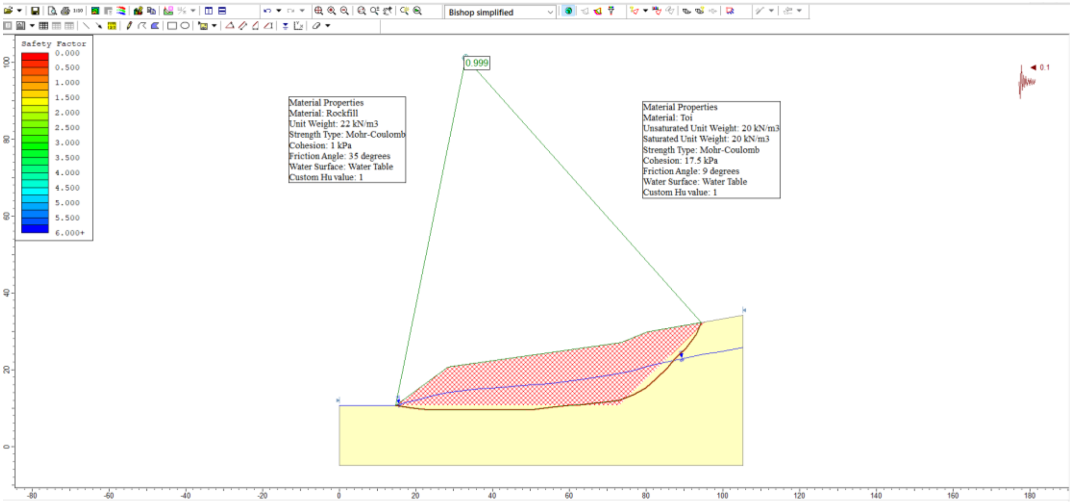
Figure 5.50. Stability analysis of KM: 113+120 cut slope with removal of sliding material solution via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.51. Stability analysis of KM: 113+120 cut slope with removal of sliding material solution via Spencer method

Lastly, solution of removal of sliding material and filling with rock was tried. The original slope was excavated up till the failure surface and filled with rock in the way that the same inclination of H/V = 3/2 was satisfied after removal and filling with rock. Relatively low safety factors of the flattened slope were obtained for all the methods except for Spencer method of analyses (Figures 5.52-5.55). Therefore, removal of sliding material and filling with rock accepted to be an alternative solution for KM:113+120 cut slope.

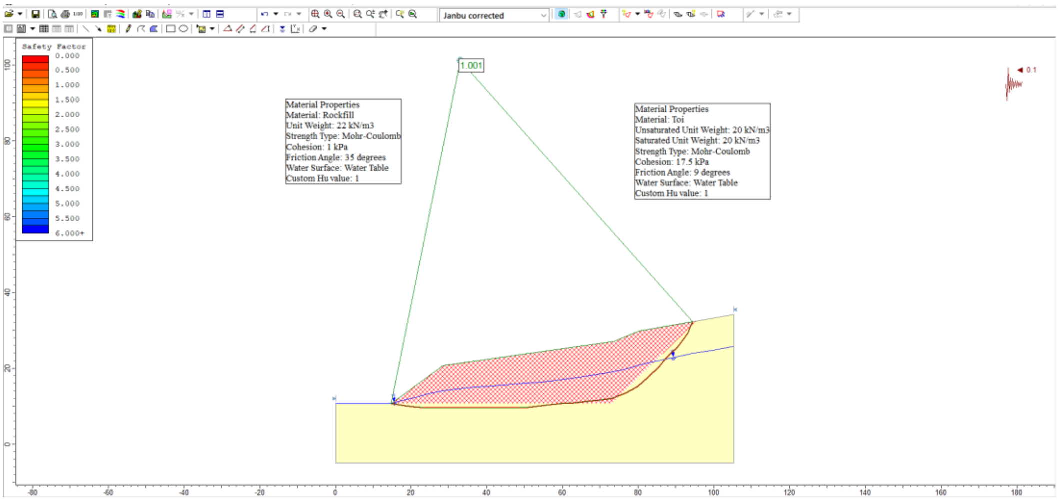


Bishop simplified

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.52. Stability analysis of KM: 113+120 cut slope with removal of sliding material+filling with rock solution via Bishop simplified method

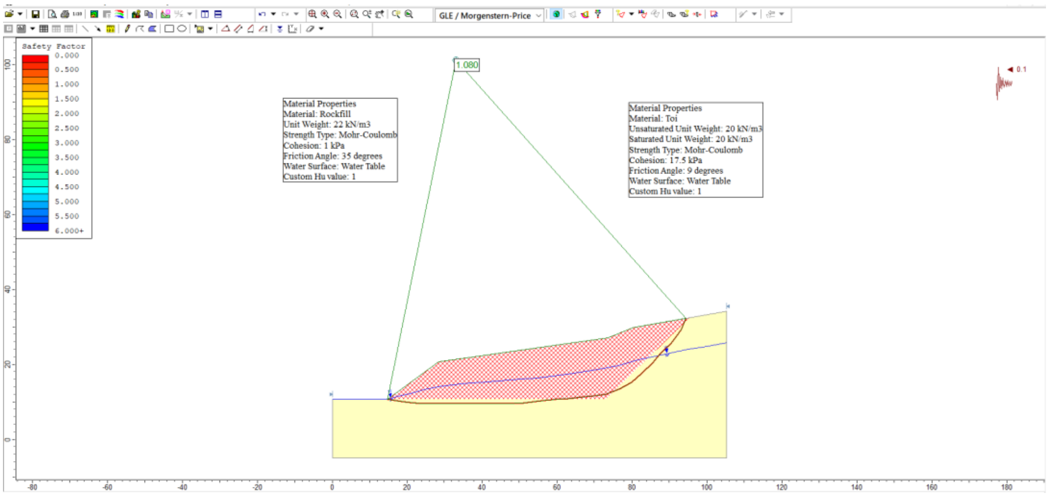


Janbu corrected

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

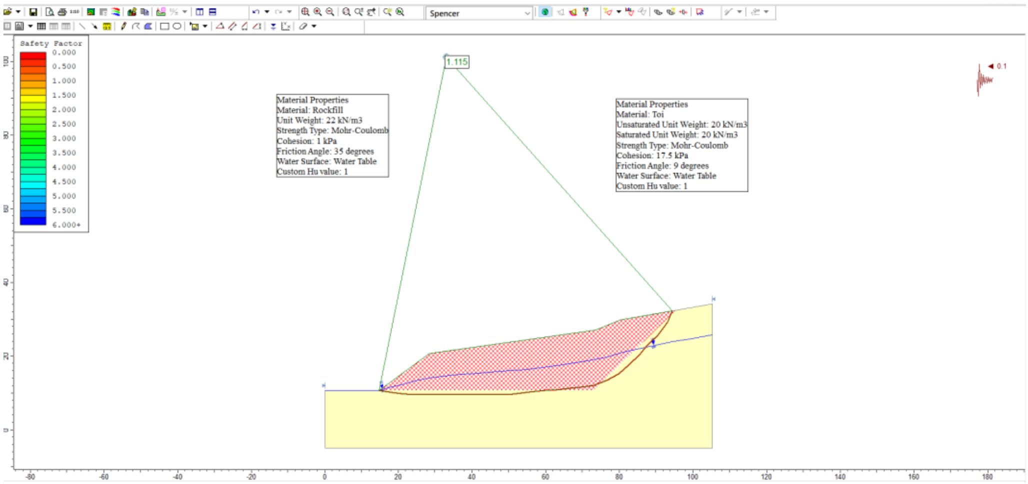
Figure 5.53. Stability analysis of KM: 113+120 cut slope with removal of sliding material+filling with rock solution via Janbu corrected method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.54. Stability analysis of KM: 113+120 cut slope with removal of sliding material+filling with rock solution via GLE/Morgenstern-Price method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

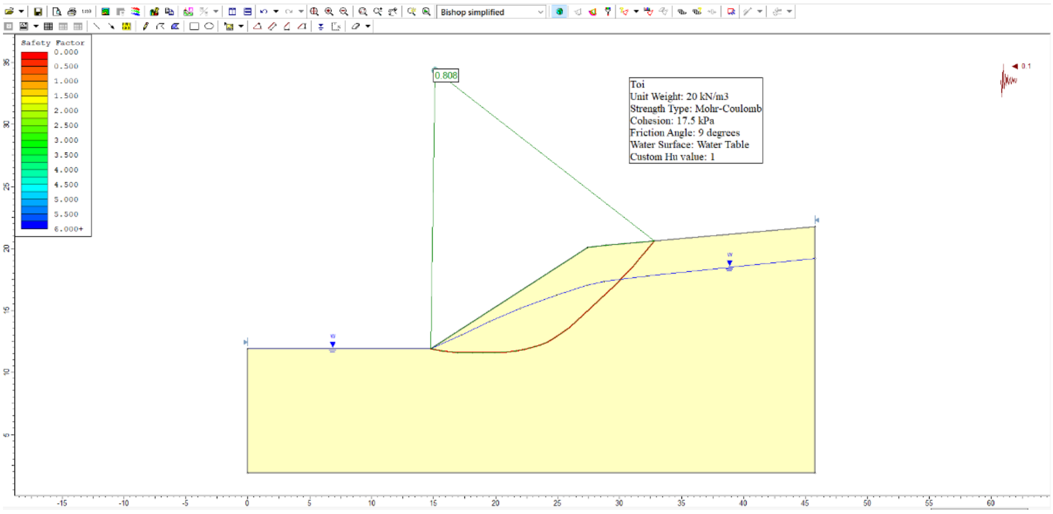
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.55. Stability analysis of KM: 113+120 cut slope with removal of sliding material+filling with rock solution via Spencer method

5.2.3. KM:121+200 Cut Slope

In order to survey the stability of KM:121+200 cut slope, slope stability analyses of the KM:121+200 cut slope were done in accordance with Bishop (1955), Janbu

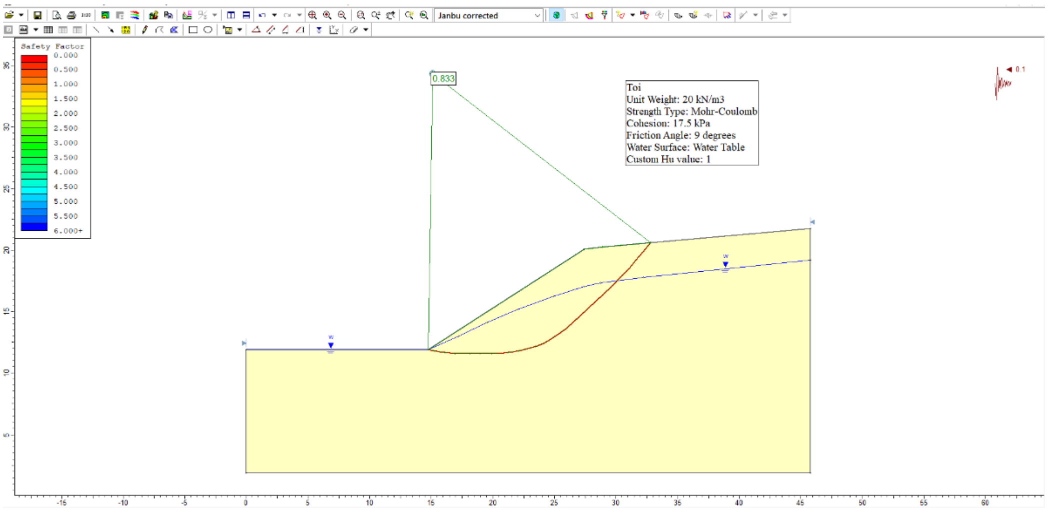
(1968), Morgenstern and Price (1965) and Spencer (1967) methods (Figures 5.56-5.59).



Bishop simplified

Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

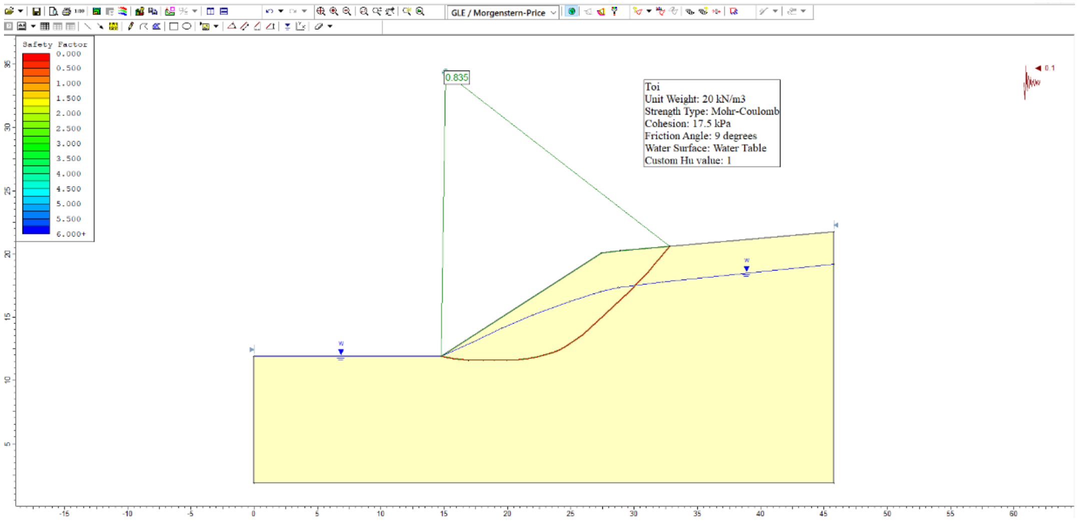
Figure 5.56. Stability analysis of KM: 121+200 cut slope based on back analysis data via Bishop simplified method



Janbu corrected

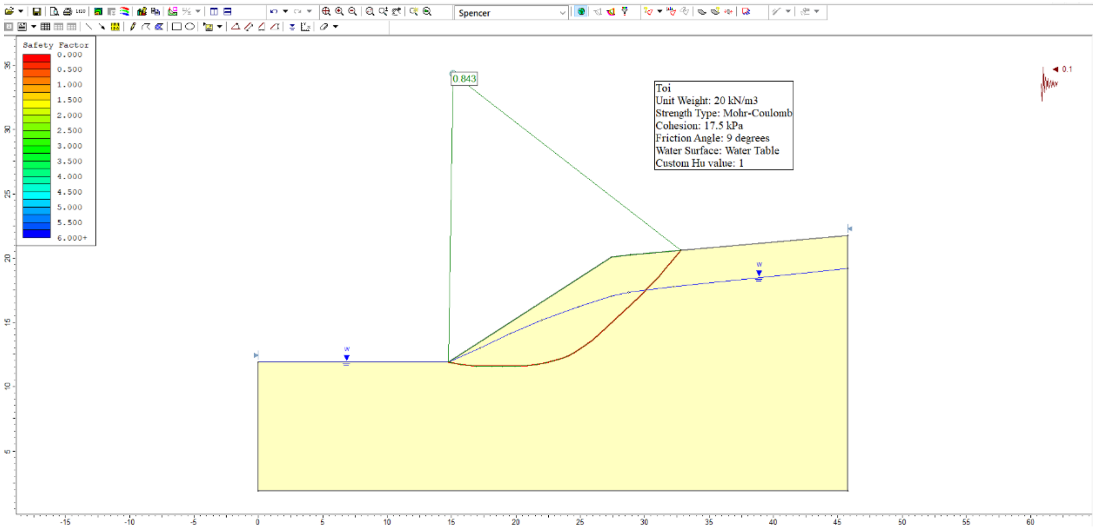
Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.57. Stability analysis of KM: 121+200 cut slope based on back analysis data via Janbu corrected method



Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

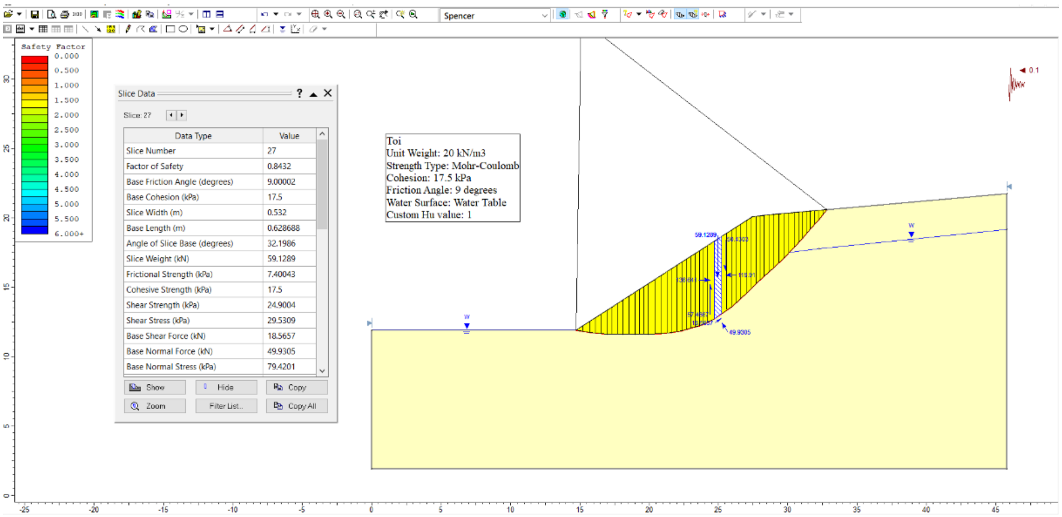
Figure 5.58. Stability analysis of KM: 121+200 cut slope based on back analysis data via GLE/Morgenstern-Price method



Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.59. Stability analysis of KM: 121+200 cut slope based on back analysis data via Spencer method

As a remedial measure, firstly piling solution was considered. In order to decide where to drive the pile, slice of highest forces was found. Slice number of 27 found to be the most critical one with the highest forces (Figure 5.60).



Slice Data

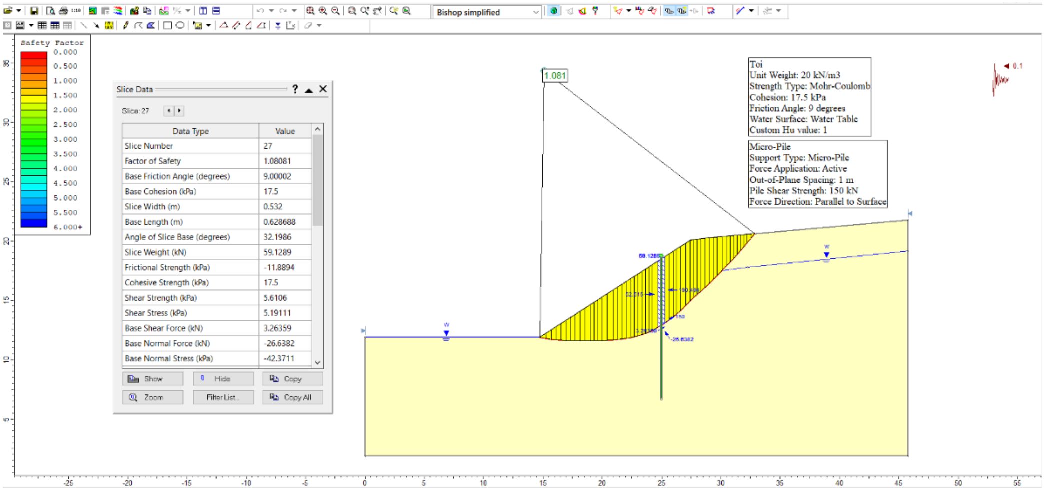
Slice: 27

Data Type	Value
Slice Number	27
Factor of Safety	0.8432
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.532
Base Length (m)	0.628688
Angle of Slice Base (degrees)	32.1986
Slice Weight (kN)	59.1289
Frictional Strength (kPa)	7.40043
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	24.9004
Shear Stress (kPa)	29.5309
Base Shear Force (kN)	18.5657
Base Normal Force (kN)	49.9305
Base Normal Stress (kPa)	79.4201

Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.60. Forces acting on slice of highest forces (Slice #27) at KM:121+200 cut slope

After a few trials, a pile with a shear strength of 150 kN and length of 12 m satisfied the value of factor of safety 1.1 for KM:121+200 cut slope by all analysis methods where specifically FS was equal to 1.133 via Spencer method (Figures 5.61-5.64). Therefore, piling turned out to be a solution for KM:121+200 cut slope.



Slice Data

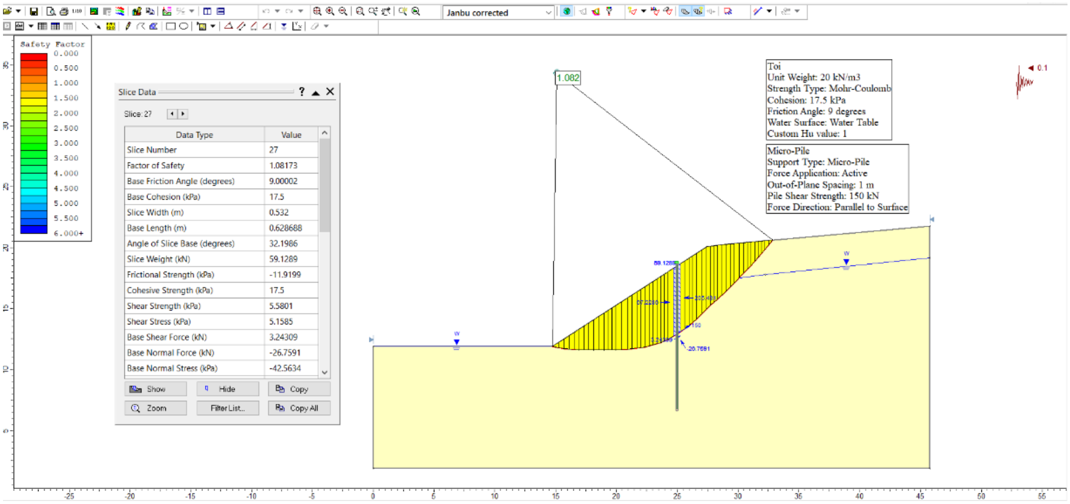
Slice: 27

Data Type	Value
Slice Number	27
Factor of Safety	1.08081
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.532
Base Length (m)	0.628688
Angle of Slice Base (degrees)	32.1986
Slice Weight (kN)	59.1289
Frictional Strength (kPa)	-11.8894
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	5.6106
Shear Stress (kPa)	5.19111
Base Shear Force (kN)	3.26359
Base Normal Force (kN)	-26.6382
Base Normal Stress (kPa)	-42.3711

Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 150 kN
Force Direction: Parallel to Surface

Figure 5.61. Stability analysis of KM: 121+200 cut slope with driven pile solution via Bishop simplified method



Slice Data

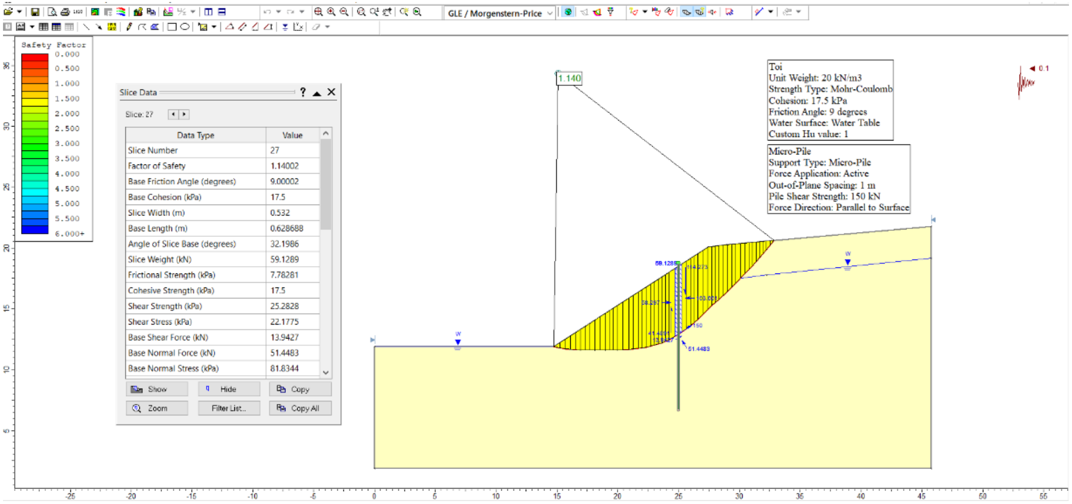
Slice: 27

Data Type	Value
Slice Number	27
Factor of Safety	1.08173
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.532
Base Length (m)	0.628688
Angle of Slice Base (degrees)	32.1986
Slice Weight (kN)	59.1289
Frictional Strength (kPa)	-11.9199
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	5.5801
Shear Stress (kPa)	5.1585
Base Shear Force (kN)	3.24309
Base Normal Force (kN)	-26.7591
Base Normal Stress (kPa)	-42.5634

Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 150 kN
Force Direction: Parallel to Surface

Figure 5.62. Stability analysis of KM: 121+200 cut slope with driven pile solution via Janbu corrected method



Slice Data

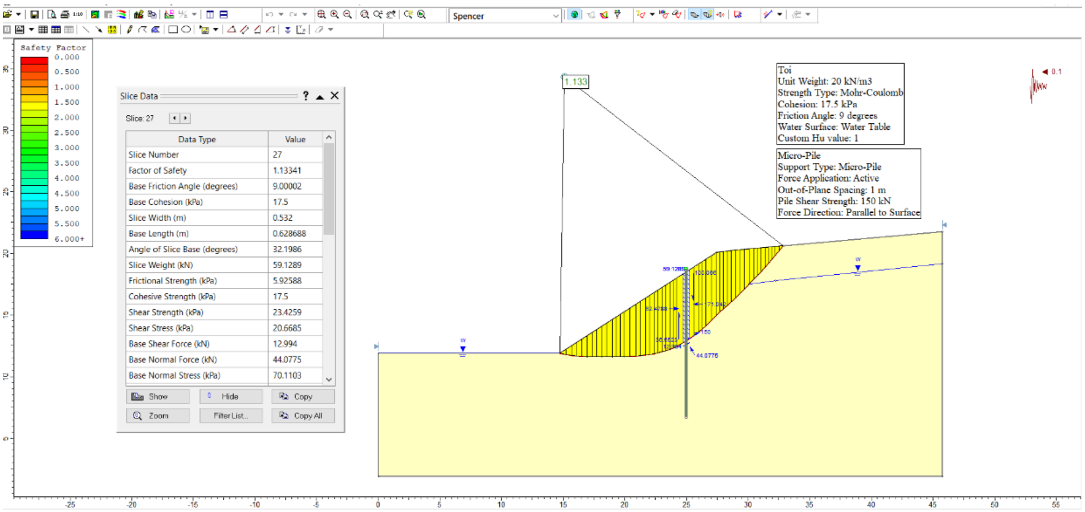
Slice: 27

Data Type	Value
Slice Number	27
Factor of Safety	1.14002
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.532
Base Length (m)	0.628688
Angle of Slice Base (degrees)	32.1986
Slice Weight (kN)	59.1289
Frictional Strength (kPa)	7.78281
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	25.2828
Shear Stress (kPa)	22.1775
Base Shear Force (kN)	13.9427
Base Normal Force (kN)	51.4483
Base Normal Stress (kPa)	81.8344

Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 150 kN
Force Direction: Parallel to Surface

Figure 5.63. Stability analysis of KM: 121+200 cut slope with driven pile solution via GLE/Morgenstern-Price method



Slice Data

Slice: 27

Data Type	Value
Slice Number	27
Factor of Safety	1.13341
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.532
Base Length (m)	0.628688
Angle of Slice Base (degrees)	32.1986
Slice Weight (kN)	59.1289
Frictional Strength (kPa)	5.92588
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	23.4259
Shear Stress (kPa)	20.6685
Base Shear Force (kN)	12.994
Base Normal Force (kN)	44.0775
Base Normal Stress (kPa)	70.1103

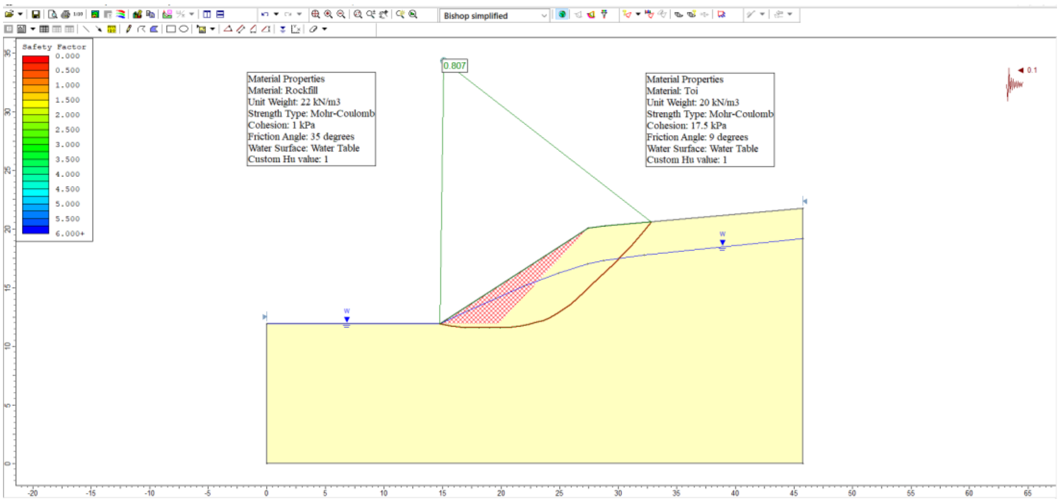
Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 150 kN
Force Direction: Parallel to Surface

Figure 5.64. Stability analysis of KM: 121+200 cut slope with driven pile solution via Spencer method

Benching did not turn out to be a solution for KM:121+200 slope due to the geometry, since the slope itself had a height of 10 m. For this reason, no benching trials were experienced for KM:121+200 cut slope.

As an alternetive solution, toe buttressing option was evaluated. Along the lower slope a strip of soil 5 m-wide at toe was removed and refilled with rock. Low safety factors of the slope were obtained for this case (Figure 5.65-5.68). For this reason, toe buttressing option did not turn out to be a solution for KM:121+200 cut slope.

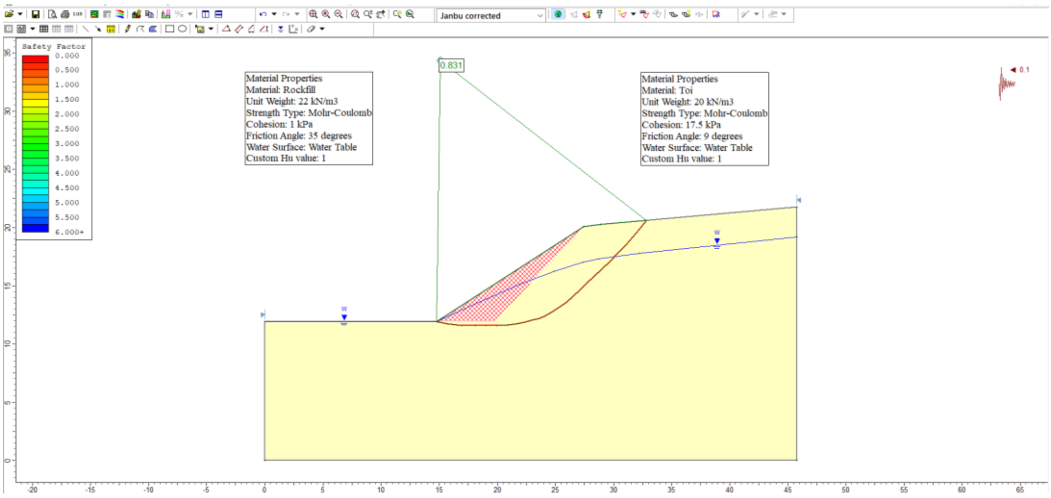


Bishop simplified

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.65. Stability analysis of KM: 121+200 cut slope with toe buttressing solution via Bishop simplified method

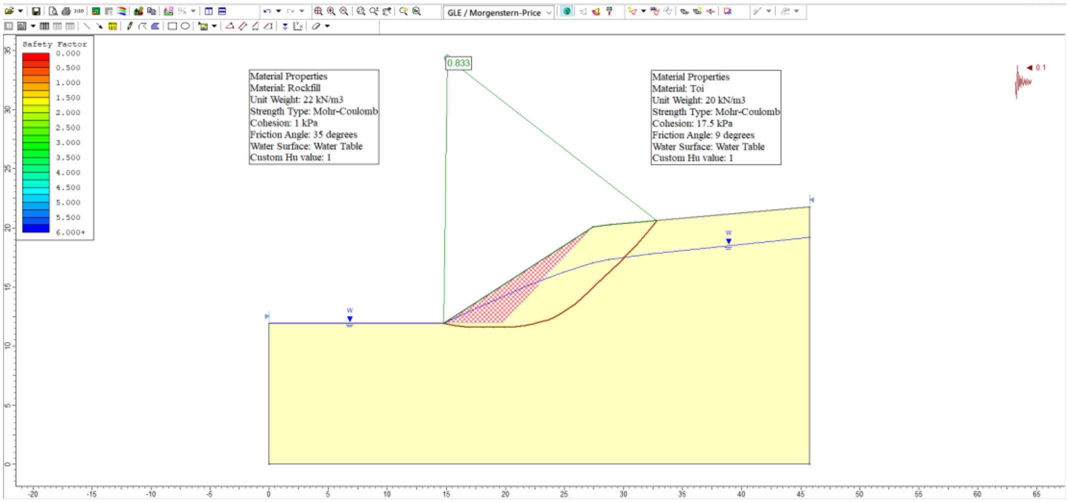


Janbu corrected

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

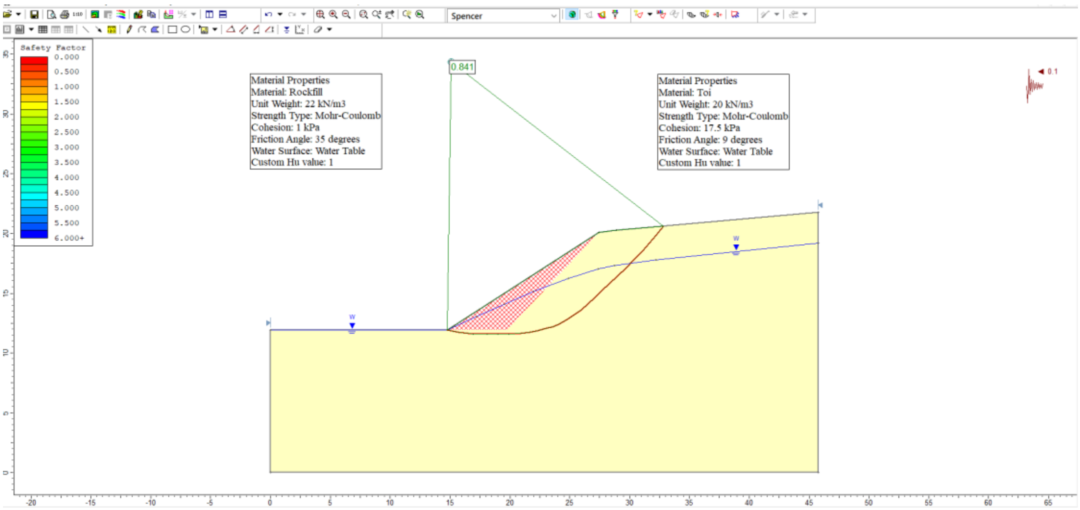
Figure 5.66. Stability analysis of KM: 121+200 cut slope with toe buttressing solution via Janbu corrected method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.67. Stability analysis of KM: 121+200 cut slope with toe buttressing solution via GLE/Morgenstern-Price method

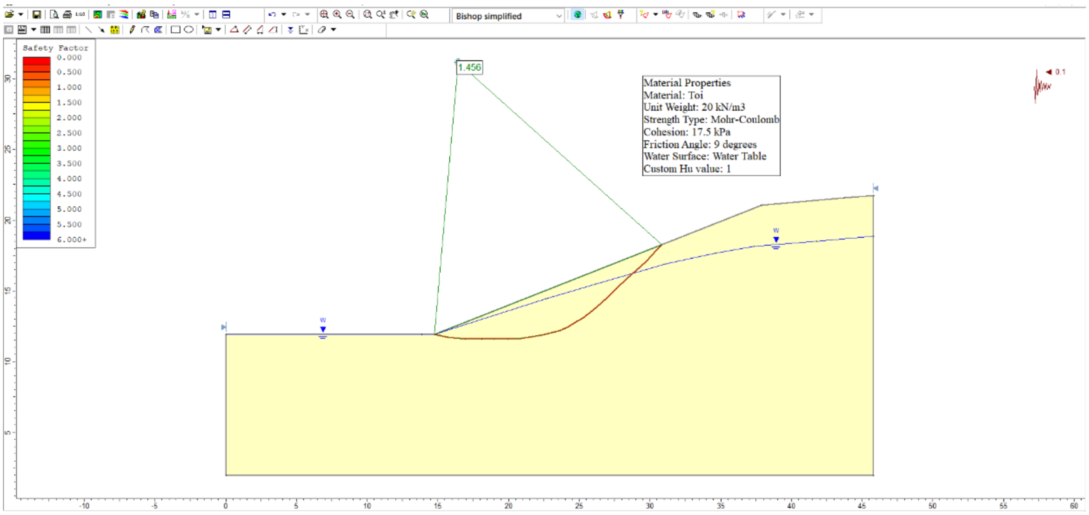


Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

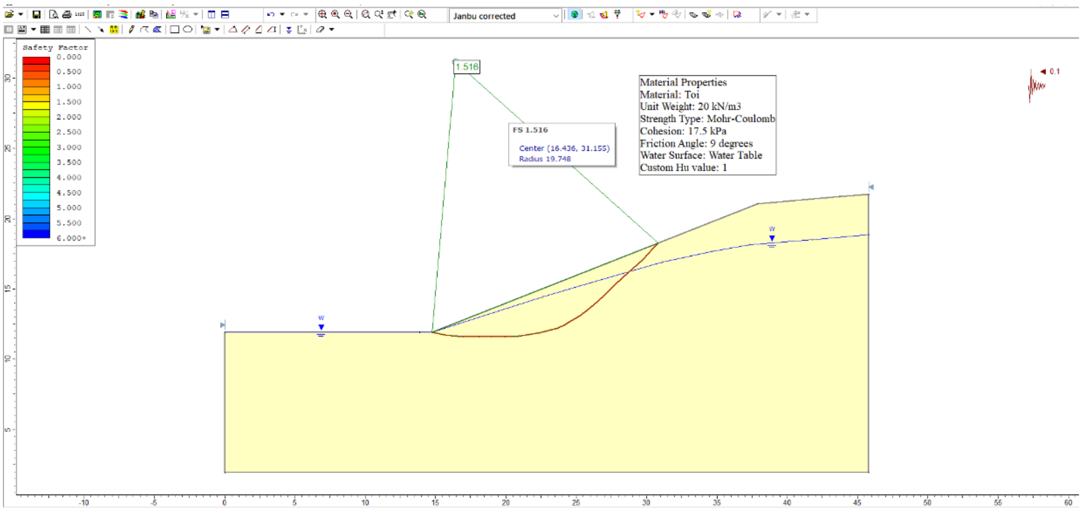
Figure 5.68. Stability analysis of KM: 121+200 cut slope with toe buttressing solution via Spencer method

Fourthly, slope flattening alternative was tried. The original slope was flattened by excavating some material throughout the slope so that new slope has an inclination of H/V=3/1. Stability analyses of the slope indicate that slope flattening yields factor of safeties significantly larger than 1.1. Therefore, it turns out to be a solution for KM:121+200 cut slope (Figures 5.69-5.72).



Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.69. Stability analysis of KM: 121+200 cut slope with flattening solution via Bishop simplified method

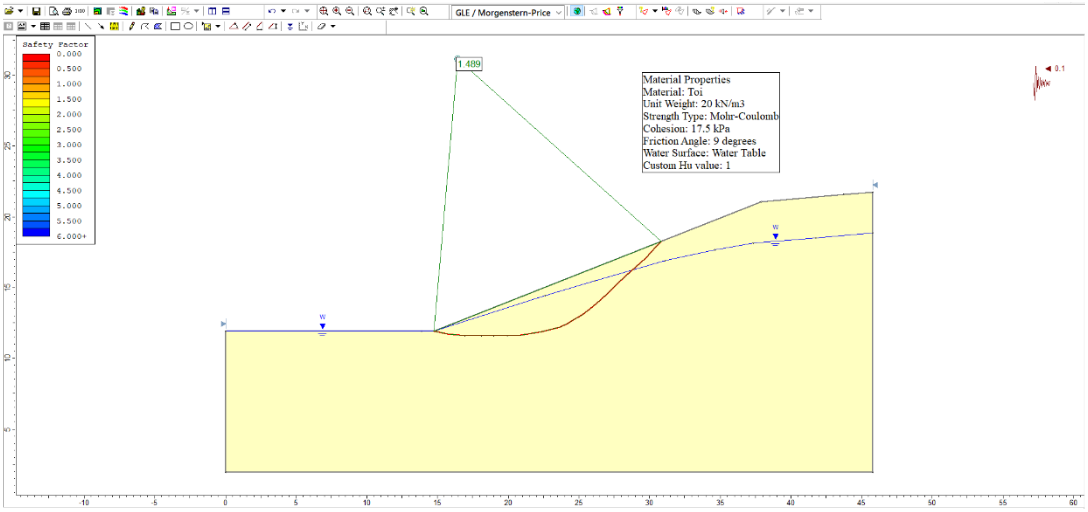


FS 1.516

Center (16.436, 31.155)
Radius 19.748

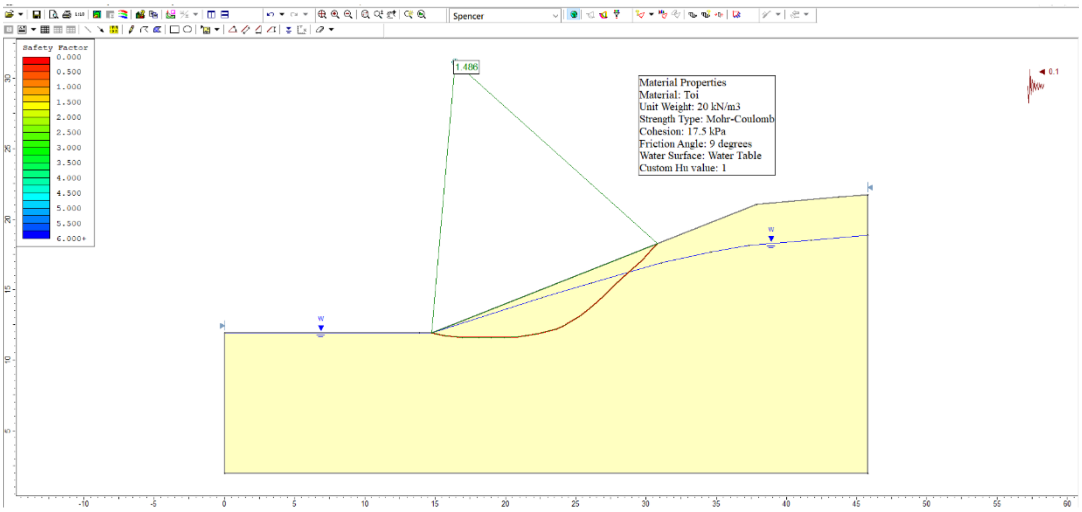
Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.70. Stability analysis of KM: 121+200 cut slope with flattening solution via Janbu corrected method



Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

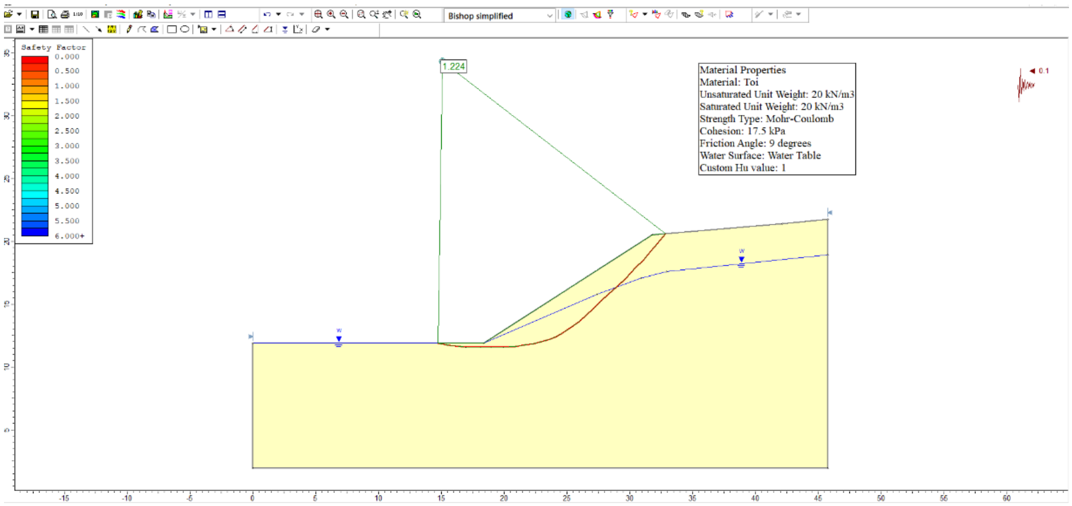
Figure 5.71. Stability analysis of KM: 121+200 cut slope with flattening solution via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.72. Stability analysis of KM: 121+200 cut slope with flattening solution via Spencer method

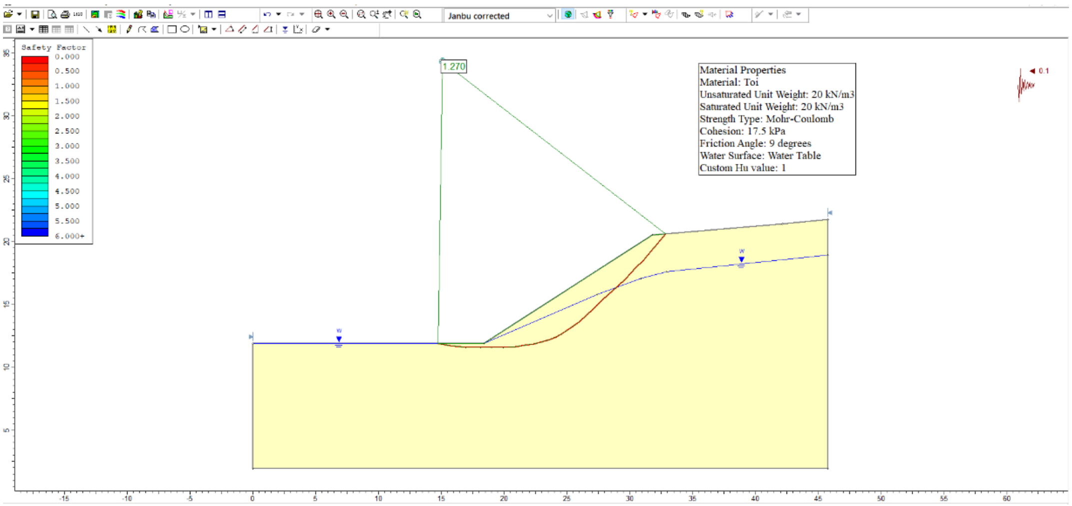
Fifthly, solution of removal of sliding material was tried. The original slope was excavated 2,5 m in width and reshaped in the way that the same inclination of H/V = 3/2 was satisfied after removal. Relatively high safety factors for the reshaped slopes were obtained using different method of analyses (Figures 5.73-5.76). Therefore, removal of sliding material turned out to be an alternative solution for KM:121+200 cut slope.



Bishop simplified

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

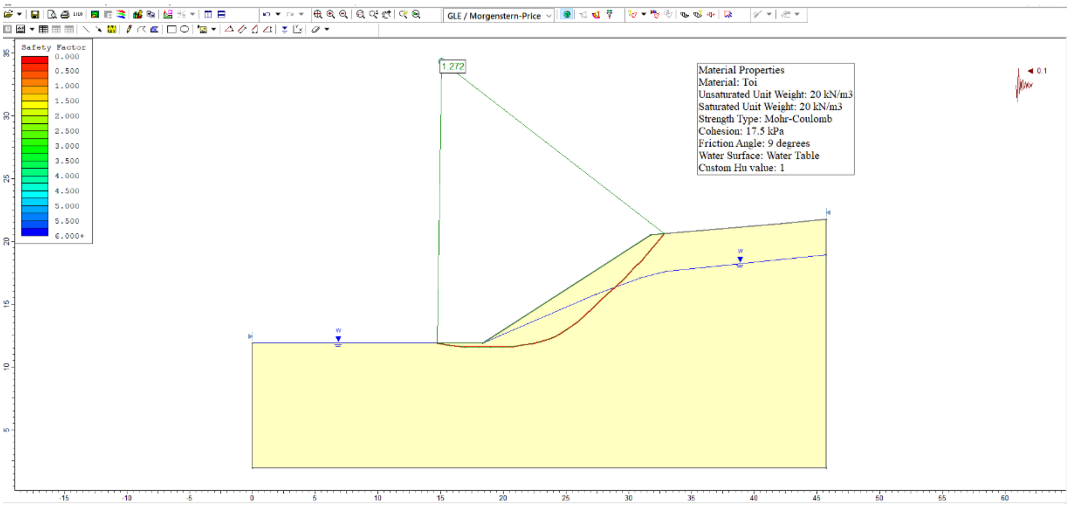
Figure 5.73. Stability analysis of KM: 121+200 cut slope with removal of sliding material solution via Bishop simplified method



Janbu corrected

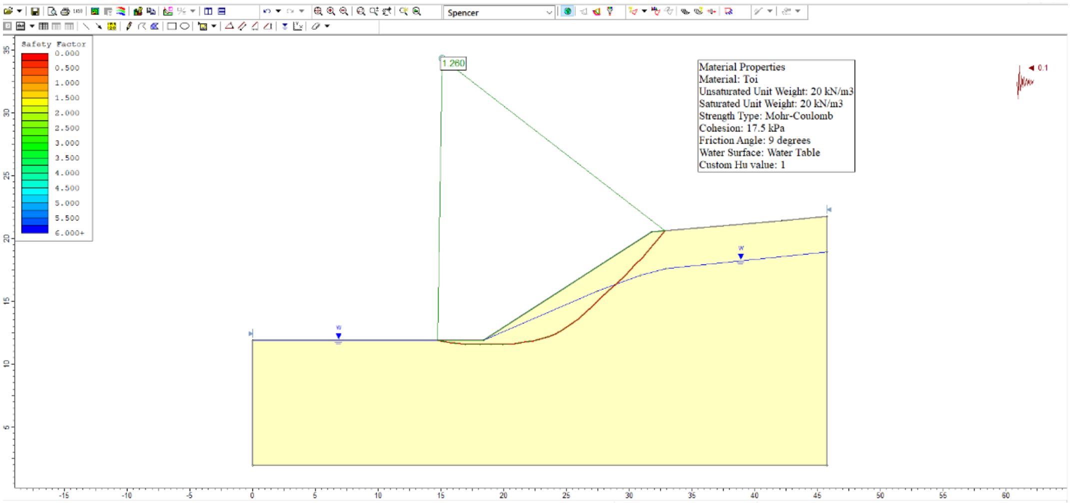
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.74. Stability analysis of KM: 121+200 cut slope with removal of sliding material solution via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

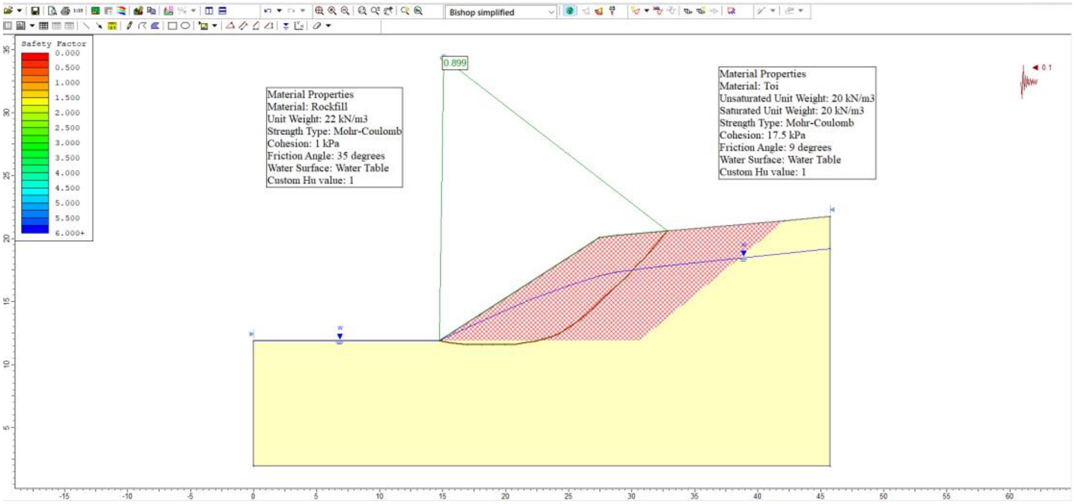
Figure 5.75. Stability analysis of KM: 121+200 cut slope with removal of sliding material solution via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.76. Stability analysis of KM: 121+200 cut slope with removal of sliding material solution via Spencer method

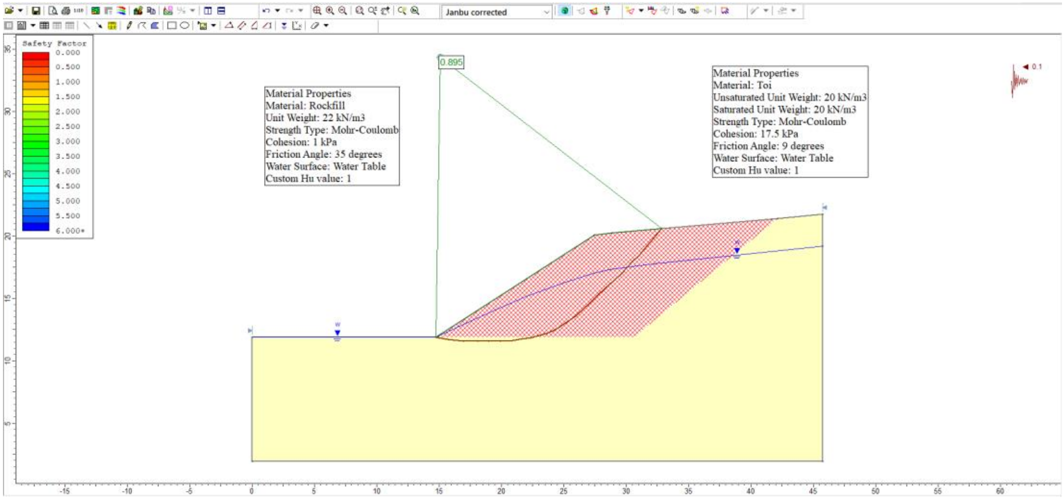
Lastly, solution of removal of sliding material and filling with rock was tried. The original slope was excavated 15 m in width and filled with rock in the way that the same inclination of H/V = 3/2 was satisfied after removal and filling with rock. Relatively low safety factors of the flattened slope were obtained for all methods of analyses (Figures 5.77-5.80). Therefore, removal of sliding material and filling with rock was not accepted to be an alternative solution for KM:121+200 cut slope.



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

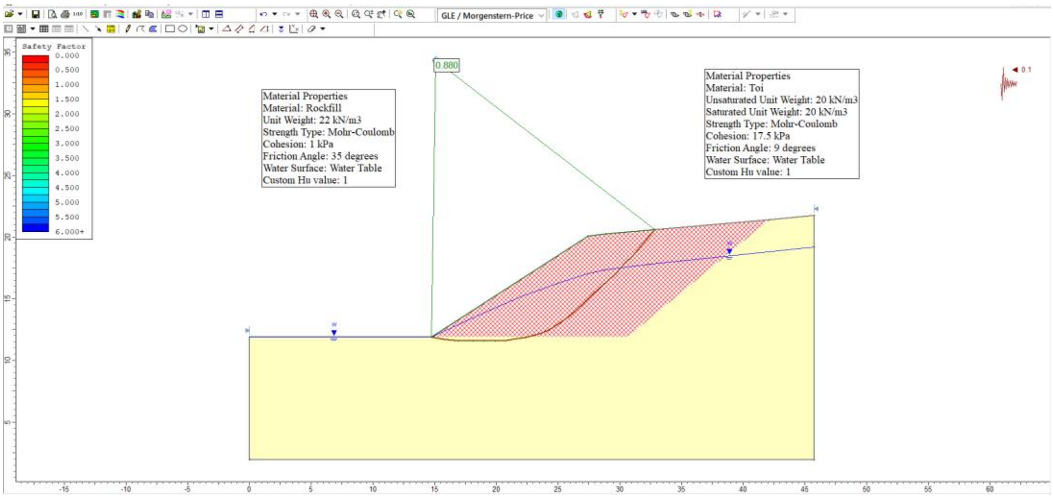
Figure 5.77. Stability analysis of KM: 121+200 cut slope with removal of sliding material+filling with rock solution via Bishop simplified method



Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

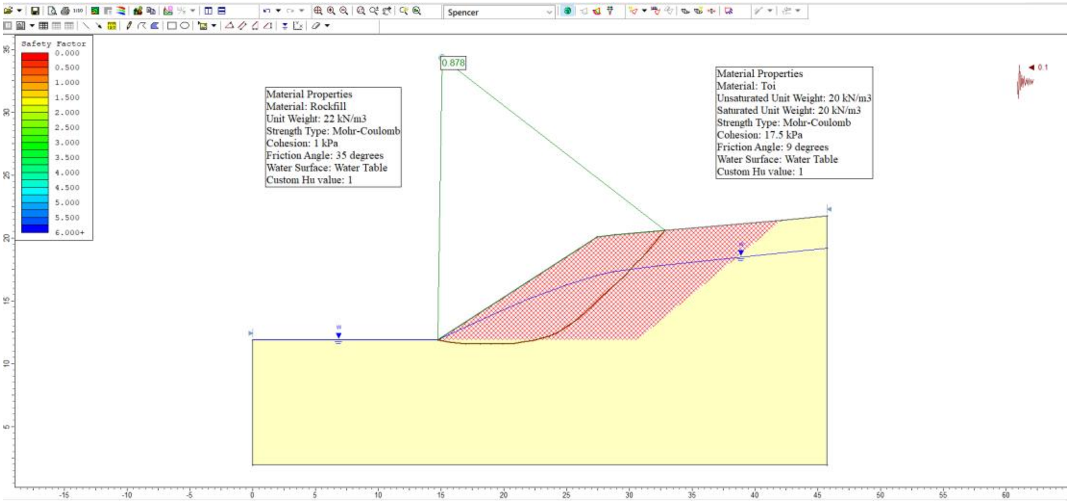
Figure 5.78. Stability analysis of KM: 121+200 cut slope with removal of sliding material+filling with rock solution via Janbu corrected method


GLE / Morgenstern-Price

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.79. Stability analysis of KM: 121+200 cut slope with removal of sliding material+filling with rock solution via GLE/Morgenstern-Price method


Spencer

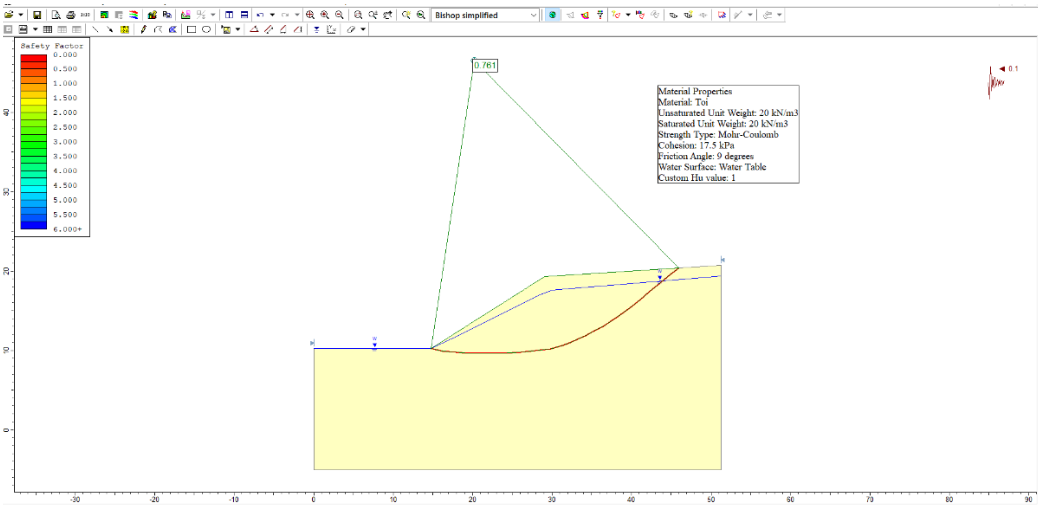
Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.80. Stability analysis of KM: 121+200 cut slope with removal of sliding material+filling with rock solution via Spencer method

5.2.4. KM:128+630 Cut Slope

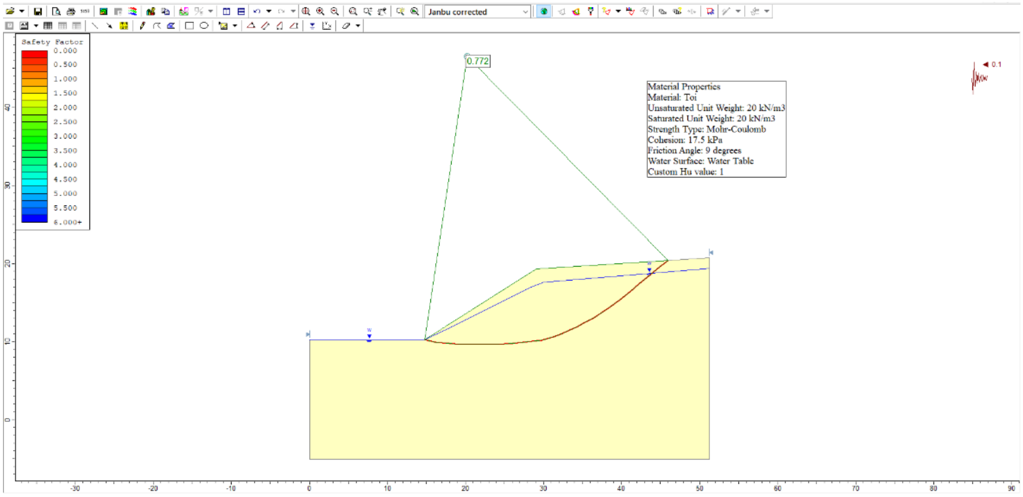
In order to survey the stability of KM:128+630 cut slope, slope stability analyses via SLIDE software were performed using Bishop (1955), Janbu (1968), Morgenstern and Price (1965) and Spencer (1967) methods (Figures 5.81-5.84).



Bishop simplified

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

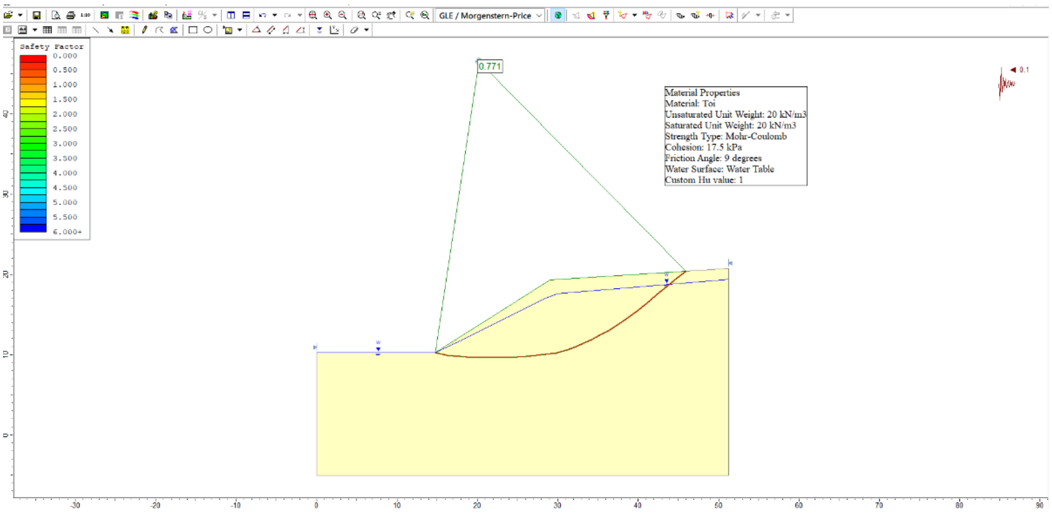
Figure 5.81. Stability analysis of KM: 128+630 cut slope based on back analysis data via Bishop simplified method



Janbu corrected

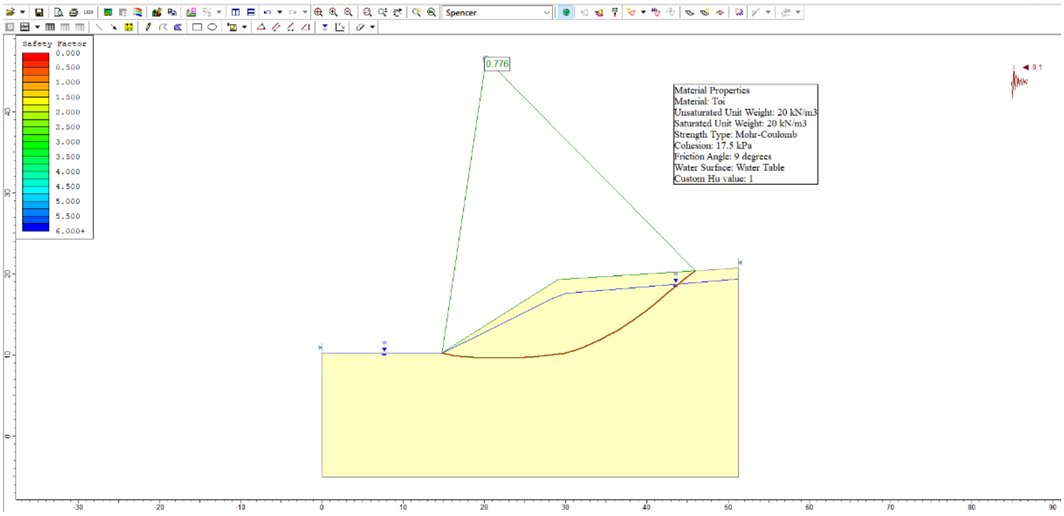
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.82. Stability analysis of KM: 128+630 cut slope based on back analysis data via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

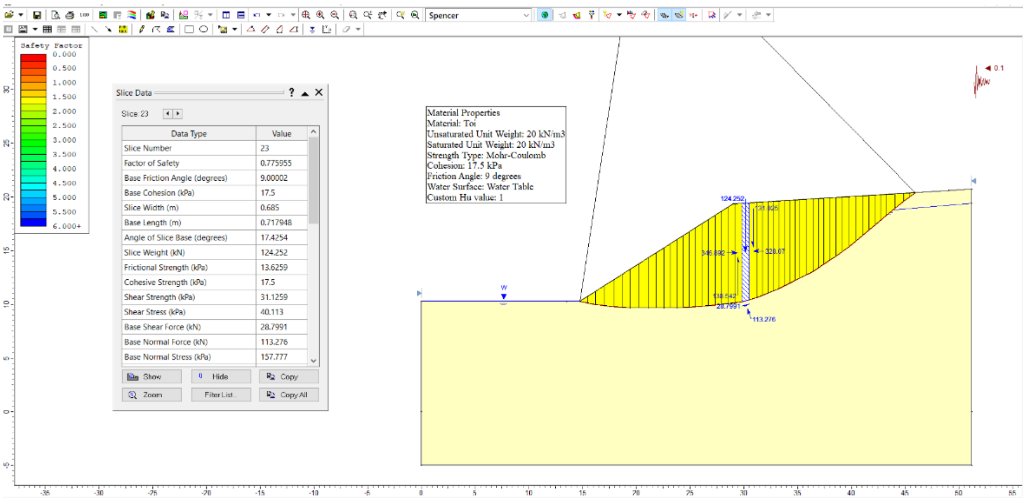
Figure 5.83. Stability analysis of KM: 128+630 cut slope based on back analysis data via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.84. Stability analysis of KM: 128+630 cut slope based on back analysis data via Spencer method

As a mitigation method, firstly piling solution was considered. In order to decide where to drive the pile, slice of highest forces (slice number 23 for this case) was found using SLIDE software (Figure 5.85).



Slice Data

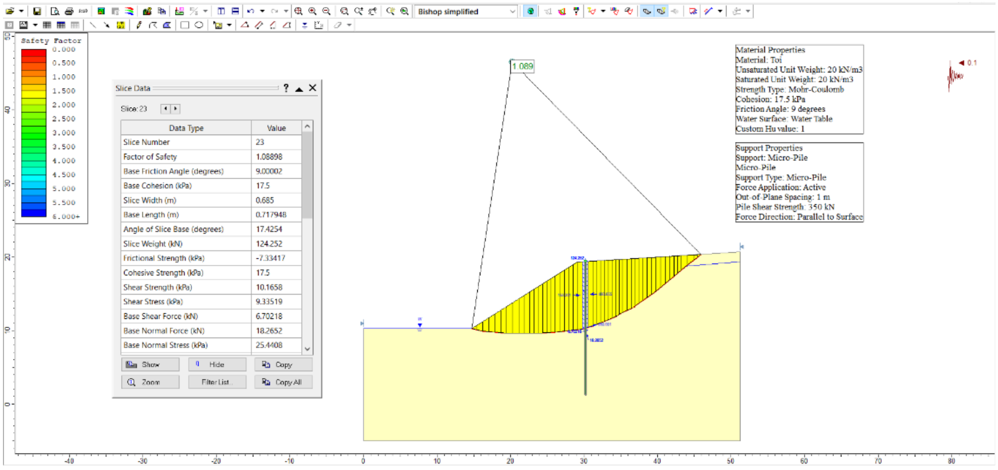
Slice: 23

Data Type	Value
Slice Number	23
Factor of Safety	0.775955
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.685
Base Length (m)	0.717948
Angle of Slice Base (degrees)	17.4254
Slice Weight (kN)	124.252
Frictional Strength (kPa)	13.6259
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	31.1259
Shear Stress (kPa)	40.113
Base Shear Force (kN)	28.7991
Base Normal Force (kN)	113.276
Base Normal Stress (kPa)	157.777

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.85. Forces acting on slice of highest forces (Slice #23) at KM:128+630 cut slope

After a few trials, a pile with a shear strength of 350 kN and length of 18 m was found to be satisfying the value of factor of safety 1.1 for KM:128+630 cut slope by all analysis methods where specifically FS was equal to 1.111 via Spencer method (Figures 5.86-5.89). For this reason, piling was turned out to be a solution for KM:128+630 cut slope.



Slice Data

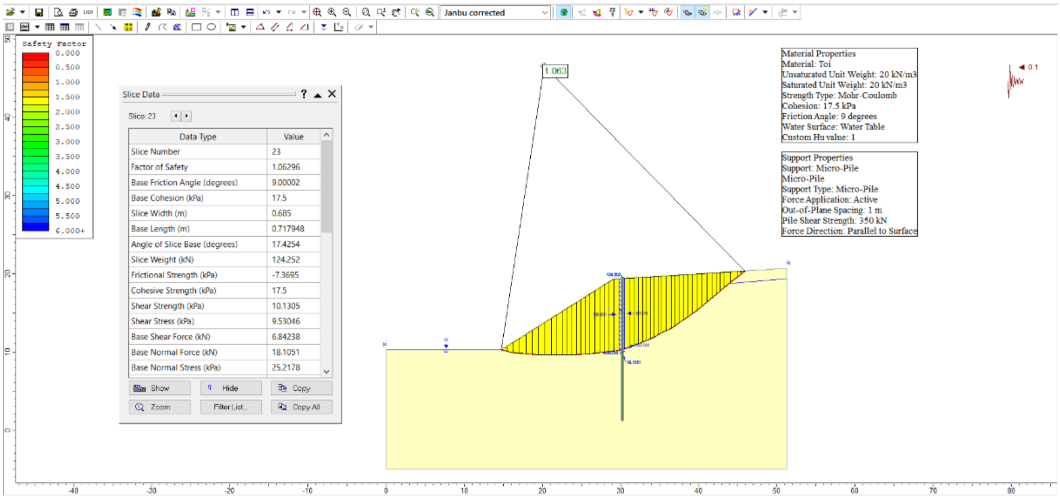
Slice: 23

Data Type	Value
Slice Number	23
Factor of Safety	1.08898
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.685
Base Length (m)	0.717948
Angle of Slice Base (degrees)	17.4254
Slice Weight (kN)	124.252
Frictional Strength (kPa)	-7.33417
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	10.1658
Shear Stress (kPa)	9.33519
Base Shear Force (kN)	6.70218
Base Normal Force (kN)	18.2652
Base Normal Stress (kPa)	25.4408

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 350 kN
Force Direction: Parallel to Surface

Figure 5.86. Stability analysis of KM: 128+630 cut slope with driven pile solution via Bishop simplified method



Slice Data

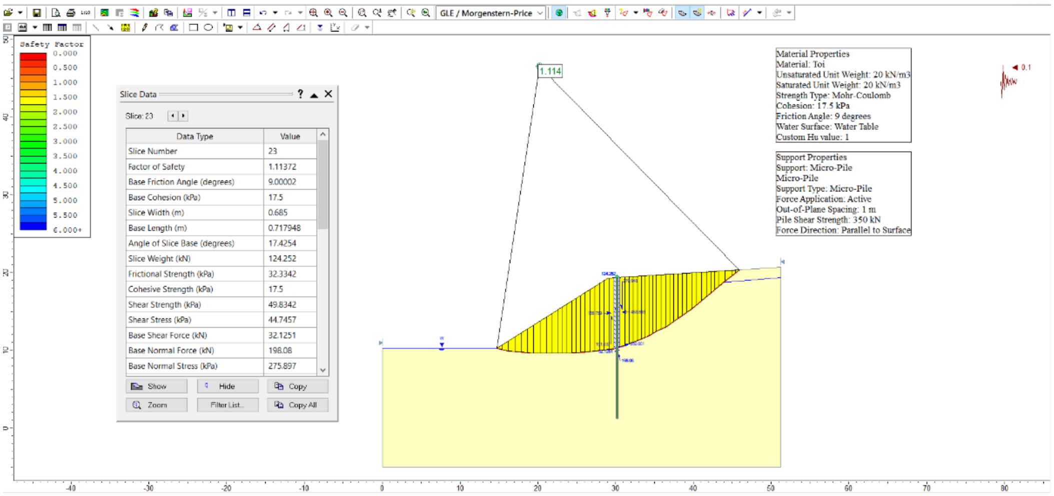
Slice: 23

Data Type	Value
Slice Number	23
Factor of Safety	1.06296
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.685
Base Length (m)	0.717948
Angle of Slice Base (degrees)	17.4254
Slice Weight (kN)	124.252
Frictional Strength (kPa)	-7.3695
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	10.1305
Shear Stress (kPa)	9.53046
Base Shear Force (kN)	6.84238
Base Normal Force (kN)	18.1051
Base Normal Stress (kPa)	25.2178

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 350 kN
Force Direction: Parallel to Surface

Figure 5.87. Stability analysis of KM: 128+630 cut slope with driven pile solution via Janbu corrected method



Slice Data

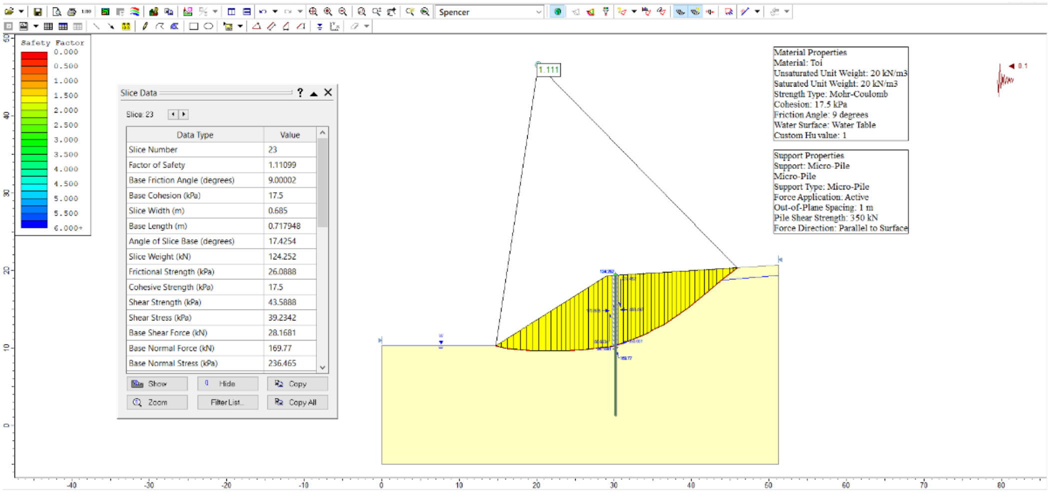
Slice: 23

Data Type	Value
Slice Number	23
Factor of Safety	1.11372
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.685
Base Length (m)	0.717948
Angle of Slice Base (degrees)	17.4254
Slice Weight (kN)	124.252
Frictional Strength (kPa)	32.3342
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	49.8342
Shear Stress (kPa)	44.7457
Base Shear Force (kN)	32.1251
Base Normal Force (kN)	198.08
Base Normal Stress (kPa)	275.897

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 350 kN
Force Direction: Parallel to Surface

Figure 5.88. Stability analysis of KM: 128+630 cut slope with driven pile solution via GLE/Morgenstern-Price method



Slice Data

Slice: 23

Data Type	Value
Slice Number	23
Factor of Safety	1.11099
Base Friction Angle (degrees)	9.00002
Base Cohesion (kPa)	17.5
Slice Width (m)	0.685
Base Length (m)	0.717948
Angle of Slice Base (degrees)	17.4254
Slice Weight (kN)	124.252
Frictional Strength (kPa)	26.0888
Cohesive Strength (kPa)	17.5
Shear Strength (kPa)	43.5888
Shear Stress (kPa)	39.2342
Base Shear Force (kN)	28.1681
Base Normal Force (kN)	169.77
Base Normal Stress (kPa)	236.465

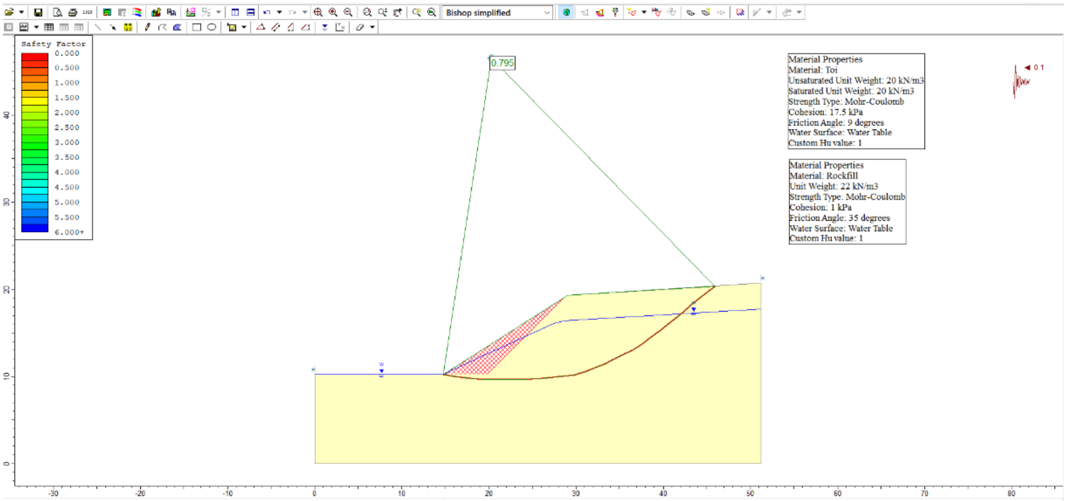
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Micro-Pile
Micro-Pile
Support Type: Micro-Pile
Force Application: Active
Out-of-Plane Spacing: 1 m
Pile Shear Strength: 350 kN
Force Direction: Parallel to Surface

Figure 5.89. Stability analysis of KM: 128+630 cut slope with driven pile solution via Spencer method

However, benching did not turn out to be a solution for KM:128+630 slope due to the geometry, since the slope itself has a height of 10m. For this reason, no benching trials were experienced for KM:128+630 slope.

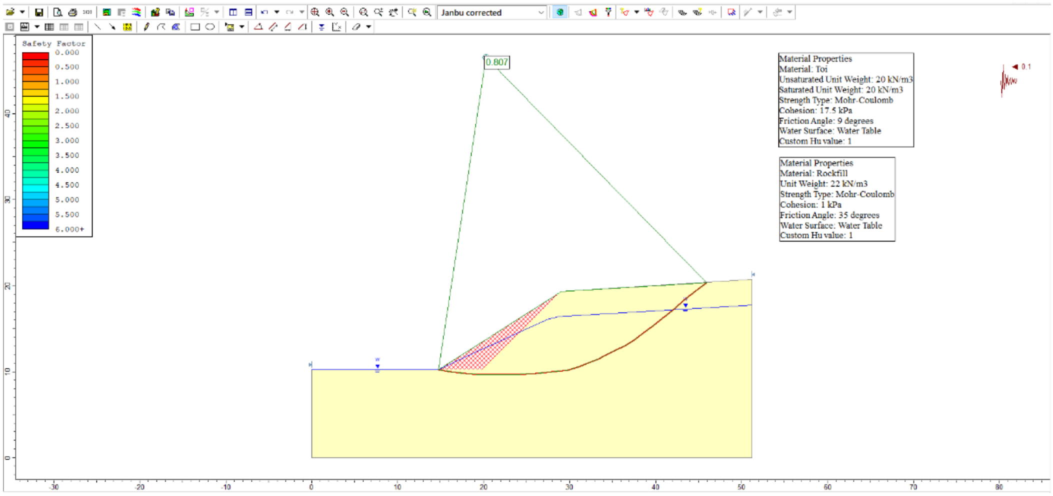
As an alternetive solution, toe buttressing option was evaluated. Along the lower slope a strip of soil 5 m-wide at toe was removed and refilled with rock. The slope stability analyses indicate that safety factor of the slope is less than the limiting value of FS=1.1 (Figures 5.90-5.93). Nevertheless, toe buttressing option did not turn out to be a solution for KM:128+630 cut slope.



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

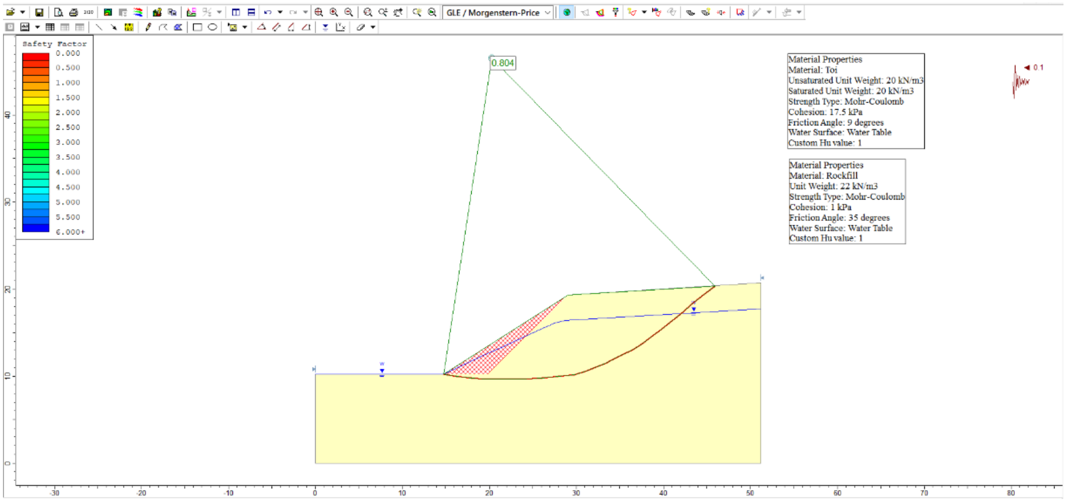
Figure 5.90. Stability analysis of KM: 128+630 cut slope with toe buttressing solution via Bishop simplified method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

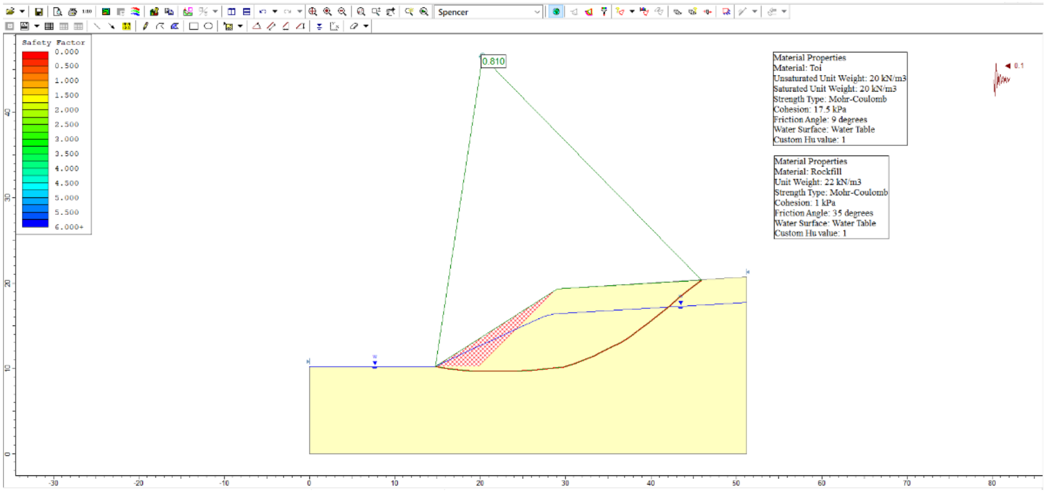
Figure 5.91. Stability analysis of KM: 128+630 cut slope with toe buttressing solution via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.92. Stability analysis of KM: 128+630 cut slope with toe buttressing solution via GLE/Morgenstern-Price method

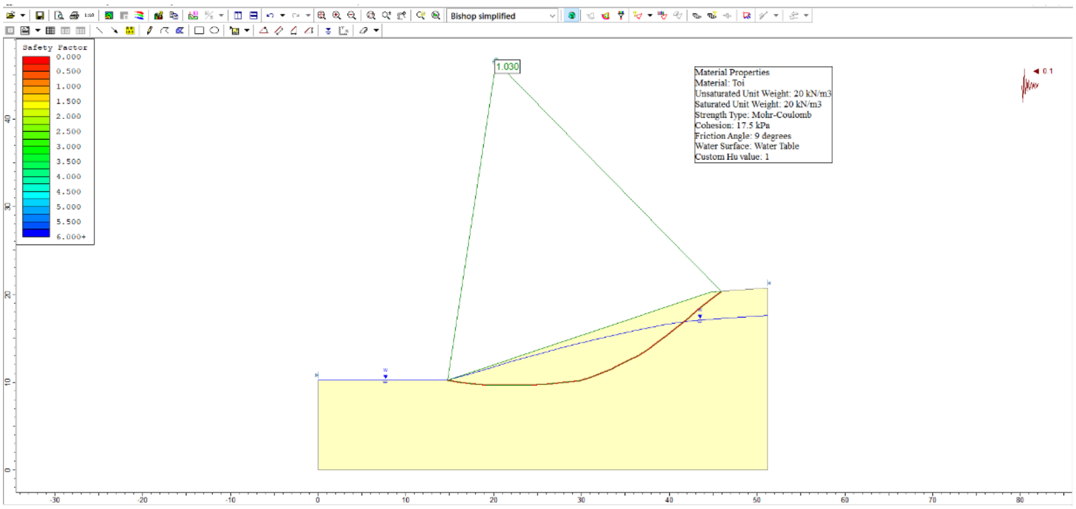


Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

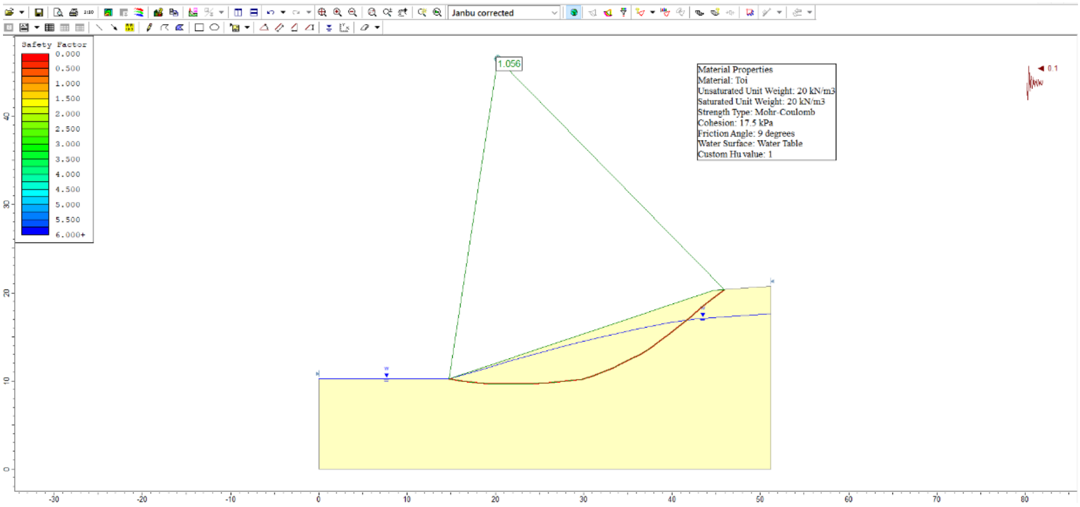
Figure 5.93. Stability analysis of KM: 128+630 cut slope with toe buttressing solution via Spencer method

Fourthly, slope flattening solution was tried. The original slope was flattened by excavating some material throughout the slope so that it has an inclination of H/V=3/1. Stability analyses of the slope show that slope flattening yields factor of safeties less than 1.1 and this method did not turn out to be a solution for KM:128+630 cut slope (Figures 5.94-5.97).



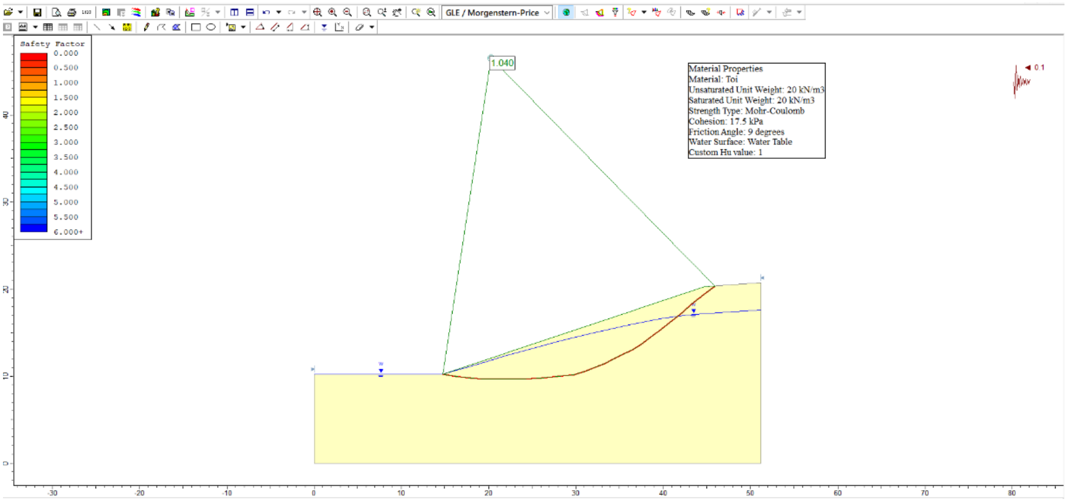
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.94. Stability analysis of KM: 128+630 cut slope with flattening solution via Bishop simplified method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

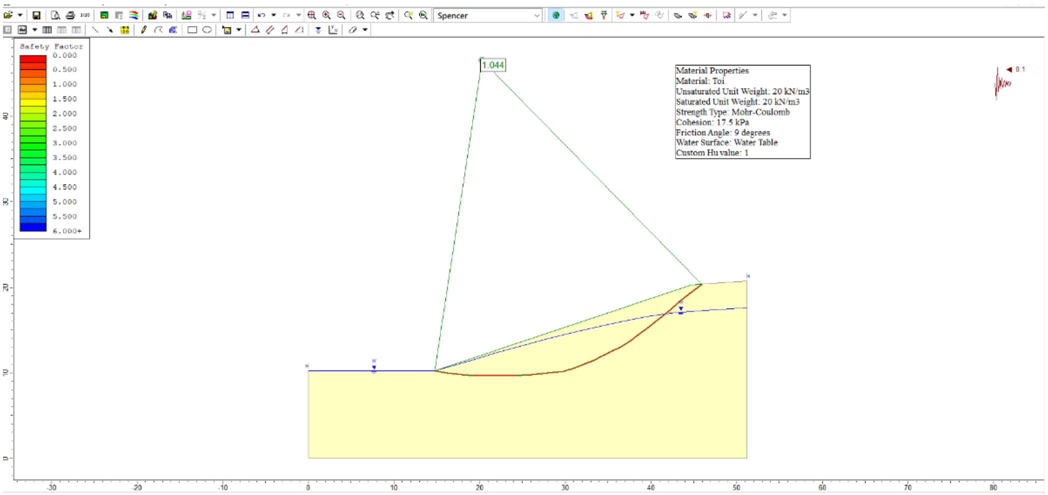
Figure 5.95. Stability analysis of KM: 128+630 cut slope with flattening solution via Janbu corrected method



GLE / Morgenstern-Price

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.96. Stability analysis of KM: 128+630 cut slope with flattening solution via GLE/Morgenstern-Price method

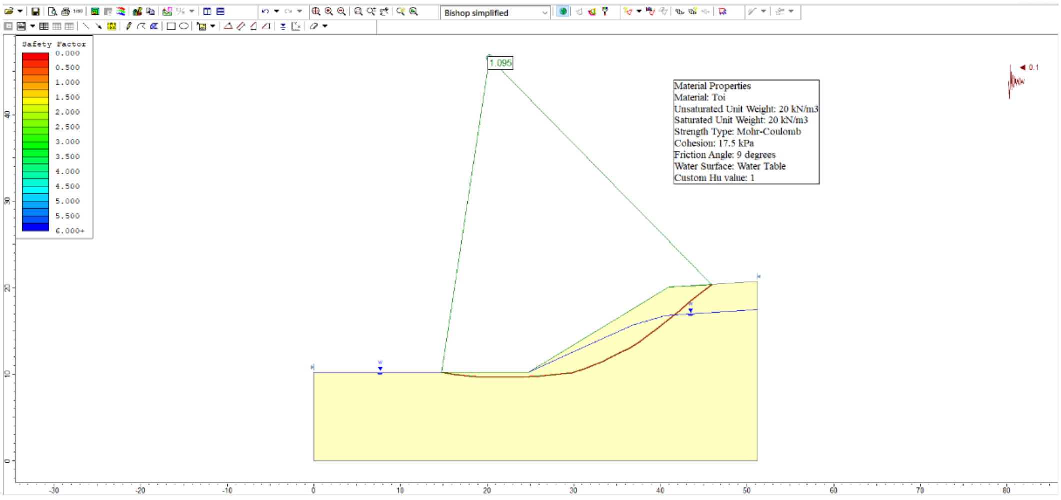


Spencer

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

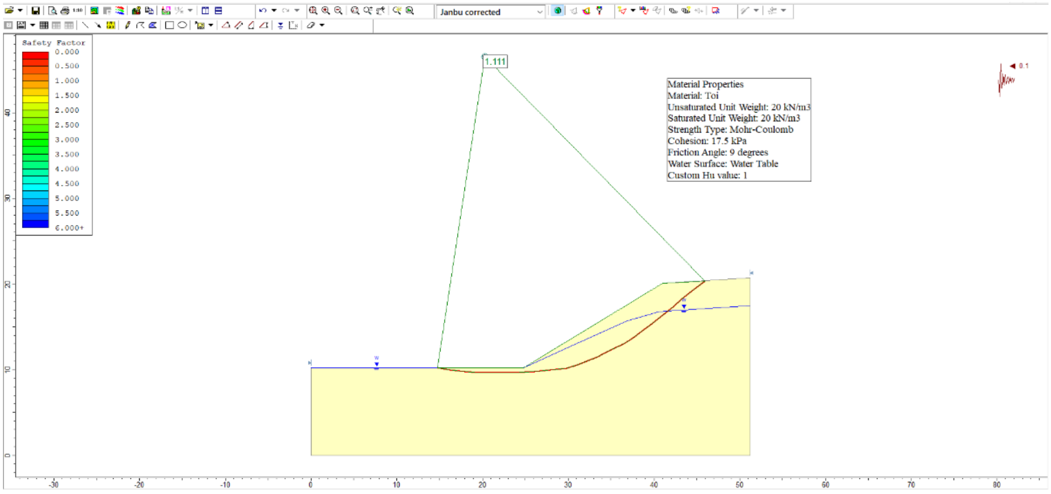
Figure 5.97. Stability analysis of KM: 128+630 cut slope with flattening solution via Spencer method

Fifthly, solution of removal of sliding material was tried. The original slope was excavated 10 m in width and reshaped in the way that the same inclination of H/V = 3/2 was satisfied after removal. Satisfactory safety factors for the reshaped slopes were obtained using different method of analyses (Figures 5.98-5.101). Therefore, removal of sliding material turned out to be an alternative solution for KM:128+630 cut slope.



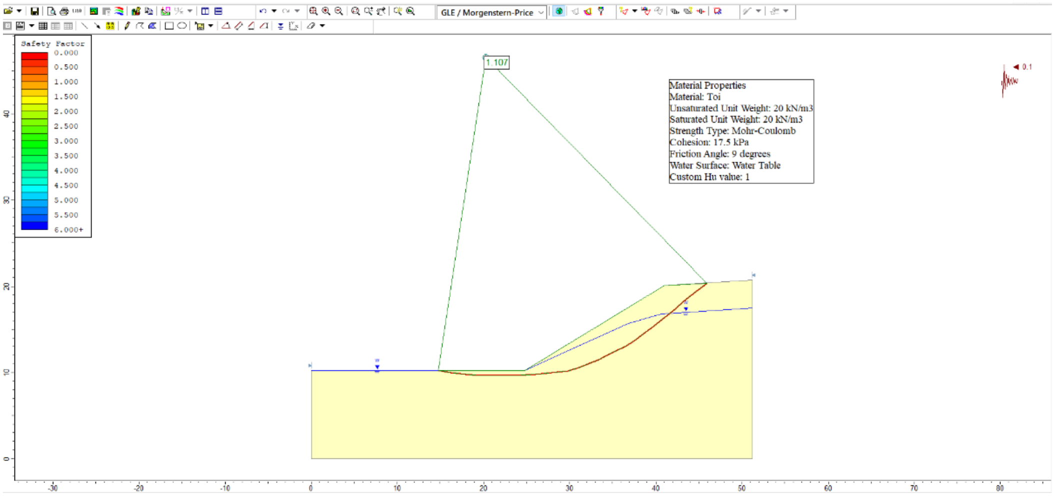
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.98. Stability analysis of KM: 128+630 cut slope with removal of sliding material solution via Bishop simplified method



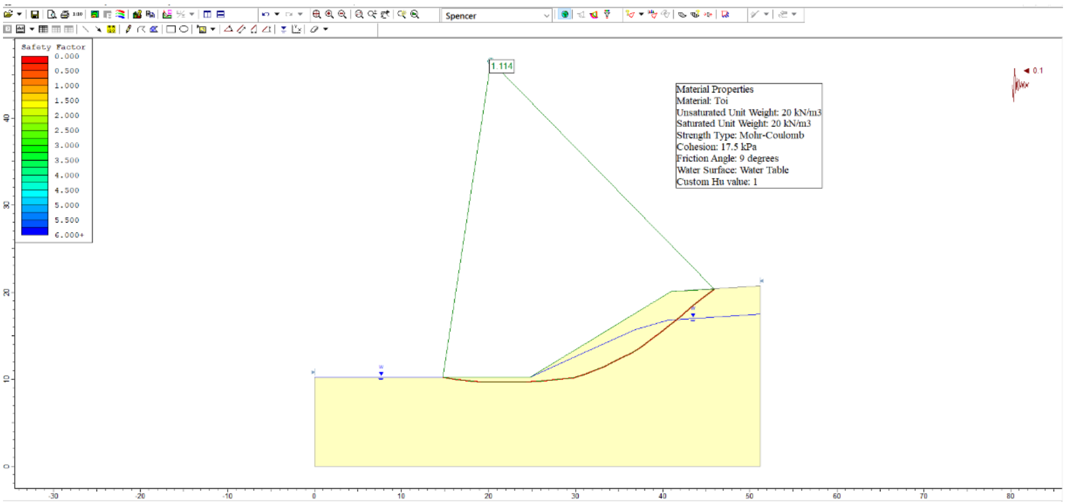
Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.99. Stability analysis of KM: 128+630 cut slope with removal of sliding material solution via Janbu corrected method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

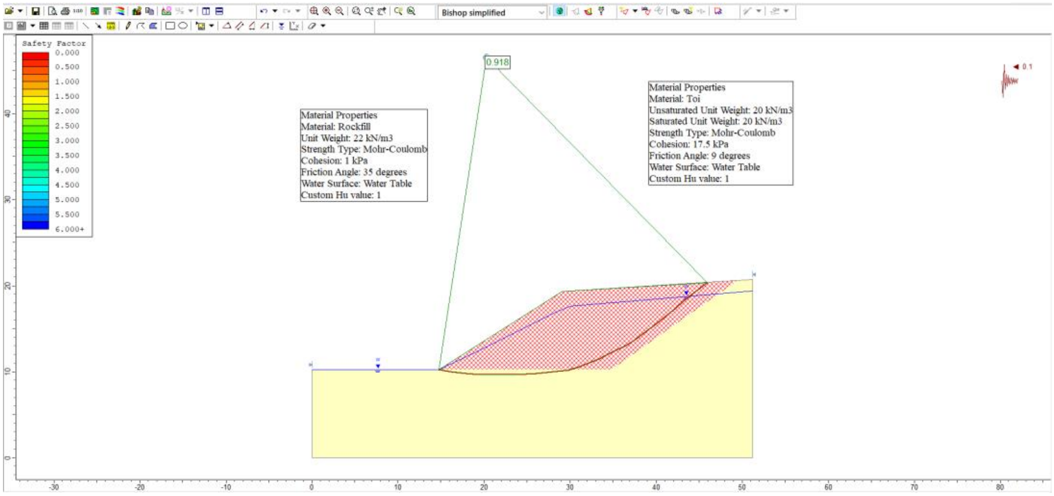
Figure 5.100. Stability analysis of KM: 128+630 cut slope with removal of sliding material solution via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.101. Stability analysis of KM: 128+630 cut slope with removal of sliding material solution via Spencer method

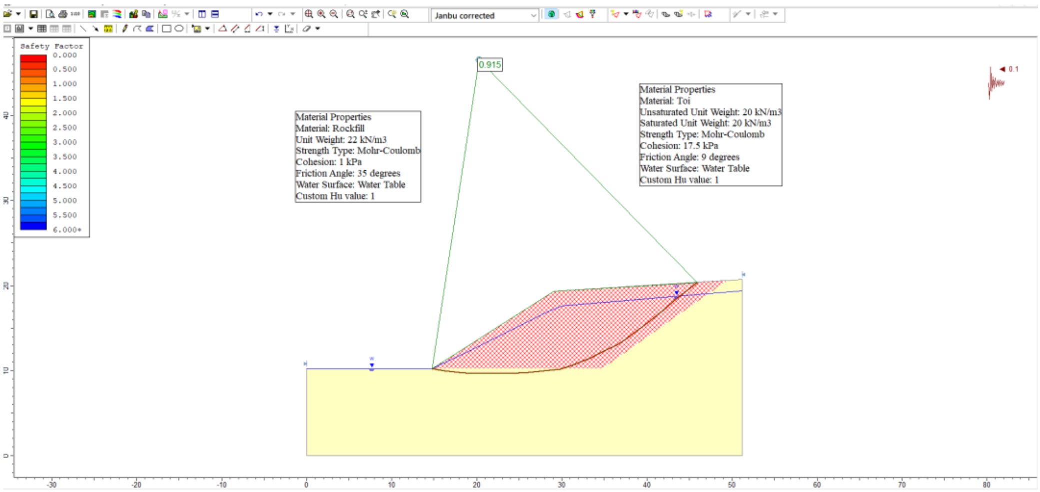
Lastly, solution of removal of sliding material and filling with rock was tried. The original slope was excavated 20 m in width and filled with rock in the way in the way that the same inclination of H/V = 3/2 was satisfied after removal and filling with rock. Relatively low safety factors of the flattened slope were obtained for all methods of analyses (Figures 5.102-5.105). Therefore, removal of sliding material and filling with rock was not accepted to be an alternative solution for KM:128+630 cut slope.


Bishop simplified

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

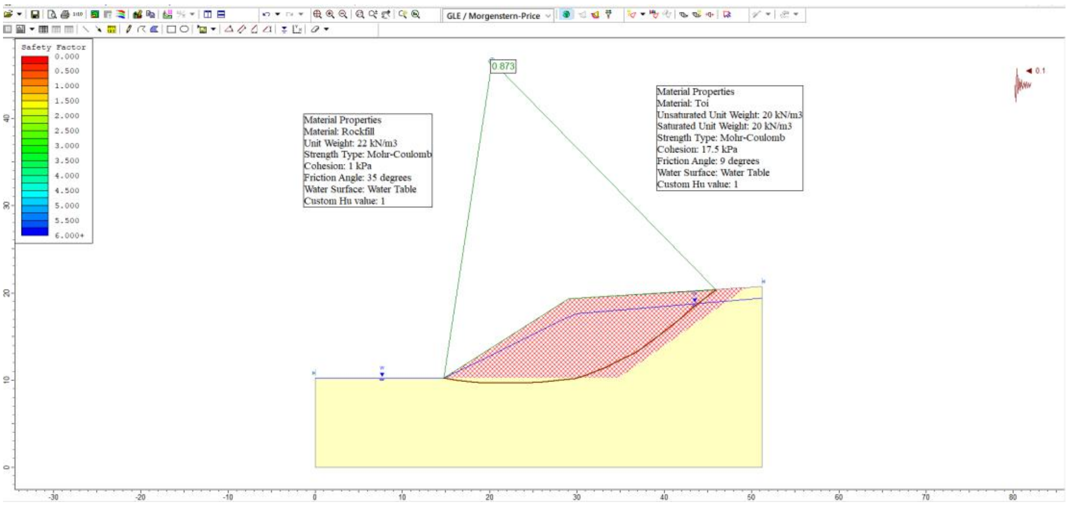
Figure 5.102. Stability analysis of KM: 128+630 cut slope with removal of sliding material+filling with rock solution via Bishop simplified method


Janbu corrected

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

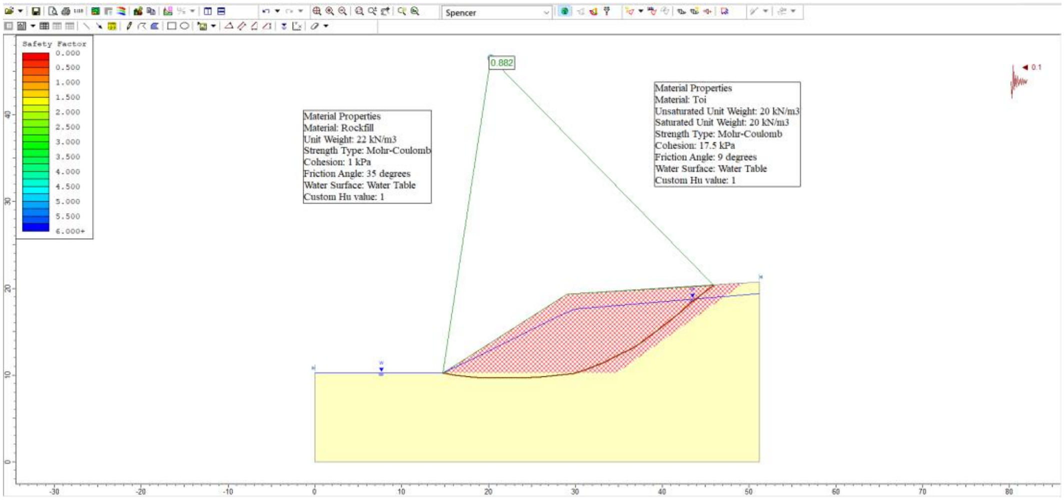
Figure 5.103. Stability analysis of KM: 128+630 cut slope with removal of sliding material+filling with rock solution via Janbu corrected method


GLE / Morgenstern-Price

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.104. Stability analysis of KM: 128+630 cut slope with removal of sliding material+filling with rock solution via GLE/Morgenstern-Price method


Spencer

Material Properties
Material: Rockfill
Unit Weight: 22 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 1 kPa
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1

Material Properties
Material: Toi
Unsaturated Unit Weight: 20 kN/m3
Saturated Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 17.5 kPa
Friction Angle: 9 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.105. Stability analysis of KM: 128+630 cut slope with removal of sliding material+filling with rock solution via Janbu corrected method

Within the scope of this study, all four types of limit equilibrium analysis methods were run in order to compare the FS values for each cut slope. From the results, it can clearly be observed that the results are divergent since relatively higher factor of safety values are calculated when practicing limit equilibrium methods satisfying both force and moment equations of equilibrium compared to the methods satisfying only force or moment equations of equilibrium (Table 5.2). Spencer's method provides a complete equilibrium of the sliding mass by taking moment and force into account. Conversely, the Janbu corrected method does not provide a moment equilibrium but ensures vertical and horizontal force equilibrium, whereas the Bishop simplified method does not provide horizontal force equilibrium but ensures vertical force and overall moment equilibrium (Carpenter, 1985). Limit equilibrium methods such as Morgenstern and Price and Spencer are more accurate than the aforementioned methods since they ensure moment and force equations of equilibrium at the same time (Solati and Habibagahi, 2006).

Table 5.2. Summary of factor of safety values for each cut slope before remedial measures

Section No	**Factor of Safety**							
	Bishop Simplified (1955)		Janbu Corrected (1968)		Morgenstern and Price (1965)		Spencer (1967)	
	Static	Dynamic	Static	Dynamic	Static	Dynamic	Static	Dynamic
KM:109+590	0.996	0.780	1.026	0.791	0.986	0.763	0.991	0.762
KM 113+120	1.138	0.776	1.155	0.783	1.151	0.781	1.166	0.785
KM 121+200	1.036	0.860	1.079	0.888	1.076	0.889	1.065	0.898
KM 128+630	1.109	0.835	1.142	0.848	1.121	0.844	1.126	0.849

5.2.5. Long Term Stability of Stable KM: 107+100 Cut Slope

As a crosscheck, the stability of the cut slopes with failure were checked against a neighbouring non-failed cut slope KM:107+100 (Figure 5.106). The KM:107+100 cut slope is also made up of the İncik formation still close to a transition zone with another formation, namely "İç Anadolu formation". Although the İncik formation shows characteristics between rock and soil, the formation demonstrates more resistant properties similar to rock at KM:107+100 cut slope. Since Hoek-Brown Failure Criterion assumes that the rock mass is characterized by an elastic-brittle-plastic behaviour while Mohr-Coulomb assumes that it is characterized by an elastic-perfectly plastic behaviour, for this cut slope Hoek-Brown Failure Criterion was adopted depending on visual inspection.

The rock mass strength parameters of the formation were specified via Rocscience RocLab software which is relying on the generalized Hoek-Brown Failure Criterion. Shear strength parameters of the İncik formation for this cut slope were requestioned on the basis of the geotechnical investigations. The RocLab calculates the Mohr-Coulomb parameters such as cohesion and internal friction angle together with the other parameters (i.e. mb, s and a) of a rock mass using Hoek-Brown Failure Criterion (Rocscience, 2002b) (Table 5.3).

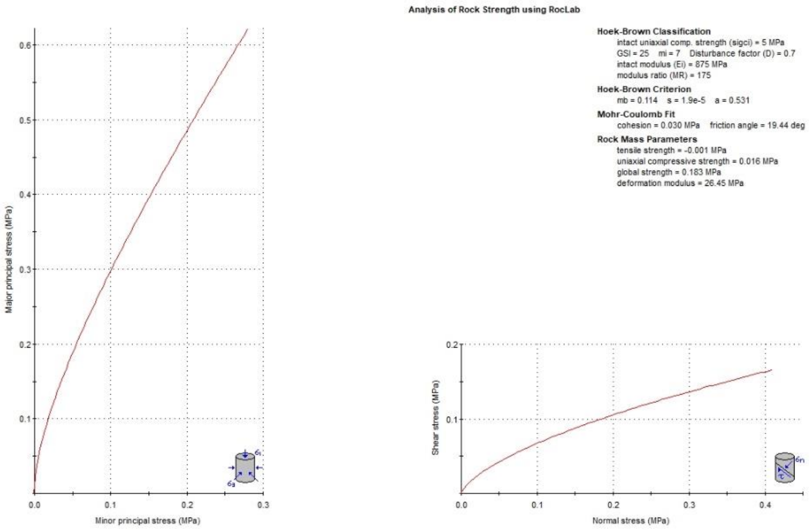
Figure 5.106. General view of KM:107+100 cut slope

Normal vs. shear stress graphs of the İncik formation are shown in Figure 5.107. Accordingly, shear strength parameters of the formation were computed to be: c = 30 kPa, ϕ = 19.44° by virtue of RocLab results.

Table 5.3. The parameters used for RocLab analysis of KM:107+100 cut slope

Hoek-Brown Classification						**Hoek-Brown Criterion**			**Failure Envelope Range**		
σ_{ci} (Mpa)	GSI	mi	D	Ei (Mpa)	MR	mb	s	a	σ_{3max}(Mpa)	γ (MN/m^3)	Slope height (m)
5	25	7	0.7	875	175	0.114	1.9e-5	0.531	0.2806	0.02	21

Having these in hand, the data were entered into SLIDE program and slope stability analyses are performed using this software (Figures 5.108-5.111). The analyses indicate that safety factors of the slope are generally higher than FS=1.1 and the slope is expected to be stable in the long-term.



Analysis of Rock Strength using RocLab

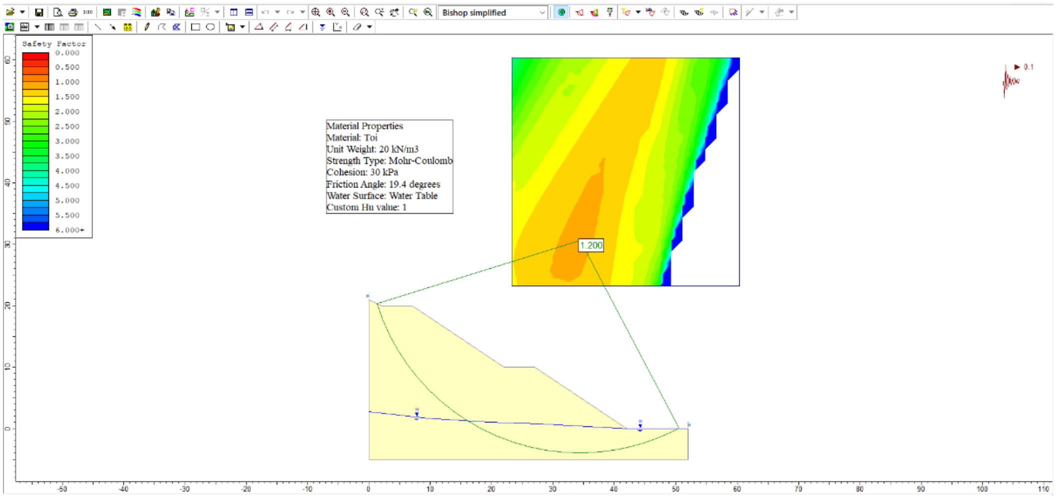
Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 5 MPa
GSI = 25   mi = 7   Disturbance factor (D) = 0.7
intact modulus (Ei) = 875 MPa
modulus ratio (MR) = 175

Hoek-Brown Criterion
mb = 0.114   s = 1.9e-5   a = 0.531

Mohr-Coulomb Fit
cohesion = 0.030 MPa   friction angle = 19.44 deg

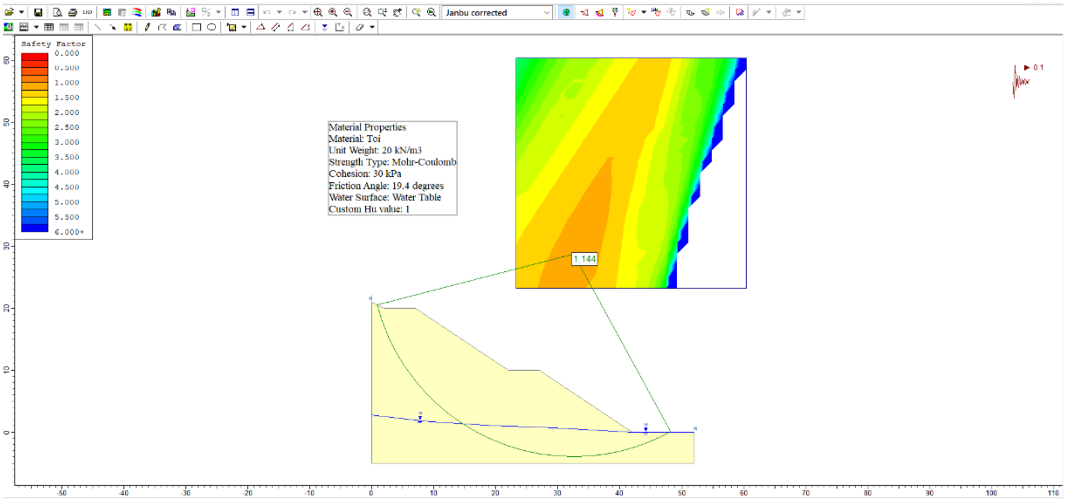
Rock Mass Parameters
tensile strength = -0.001 MPa
uniaxial compressive strength = 0.016 MPa
global strength = 0.183 MPa
deformation modulus = 26.45 MPa

Figure 5.107. Normal vs. shear stress graphs of KM: 107+100 cut slope



Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 30 kPa
Friction Angle: 19.4 degrees
Water Surface: Water Table
Custom Hu value: 1

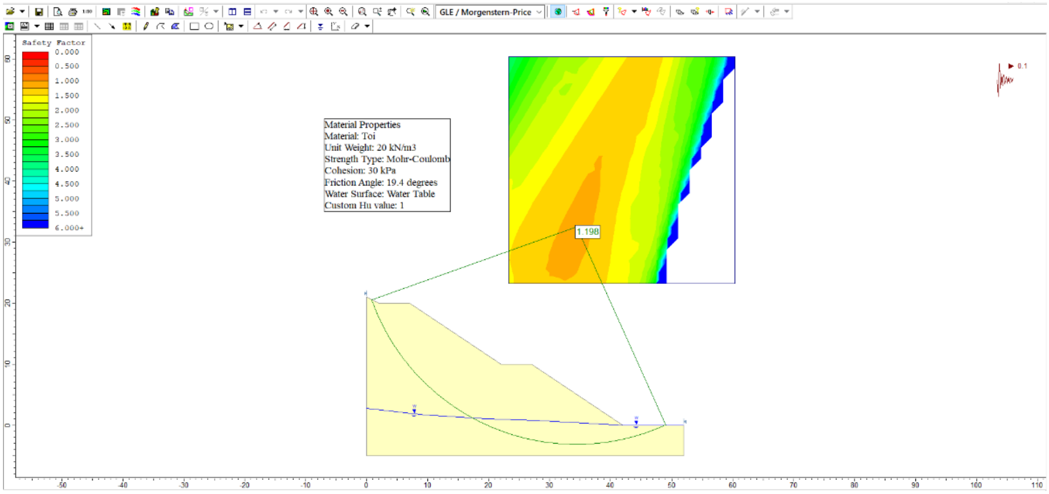
Figure 5.108. Stability analysis of KM: 107+100 cut slope based on Roclab analysis data via Bishop simplified method



Janbu corrected

Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 30 kPa
Friction Angle: 19.4 degrees
Water Surface: Water Table
Custom Hu value: 1

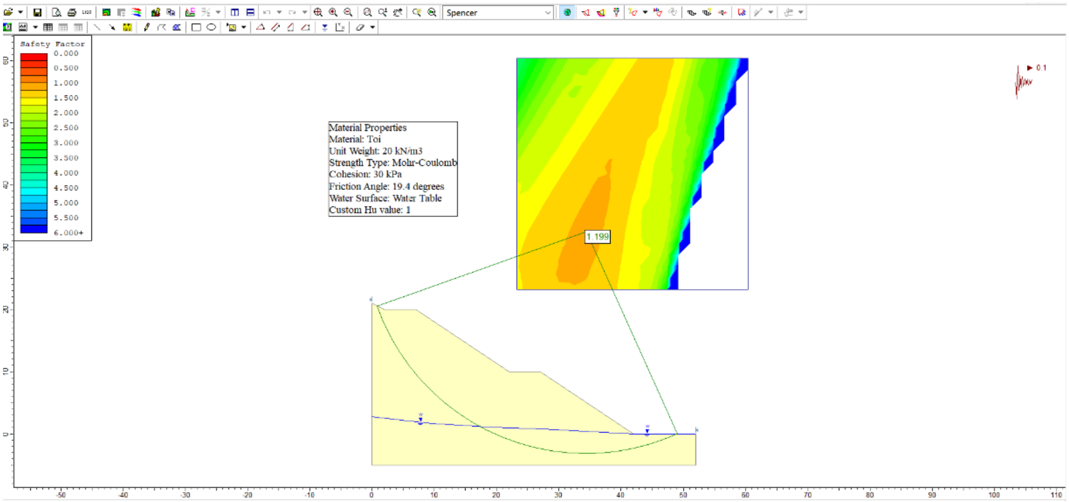
Figure 5.109. Stability analysis of KM: 107+100 cut slope based on Roclab analysis data via Janbu corrected method



GLE / Morgenstern-Price

Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 30 kPa
Friction Angle: 19.4 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.110. Stability analysis of KM: 107+100 cut slope based on Roclab analysis data via GLE/Morgenstern-Price method



Material Properties
Material: Toi
Unit Weight: 20 kN/m3
Strength Type: Mohr-Coulomb
Cohesion: 30 kPa
Friction Angle: 19.4 degrees
Water Surface: Water Table
Custom Hu value: 1

Figure 5.111. Stability analysis of KM: 107+100 cut slope based on Roclab analysis data via Spencer method

CHAPTER 6

DISCUSSIONS

Stability of four failed cut slopes with failure were examined within the scope of this thesis. Based on the site investigations, in-situ and laboratory tests, failure configurations were assembled. Shear strength parameters were estimated via SLIDE 6.0 software by performing back analysis method on representative cross sections of four cut slopes. Having relatively low shear strength parameters where c'=17,5 kPa and φ'=9°, all four cut slopes failed following a rainy season after excavation.

The situation for İncik formation is not the only case in literature, of course. Having several characteristics in common with the İncik formation, Ankara Clay is dominantly made up of clay, in which mineralogical and engineering properties change locally (Aras et al, 1991). Thereby, a number of slopes excavated in Ankara Clay are robust at steep angles for a long time while some others with very mild slopes fail easily. As same for the İncik formation, Ankara Clay involves carbonate concretions within clay sequences at particular locations (Birand, 1978) where the sloping surface is fully saturated. Adverse effects of water inclusion is indicated by low FS values as a result of high groundwater level (Teoman, et al., 2004). On the other hand, the landsliding activities around Koyulhisar settlement area show similar geotechnical characteristics with the İncik formation where c=1kPa and φ=16° at the time of failure based on back analysis results obtained for flyschoidal sequence overlain by colluvium. Both flyschoidal sequence and colluvium are composed of mostly clay, silt and gravel sized particles with clay and sand intercations (Hatiboğlu, 2009). According to Hatiboğlu (2009), high level of groundwater is one of the most significant parameters causing instability. As Karl Terzaghi said in 1939, "…In engineering practice, difficulties with soils are almost exclusively due not to soils themselves but to water contained in their voids." So, saturation of the geological formation with water and thus an increase in weight and decrease in cohesion of the units can be stated as one of the common major causes for such cases.

All four cut slopes are more or less composed of the same geological formation and have similar geotechnical properties. They are all made up of the İncik formation with some ignorable variations, such as fine evaporite vessels, color changes, local conglomerate bands, etc. (Figures 6.1-6.3). Knowing that conglomerate has higher potential to hold water than claystone, evaporites are prone to disintegration in relation to water and all are presenting different behaviours with increasing water content, these variations can have contributory effects for failures.

Figure 6.1. Fine evaporite vessels at KM: 121+120 cut slope

As remedial measurements, piling, bench and/or toe buttressing, slope flattening, removal of sliding material and filling with rock methods were implemented by using Bishop Simplified, Janbu Corrected, Morgenstern and Price and Spencer methods via non-circular failure providing minimum safety factor of 1.1 for long term stability of the cut slopes. Among these methods, results based on Spencer method were taken as basis since it takes both moment and force equilibrium into consideration throughout the analyses and its accuracy is dominantly agreed in literature.

Figure 6.2. Color changes at KM: 109+590 cut slope

Figure 6.3. Local conglomerate bands at KM: 128+630 cut slope

A constant rate of the normal to shear interslice force is excepted for all slices in the Spencer method. Otherwise, the resultant interslice forces were excepted to be in the same direction with all the slices. For the Morgenstern and Price method, the direction of the interslice forces were assumed to be described by an arbitrary function. So, the Spencer method is supposed to be a particular case of Morgenstern and Price in which a constant function is implemented on behalf of interslice forces. The Spencer method operates a collective process of stability analysis, where force and moment equilibrium are satisfied at the same time. For this reason, higher factor of safeties are obtained via Spencer method compared to the methods like Bishop Simplified, satisfying only the moment equilibrium equation (Solati and Habibagahi, 2006). On the other hand, calculation of factor of safety values based on moment ratios is not favorable in non-circular analyses in which an arbitrary sliding surface is present. Hence, Spencer's method was chosen to be the key indicator of the factor of safety results within this study since it satisfies a complete equilibrium for the failure by means of both force and moment equilibrium.

Another concept that deserves to be mentioned of is progressing with the opt for circular or non-circular failure throughout the analyses. Mostly, the shape of the slip surface is accepted to be circular in the solution of slope stability problems. The reason for chosing a circular slip surface is dominantly for making the calculations easier (Morgenstern and Price, 1965). According to Bishop (1957), when an analysis is based only on circular slip surfaces, the safety factor can be substantially overestimated. Meanwhile, according to Morgenstern and Price (1965), the opting for non-circular sliding in the stability analysis of rock slopes is necessary. Concordantly, non-circular failures were implemented for the cut slopes with failure on the basis of visual inspection through field surveys. Bishop Simplified is not an appropriate method for non-circular failure, yet it was just run for comparison of the results in this thesis. For KM: 107+100 cut slope, circular failure was applied for simplicity since no failure occured on this cut slope.

For the geological formations like İncik formation that reveal properties in between rock and soil, determining the extent of sliding mass disturbance is essential for deciding on whether to introduce circular and non-circular movement for analysis. Relatively small surface disturbances are detected in cases where soil mass is mobilized in the form of rigid body rotation. In the case of non-circular failures, relatively rough surface disturbances are observed (Morgenstern and Price, 1965). This type of movements induce intense shear stresses in the rock mass. Specifially, KM: 113+120 cut slope is a good example of severe disturbance. Being the largest of the four cut slopes with a maximum height of 33 meters, length of 1456 meters and longitudinal span of 70m and having the maxiumum mobilized mass due to failure, the KM: 113+120 cut slope is the most intriguing one among four cut slopes. By taking these facts into consideration, the results based on Spencer method via non-circular slip surfaces were taken as a basis for the analyses while the results based on the other methods were used for the aim of comparison to observe the variations in the results of the analyses. Factor of safety values obtained for each cut slope are given in Table 5.2.

KM:109+590 cut slope is a relatively low one with a height of 16 m. As it is stated in analyses, piling, slope flattening, removal of sliding material and filling with rock techniques were successful for KM:109+590 cut slope. A maximum pile strength of 550 kN and the original slope flattened by a vertical (V)-horizontal (H) ratio of 1:3 satisfied FS=1.1 condition by all analysis methods. It is appeerent that the analyses via Janbu Corrected method give slightly higher factor of safety values compared to others for piling solution. This could be due to the slope experiencing both planar and rotational failure but dominantly rotational type. Still, since both types of failure take charge in failure, methods using both moment and force equilibrium (i.e. Morgenstern and Price and Spencer) are expected to give more reliable results compared to the methods which use only force or moment equilibrium in calculation.

Among these four cut slopes, KM:113+120 was the most critical one considering the depth of failure and the amount of mobilized material. This can be attributed to its height (33 m). For this reason, an in-depth and generic remediation was necessary for this cut slope. As it is stated in analyses, only the remediation techniques of piling and removal of sliding material solutions were successful for KM:113+120 cut slope where a maxiumum pile strength of 1550 kN scarcely satisfied FS=1.1 condition via Spencer method. The other methods especially Bishop Simplified fell behind the target factor of safety. Since more reliable results were obtained by implementing methods using both moment and force equilibrium, this could be due to the slope experiencing different failure types, such as both planar and rotational. For this reason, methods using both moment and force equilibrium (i.e. Morgenstern and Price and Spencer) give more reliable results compared to the methods which use only force or moment equilibrium in calculation.

KM:121+200 cut slope is a moderate one with a height of 21 m. As it is stated in analyses, piling, and removal of sliding material techniques were successful for KM:121+200 cut slope, while the original slope flattened by a vertical (V)-horizontal (H) ratio of 1:3 did not satisfy the FS=1.1 condition by any analysis methods. A maximum pile strength of 150 kN satisfied FS=1.1 condition by all analysis methods. Still, Bishop Simplified and Janbu Corrected methods fell slightly behind the target factor of safety. Since more reliable results were obtained by implementing methods using both moment and force equilibrium, this could be due to the slope experiencing different failure types, such as both planar and rotational. For this reason, methods using both moment and force equilibrium (i.e. Morgenstern and Price and Spencer) give more reliable results compared to the methods which use only moment or force equilibrium in calculation.

KM:128+630 cut slope is the lowest one with a height of 15 m. As it is stated in analyses, piling, slope flattening and removal of sliding material techniques were successful for KM:128+630 cut slope. A maximum pile strength of 350 kN and the original slope flattened by a vertical (V)-horizontal (H) ratio of 1:3. satisfied FS=1.1 condition by all analysis methods. Still, Bishop Simplified and Janbu Corrected methods fell slightly behind the target factor of safety. Since more reliable results were obtained by implementing methods using both moment and force equilibrium, this could be due to the slope experiencing different failure types, such as both planar and rotational. For this reason, methods using both force and equilibrium (i.e. Morgenstern and Price and Spencer) give more reliable results compared to the methods which use only moment or force equilibrium in calculation.

Stability of the neighbouring stable cut slope (KM:107+100) was also crosschecked within the scope of the thesis. The stable KM:107+100 cut slope consists of the same geological material as the four cut slopes with failure, but with different shear strength parameters (c’ = 30 kPa, φ’= 19.44°). This can be explained by the existence of a transition zone between the İncik formation and a more competent İç Anadolu Group. In this cut slope, it can be observed that the stability analysis via Janbu Corrected method conspicously gave a lower factor of safety value compared to the other methods. This can be due to result from the type of failure dominating the failure. Still, Janbu Corrected method is based on force equilibrium, not momental equilibrium, attaining a low factor of safety value is not surprising.

Excavation of the four cut slopes dates back to January 2015. While no failure was met at time of excavation, traces of groundwater outlets and intensive erosion came up around July 2015. On the other hand, during the rainy spring season, groundwater level rised and surface water and by extension rill erosion emerged. Since the mineralogical and geotechnical properties of the İncik formation vary significantly from location to location, all cut slopes on the railway route are under the risk of failure. Local conglomerate and gypsum inclusions are the typical examples for this situation. As in the case of KM:107+100 cut slope, currently stable cut slopes may lose strength due to external effects by time, either. Moreover, the failures are not restricted with only high cut slopes, failure is also present at relatively low cut slopes like KM:128+630 with a height of 15m. Year after year, destructive effects of precipitation and erosion will become appearent. Considering the railway route as a whole, some other failures are expected to occur unless remedial measures are taken.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

Within the scope of this study, engineering geological properties of the İncik formation at the time of failure, the reasons underlying the failure and remedial techniques at four cut slopes; namely, KM:109+590, KM:113+120, KM:121+200 and KM:128+630 of Kırıkkale-Yerköy Section (Section-2) of Ankara-Sivas High-Speed Railway Project were investigated.

The conclusions and recommendations deduced following this study are as follows:

1) The İncik formation exposed in the study area is reddish-brown, gray colored, parallel and cross tabulated, poorly-sorted, partially uncemented continental conglomerate, sandstone, mudstone alternation with evaporates.
2) For all four cut slopes, gradient of H/V= 3/2 resulted in failure regarding to long term seismic stability.
3) Shear strength parameters of the İncik formation along four profiles were investigated for non-circular failure by Rocscience SLIDE software at the time of failure via back analysis method and the results came up to be: c=17,5 kPa and φ=9°.
4) As remedial measurements, piling, benching and/or toe buttressing, slope flattening, removal of sliding material and filling with rock methods were implemented by using Bishop Simplified, Janbu Corrected, Morgenstern and Price and Spencer methods via non-circular failure providing minimum safety factor of 1.1 for long term stability of the cut slopes. Spencer's method of slope

stability analysis is inferred to give the most realistic results for failure characterization and non-circular failure.

5) The mutual remediation techniques are piling and removal of sliding material for four cut slopes.
6) The stability analyses for a stable cut slope KM: 107+100 at a transition zone between the İncik formation and the competent İç Anadolu Group where shear strength parameters were estimated to be: c=30 kPa and φ’=19.44° yielded a stable slope due its high shear strength parameters.
7) Toe butressing did not come up to be a solution for any of the cut slopes.
8) 1/3 slope flattening method came up to be a solution for two of cut slopes i.e. KM: 109+590 and KM:121+120; yet, application of this type of a technique will result in widening of the exposed slope surface and enhance rill erosion due to seepage of surface water. Nevertheless, in all types of excavation, the exposure to precipitation will rise and an increase in rill erosion which may adversely affect the slope stability is expected to come up at last. Head and heel ditches could be solutions partially for surface waters.
9) Removal of sliding material and filling with rock alternative are the feasible remediation techniques taking economical and environmental criteria into account and removal of sliding material alternative the most appropriate remediation technique for all four cut slopes.
10) For all cut slopes excavated in the İncik formation, freeze and thaw weathering must be considered as a secondary problem to be solved based on atmospheric conditions since the climate is dry and the temperature changes are high around study area.

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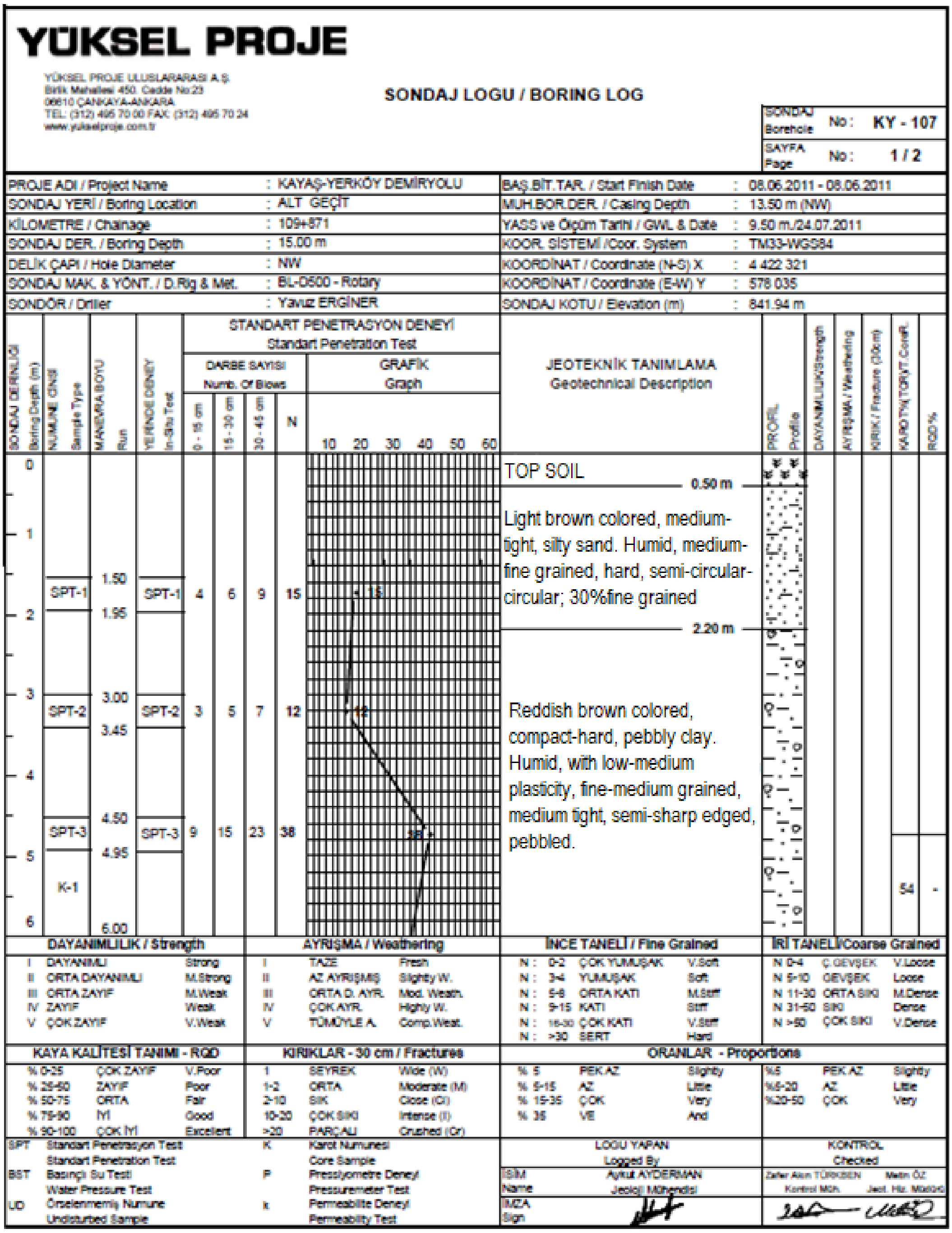
APPENDICES

A. BORING LOGS



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole	No :	KY - 107
SAYFA Page	No :	1 / 2

PROJE ADI / Project Name	: KAYAŞ-YERKÖY DEMİRYOLU	BAŞ.BİT.TAR. / Start Finish Date	: 08.06.2011 - 08.06.2011
SONDAJ YERİ / Boring Location	: ALT GEÇİT	MUH.BOR.DER. / Casing Depth	: 13.50 m (NW)
KİLOMETRE / Chainage	: 109+871	YASS ve Ölçüm Tarihi / GWL & Date	: 9.50 m./24.07.2011
SONDAJ DER. / Boring Depth	: 15.00 m	KOOR. SİSTEMİ /Coor. System	: TM33-WGS84
DELİK ÇAPI / Hole Diameter	: NW	KOORDİNAT / Coordinate (N-S) X	: 4 422 321
SONDAJ MAK. & YÖNT. / D.Rig & Met.	: BL-D500 - Rotary	KOORDİNAT / Coordinate (E-W) Y	: 578 035
SONDÖR / Driller	: Yavuz ERGİNER	SONDAJ KOTU / Elevation (m)	: 841.94 m

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test DARBE SAYISI Numb. Of Blows 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	JEOTEKNİK TANIMLAMA Geotechnical Description	KAROT%/TCR%/T.Core R.	RQD %
0								TOP SOIL		
0.50 m								Light brown colored, medium-tight, silty sand. Humid, medium-fine grained, hard, semi-circular-circular; 30%fine grained		
1										
	SPT-1	1.50 1.95	SPT-1	4	6	9	15			
2										
2.20 m								Reddish brown colored, compact-hard, pebbly clay. Humid, with low-medium plasticity, fine-medium grained, medium tight, semi-sharp edged, pebbled.		
3	SPT-2	3.00 3.45	SPT-2	3	5	7	12			
4										
	SPT-3	4.50 4.95	SPT-3	9	15	23	38			
5	K-1								54	-
6		6.00								

DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE TANELİ / Fine Grained			İRİ TANELİ/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N 0-4	Ç.GEVŞEK	V.Loose
II	ORTA DAYANIMLI	M.Strong	II	AZ AYRIŞMIŞ	Slighty 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.	Highly 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	Slighty	%5	PEK AZ	Slighty
% 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	%20-50	ÇOK	Very
% 75-90	İYİ	Good	10-20	ÇOK SIKI	Intense (I)	% 35	VE	And			
% 90-100	ÇOK İYİ	Excelent	>20	PARÇALI	Crushed (Cr)						

SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample
BST	Basınçlı Su Testi Water Pressure Test	P	Presslyometre Deneyi Pressuremeter Test
UD	Örselenmemiş Numune Undisturbed Sample	k	Permeabilite Deneyi Permeability Test

	LOGU YAPAN Logged By	KONTROL Checked
İSİM Name	Aykut AYDERMAN Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh. Metin ÖZ Jeot. Hiz. Müdürü
İMZA Sign		

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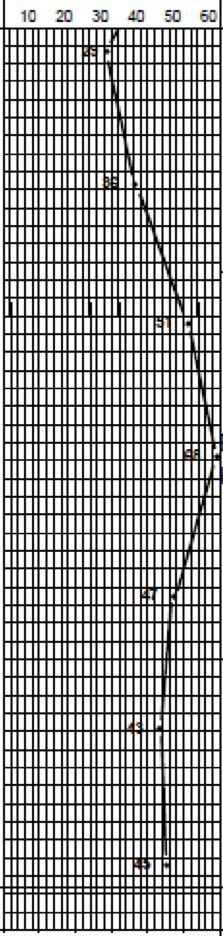
SONDAJ LOGU / BORING LOG

SONDAJ Borehole No : **KY - 107**
SAYFA Page No : **2 / 2**

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test: DARBE SAYISI Numb. Of Blows 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	KAROT%(TCR)/T.CoreR.	RQD %
6	SPT-4	6.00 6.45	SPT-4	9	11	18	29		
7	K-2							100	-
	SPT-5	7.50 7.95	SPT-5	11	15	21	36		
8	K-3							100	-
9	SPT-6	9.00 9.45	SPT-6	11	25	26	51		
10	K-4							51	-
	SPT-7	10.50 10.95	SPT-7	16	32	36	68		
11	K-5							94	-
12	SPT-8	12.00 12.45	SPT-8	13	18	29	47		
13	K-6							100	0
	SPT-9	13.50 13.95	SPT-9	10	17	26	43		
14	K-7							100	0
15	SPT-10	15.00 15.45	SPT-10	11	18	27	45		
16									

GRAFİK Graph



JEOTEKNİK TANIMLAMA Geotechnical Description

Reddish brown, very compact-hard, competent pebbly clay. Humid, with low-medium plasticity, fine-medium grained, hard, semi-sharp edged pebbled.

8.70 m

Yellowish green colored,hard sandy clay. Humid, with medium-high plasticity;%25-35 fine grained, hard, circular-semi-circular, sandy.

10.60 m

Brown colored, hard, pebbly clay. Humid,low-medium plasticity

11.00 m

Reddish brown colored, hard, pebbly clay/clayey sand. Humid, with low-medium plasticity;fine-medium grained, semi-sharp edged, pebbled.

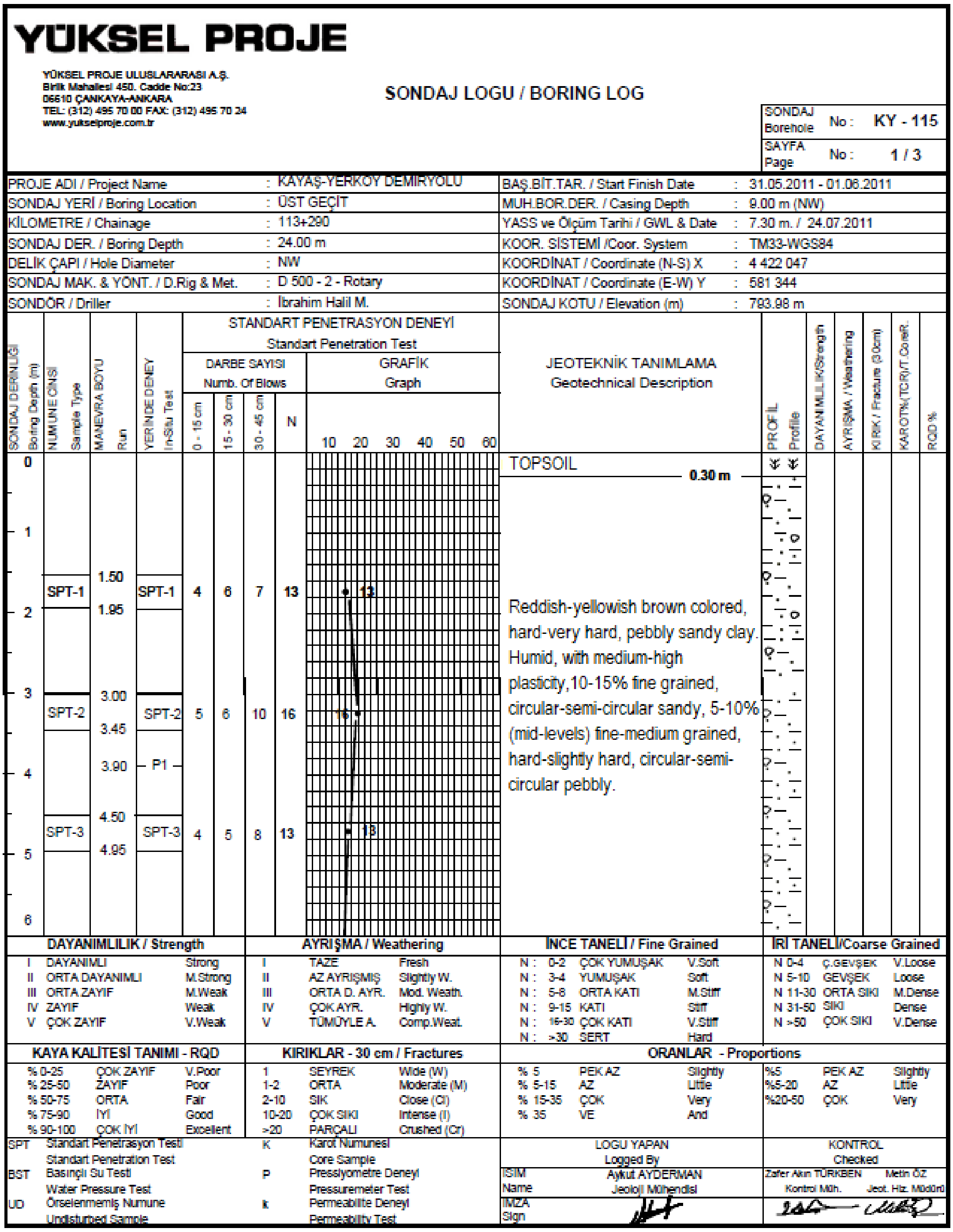
13.00 m

PEBBLED MUDSTONE Reddish brown colored, friable fragile, very incompetent, highly-totally weathered. Unit contains medium-coarse grained, hard pebbles.

PROFİL Profile | DAYANIMLILIK/Strength: V | AYRIŞMA / Weathering: IV V | KIRIK / Fracture (30cm)

Length: 15.45 m
1/2 (4.95 - 11.20 m); 2/2 (11.20 - 15.45 m)

	LOGU YAPAN Logged By	KONTROL Checked	
İSİM Name	Aykut AYDERMAN Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh.	Metin ÖZ Jeot. Hiz. Müdürü
İMZA Sign			



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole	No :	KY - 115
SAYFA Page	No :	1 / 3

PROJE ADI / Project Name	: KAYAŞ-YERKÖY DEMİRYOLU	BAŞ.BİT.TAR. / Start Finish Date	: 31.05.2011 - 01.06.2011
SONDAJ YERİ / Boring Location	: ÜST GEÇİT	MUH.BOR.DER. / Casing Depth	: 9.00 m (NW)
KİLOMETRE / Chainage	: 113+290	YASS ve Ölçüm Tarihi / GWL & Date	: 7.30 m. / 24.07.2011
SONDAJ DER. / Boring Depth	: 24.00 m	KOOR. SİSTEMİ /Coor. System	: TM33-WGS84
DELİK ÇAPI / Hole Diameter	: NW	KOORDİNAT / Coordinate (N-S) X	: 4 422 047
SONDAJ MAK. & YÖNT. / D.Rig & Met.	: D 500 - 2 - Rotary	KOORDİNAT / Coordinate (E-W) Y	: 581 344
SONDÖR / Driller	: İbrahim Halil M.	SONDAJ KOTU / Elevation (m)	: 793.98 m

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test: DARBE SAYISI Numb. Of Blows 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA / Weathering	KIRIK / Fracture (30cm)	KAROT%(TCR)/T.CoreR.	RQD %
0									TOPSOIL 0.30 m						
1															
	SPT-1	1.50 1.95	SPT-1	4	6	7	13	13							
2									Reddish-yellowish brown colored, hard-very hard, pebbly sandy clay. Humid, with medium-high plasticity,10-15% fine grained, circular-semi-circular sandy, 5-10% (mid-levels) fine-medium grained, hard-slightly hard, circular-semi-circular pebbly.						
3	SPT-2	3.00 3.45	SPT-2	5	6	10	16	16							
4		3.90	P1												
	SPT-3	4.50 4.95	SPT-3	4	5	8	13	13							
5															
6															

DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE TANELİ / Fine Grained			İRİ TANELİ/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N 0-4	Ç.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.	Highly 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	%20-50	ÇOK	Very
% 75-90	İYİ	Good	10-20	ÇOK SIKI	Intense (I)	% 35	VE	And			
% 90-100	ÇOK İYİ	Excellent	>20	PARÇALI	Crushed (Cr)						

					LOGU YAPAN Logged By	KONTROL Checked	
SPT	Standart Penetrasyon Testi Standart Penetration Test	K	Karot Numunesi Core Sample				
BST	Basınçlı Su Testi Water Pressure Test	P	Pressiyometre Deneyi Pressuremeter Test	İSİM Name	Aykut AYDERMAN Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh.	Metin ÖZ Jeot. Hiz. Müdürü
UD	Örselenmemiş Numune Undisturbed Sample	k	Permeabilite Deneyi Permeability Test	İMZA Sign			

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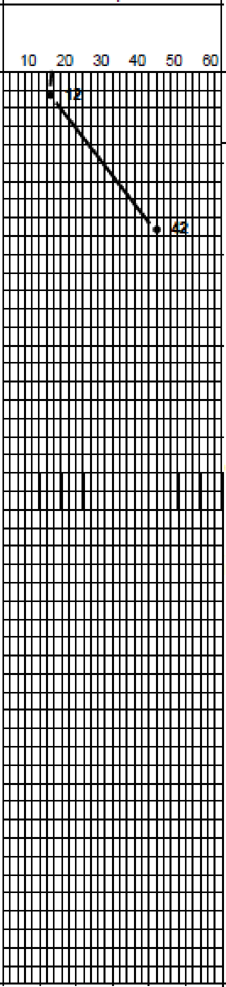
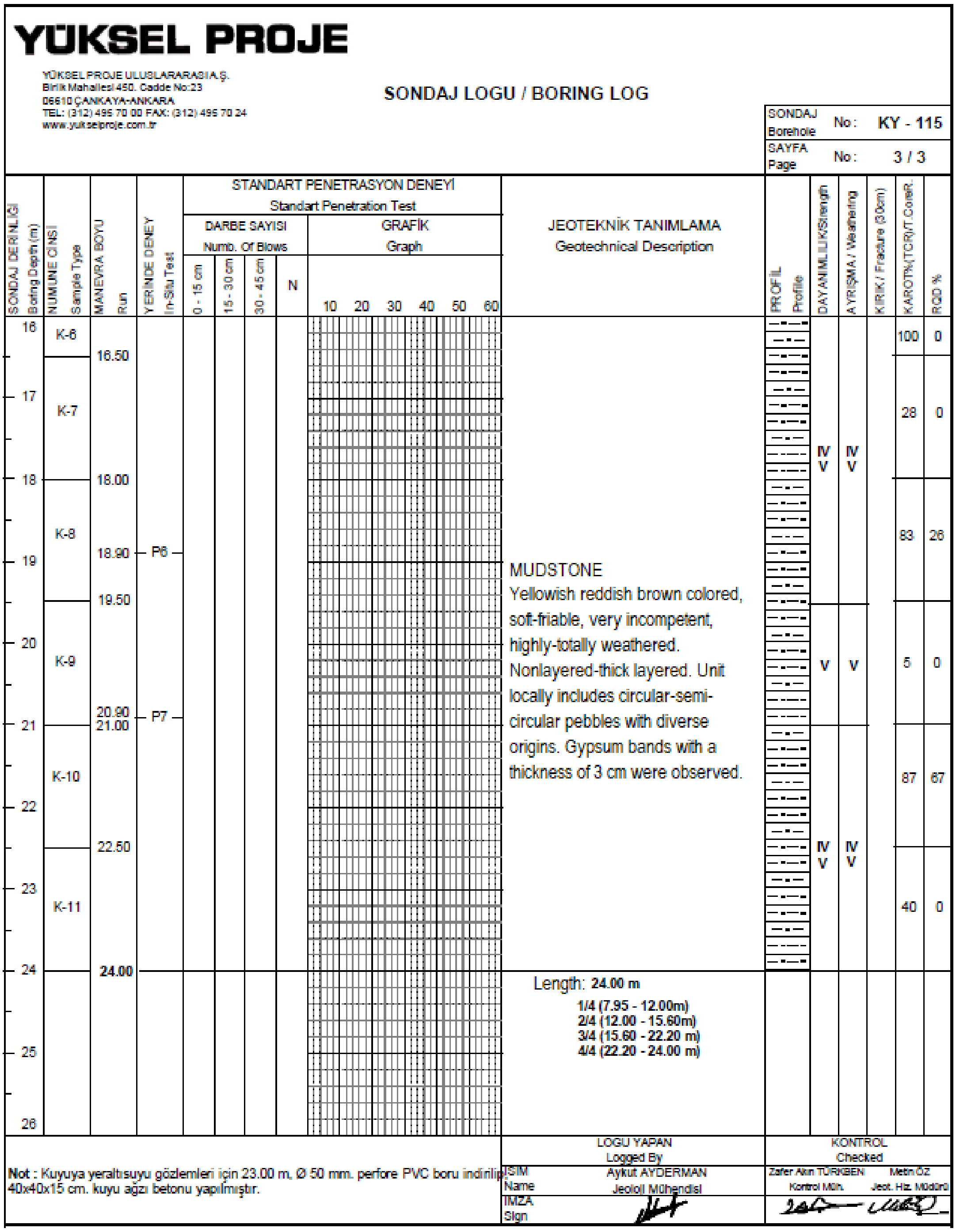
SONDAJ LOGU / BORING LOG

SONDAJ Borehole No : **KY - 115**
SAYFA Page No : **2 / 3**

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test: DARBE SAYISI Numb. Of Blows 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	GRAFİK Graph	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA / Weathering	KIRIK / Fracture (30cm)	KAROT%(TCR)/T.CoreR.	RQD %
6	SPT-4	6.00 6.45	SPT-4	3	5	7	12	12	Definition is given at page 1/3. **6.80 m**						
7		6.90	P2												
	SPT-5	7.50 7.95	SPT-5	13	17	25	42	42							
8	K-1													100	0
9	K-2	9.00 9.90	P3											100	20
10															
11	K-3	10.50							MUDSTONE Yellowish reddish brown colored, soft-friable, very incompetent, highly-totally weathered. Nonlayered-thick layered. Unit locally includes circular-semi-circular pebbles with diverse origins. Gypsum bands with a thickness of 3 cm were observed.		IV V	IV V		100	0
12	K-4	12.00 12.90	P4											100	30
13															
14	K-5	13.50												100	39
15	K-6	15.00 15.40	P5											100	0
16															



	LOGU YAPAN Logged By	KONTROL Checked	
İSİM Name	Aykut AYDERMAN Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh.	Metin ÖZ Jeot. Hiz. Müdürü
İMZA Sign			



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole No : **KY - 115**
SAYFA Page No : **3 / 3**

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test – DARBE SAYISI Numb. Of Blows: 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	GRAFİK Graph: 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA / Weathering	KIRIK / Fracture (30cm)	KAROT%(TCR)/T.CoreR.	RQD %
16	K-6								MUDSTONE Yellowish reddish brown colored, soft-friable, very incompetent, highly-totally weathered. Nonlayered-thick layered. Unit locally includes circular-semi-circular pebbles with diverse origins. Gypsum bands with a thickness of 3 cm were observed.					100	0
17	K-7	16.50												28	0
18	K-8	18.00	18.90 P6								IV V	IV V		83	26
19															
20	K-9	19.50									V	V		5	0
21	K-10	20.90 21.00	20.90 P7											87	67
22															
23	K-11	22.50									IV V	IV V		40	0
24		**24.00**							Length: **24.00 m** **1/4 (7.95 - 12.00m)** **2/4 (12.00 - 15.60m)** **3/4 (15.60 - 22.20 m)** **4/4 (22.20 - 24.00 m)**						
25															
26															

Not : Kuyuya yeraltısuyu gözlemleri için 23.00 m, Ø 50 mm. perfore PVC boru indirilip 40x40x15 cm. kuyu ağzı betonu yapılmıştır.

	LOGU YAPAN Logged By	KONTROL Checked	
İSIM Name	Aykut AYDERMAN Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh.	Metin ÖZ Jeot. Hiz. Müdürü
İMZA Sign			

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Birlik Mahallesi 450. Cadde No:23
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SONDAJ LOGU / BORING LOG

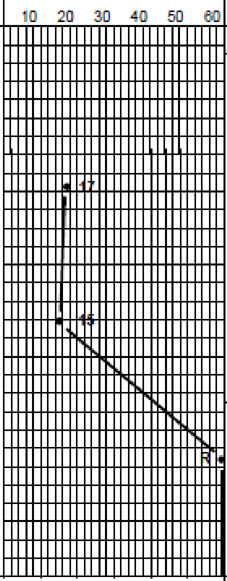
SONDAJ Borehole	No :	KY - 128
SAYFA Page	No :	1 / 2

PROJE ADI / Project Name	: KAYAŞ-YERKÖY DEMİRYOLU	BAŞ.BİT.TAR. / Start Finish Date	: 12.05.2011 - 13.05.2011
SONDAJ YERİ / Boring Location	: ALT GEÇİT	MUH.BOR.DER. / Casing Depth	: 6.00 m (NW)
KİLOMETRE / Chainage	: 120+659	YASS ve Ölçüm Tarihi / GWL & Date	: 3.50 m. / 24.07.2011
SONDAJ DER. / Boring Depth	: 12.00 m	KOOR. SİSTEMİ /Coor. System	: TM33-WGS84
DELİK ÇAPI / Hole Diameter	: NW	KOORDİNAT / Coordinate (N-S) X	: 4 415 771
SONDAJ MAK. & YÖNT. / D.Rig & Met.	: D500 - 2	KOORDİNAT / Coordinate (E-W) Y	: 584 534
SONDÖR / Driller	: İbrahim H. M.	SONDAJ KOTU / Elevation (m)	: 698.60 m

STANDART PENETRASYON DENEYİ / Standart Penetration Test

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	DARBE SAYISI Numb. Of Blows 0 - 15 cm	15 - 30 cm	30 - 45 cm	N
0							
1							
	SPT-1	1.50 1.95	SPT-1	5	7	10	17
2							
3	SPT-2	3.00 3.45	SPT-2	4	7	8	15
4							
	SPT-3	4.50 4.94	SPT-3	19	42	50/14	R
5	K-1	6.00					
6							

GRAFİK Graph



JEOTEKNİK TANIMLAMA
Geotechnical Description

TOPSOIL

0.40 m

Reddish brown-wine colored, stiff-very stiff silty clay. Humid-dry, with moderate-high plasticity. Unit includes gypsum bands.

4.10 m

SILTSTONE

Brown colored-reddish Brown colored, slightly stiff, moderately weak-weak, moderately weathered. Discontinuities; 0°, 20°, closed, 1-2 mm, filled with a trace of gypsum.

PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA / Weathering	KIRIK / Fracture (30cm)	KAROT%(TCR)/T. CoreR.	RQD %
	IV				
	III IV	III	2 - 10	73	47

DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE TANELİ / Fine Grained			İRİ TANELİ/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N 0-4	Ç.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.	Highly 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	%20-50	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
BST	Basınçlı Su Testi Water Pressure Test	P	Pressiyometre Deneyi Pressuremeter Test
UD	Örselenmemiş Numune Undisturbed Sample	k	Permeablite Deneyi Permeability Test

	LOGU YAPAN Logged By	KONTROL Checked	
İSİM Name	Aykut AYDERMAN Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh.	Metin ÖZ Jeot. Hiz. Müdürü
İMZA Sign			

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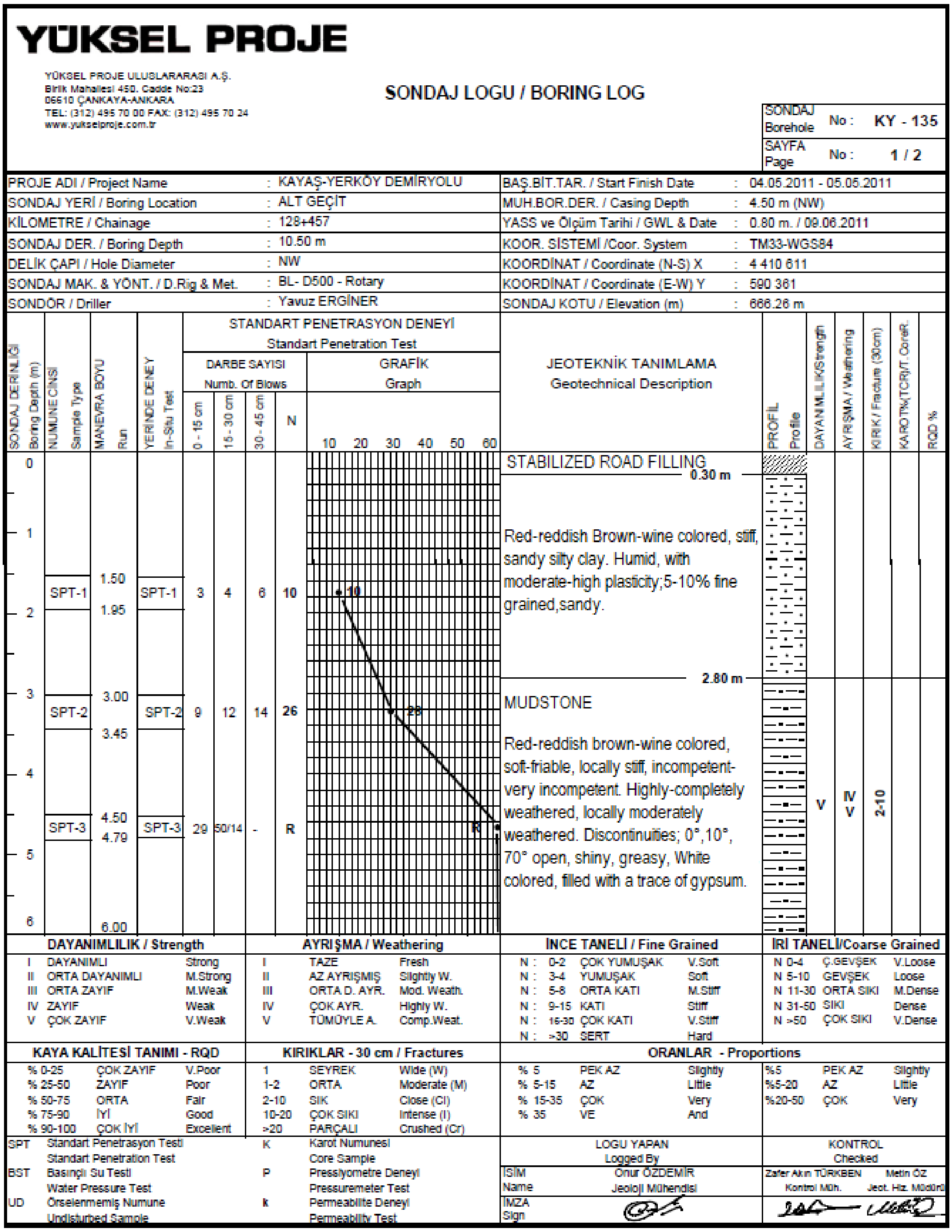
SONDAJ LOGU / BORING LOG

SONDAJ Borehole No : **KY - 128**
SAYFA Page No : **2 / 2**

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test: DARBE SAYISI Numb. Of Blows 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA / Weathering	KIRIK / Fracture (30cm)	KAROT%(TCR)/T.CoreR.	RQD %
6 7	K-2	6.00							Definition is given at page 1/2. **7.00 m**		III IV	III	2 - 10	80	22
8	K-3	7.50							MUDSTONE Brown colored-reddish brown colored, friable-crumbly,incompetent-very incompetent, highly-totally weathered. Nonlayered-thick layered.		IV V	IV V		90	0
9 10	K-4	9.00												80	19
11	K-5	10.50												80	0
12 13 14 15 16		**12.00**							Length: **12.00 m** **1/2 (4.94 - 9.65 m)** **2/2 (9.65 - 12.00 m)**						

	LOGU YAPAN Logged By	KONTROL Checked	
İSİM Name	Aykut AYDERMAN Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh.	Metin ÖZ Jeot. Hiz. Müdürü
İMZA Sign			



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SONDAJ LOGU / BORING LOG

SONDAJ Borehole	No :	KY - 135
SAYFA Page	No :	1 / 2

PROJE ADI / Project Name	: KAYAŞ-YERKÖY DEMİRYOLU	BAŞ.BİT.TAR. / Start Finish Date	: 04.05.2011 - 05.05.2011
SONDAJ YERİ / Boring Location	: ALT GEÇİT	MUH.BOR.DER. / Casing Depth	: 4.50 m (NW)
KİLOMETRE / Chainage	: 128+457	YASS ve Ölçüm Tarihi / GWL & Date	: 0.80 m. / 09.06.2011
SONDAJ DER. / Boring Depth	: 10.50 m	KOOR. SİSTEMİ /Coor. System	: TM33-WGS84
DELİK ÇAPI / Hole Diameter	: NW	KOORDİNAT / Coordinate (N-S) X	: 4 410 611
SONDAJ MAK. & YÖNT. / D.Rig & Met.	: BL- D500 - Rotary	KOORDİNAT / Coordinate (E-W) Y	: 590 361
SONDÖR / Driller	: Yavuz ERGİNER	SONDAJ KOTU / Elevation (m)	: 666.26 m

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test — DARBE SAYISI Numb. Of Blows 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	GRAFİK Graph 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA / Weathering	KIRIK / Fracture (30cm)	KAROT%(TCR)/T. CoreR.	RQD %
0									STABILIZED ROAD FILLING 0.30 m						
1									Red-reddish Brown-wine colored, stiff, sandy silty clay. Humid, with moderate-high plasticity;5-10% fine grained,sandy.						
2	SPT-1	1.50 1.95	SPT-1	3	4	6	10	10							
3	SPT-2	3.00 3.45	SPT-2	9	12	14	26	26	2.80 m MUDSTONE Red-reddish brown-wine colored, soft-friable, locally stiff, incompetent-very incompetent. Highly-completely weathered, locally moderately weathered. Discontinuities; 0°,10°, 70° open, shiny, greasy, White colored, filled with a trace of gypsum.						
4											V	IV V	2-10		
5	SPT-3	4.50 4.79	SPT-3	29	50/14	-	R	R							
6		6.00													

DAYANIMLILIK / Strength			AYRIŞMA / Weathering			İNCE TANELİ / Fine Grained			İRİ TANELİ/Coarse Grained		
I	DAYANIMLI	Strong	I	TAZE	Fresh	N : 0-2	ÇOK YUMUŞAK	V.Soft	N 0-4	Ç.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.	Highly 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	%20-50	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
BST	Basınçlı Su Testi Water Pressure Test	P	Pressiyometre Deneyi Pressuremeter Test	İSİM Name: Onur ÖZDEMİR Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh. Metin ÖZ Jeot. Hiz. Müdürü
UD	Örselenmemiş Numune Undisturbed Sample	k	Permeabilite Deneyi Permeability Test	İMZA Sign	

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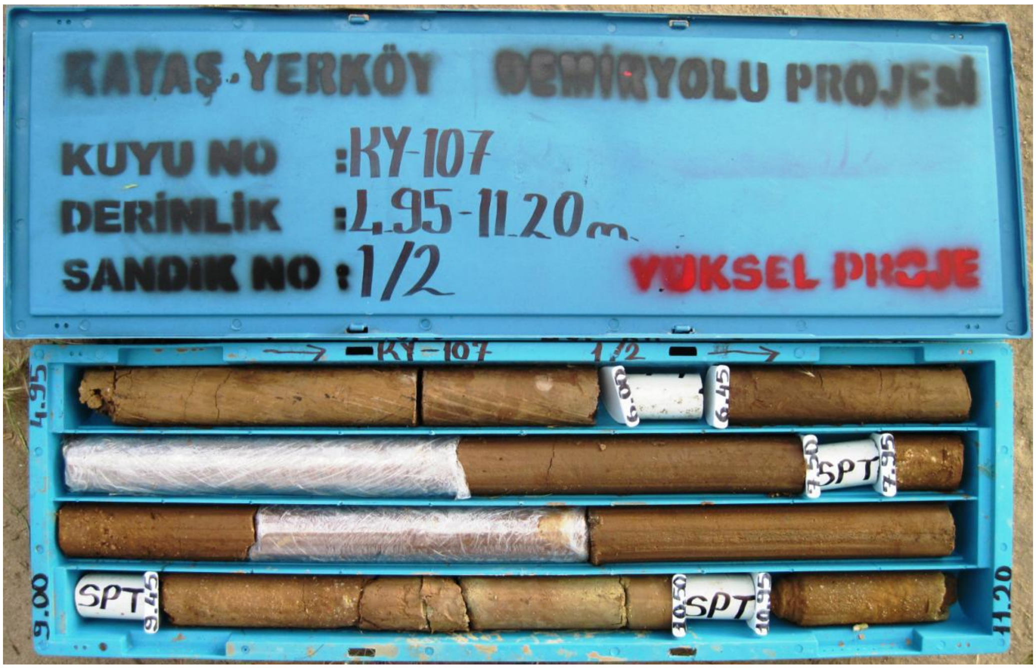
SONDAJ LOGU / BORING LOG

SONDAJ Borehole	No :	KY - 135
SAYFA Page	No :	2 / 2

SONDAJ DERİNLİĞİ Boring Depth (m)	NUMUNE CİNSİ Sample Type	MANEVRA BOYU Run	YERİNDE DENEY In-Situ Test	STANDART PENETRASYON DENEYİ Standart Penetration Test — DARBE SAYISI Numb. Of Blows: 0 - 15 cm	15 - 30 cm	30 - 45 cm	N	GRAFİK Graph: 10 20 30 40 50 60	JEOTEKNİK TANIMLAMA Geotechnical Description	PROFİL Profile	DAYANIMLILIK/Strength	AYRIŞMA / Weathering	KIRIK / Fracture (30cm)	KAROT%(TCR)/T.CoreR.	RQD %
6	SPT-4	6.00	SPT-4	50/12	-	-	R	R			V				
7	K-1	6.12							MUDSTONE Red-reddish brown-wine colored, soft-friable, locally stiff, incompetent-very incompetent. Highly-completely weathered, locally moderately weathered. Discontinuities; 0°, 10°, 70° open, shiny, greasy, White colored, filled with a trace of gypsum.		IV V	IV V	2-10	100	90
8	K-2	7.50												70	40
9 10	K-3	9.00							9.30 m SANDSTONE Red-reddish brown-wine colored, friable-stiff, incompetent-moderately strong sandstone. Upper and middle levels are highly-weathered. very fine-grained, with clay-silt matrix. Discontinuities; 0°, 10°, open, opaque, rough, filled clay.		III IV	III IV		73	48
		10.5													
11															
12															
13									Length: **10.50 m** **1/1 (6.12 - 10.50 m)**						
14															
15															
16															

	LOGU YAPAN Logged By	KONTROL Checked	
İSİM Name	Onur ÖZDEMİR Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Müh.	Metin ÖZ Jeot. Hiz. Müdürü
İMZA Sign			

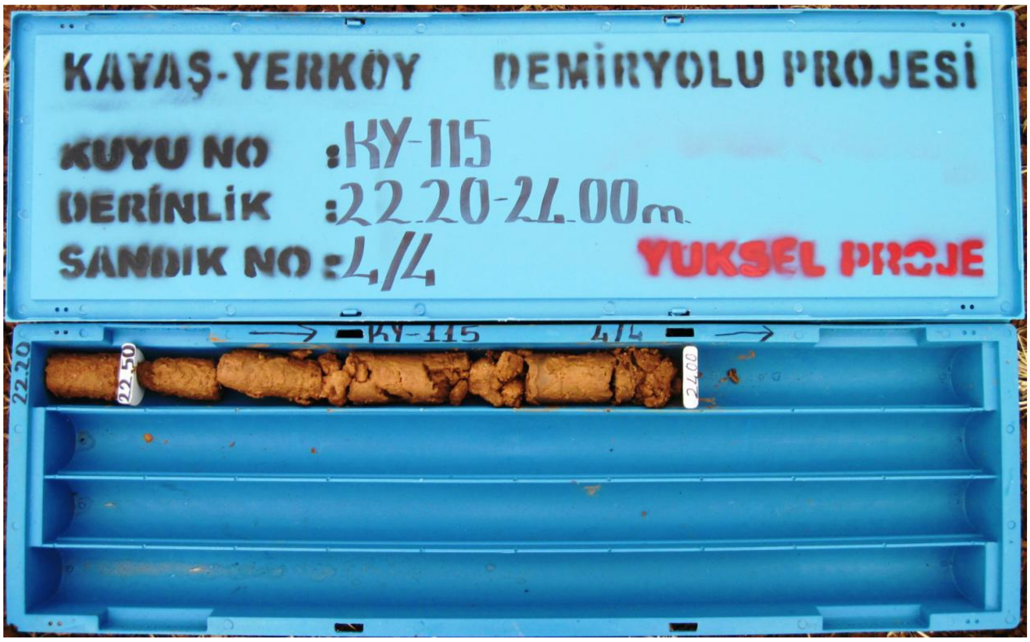
B. DRILLING SAMPLES




KAYAŞ-YERKOY DEMİRYOLU PROJESİ
KUYU NO :KY-115
DERİNLİK :7.95-12.00m.
SANDIK NO :1/4
YUKSEL PROJE
KY-115 1/4
7.95
9.00
10.50
12.00


KAYAŞ-YERKOY DEMİRYOLU PROJESİ
KUYU NO :KY-115
DERİNLİK :12.00-15.60m.
SANDIK NO :2/4
YUKSEL PROJE
KY-115 2/4
12.00
13.50
15.00
15.60


KAYAŞ-YERKÖY DEMİRYOLU PROJESİ
KUYU NO :KY-115
DERİNLİK :15.60-22.20m.
SANDIK NO :3/4
YUKSEL PROJE
KY-115
3/4
15.60
16.50
18.00
19.50
21.00
22.20

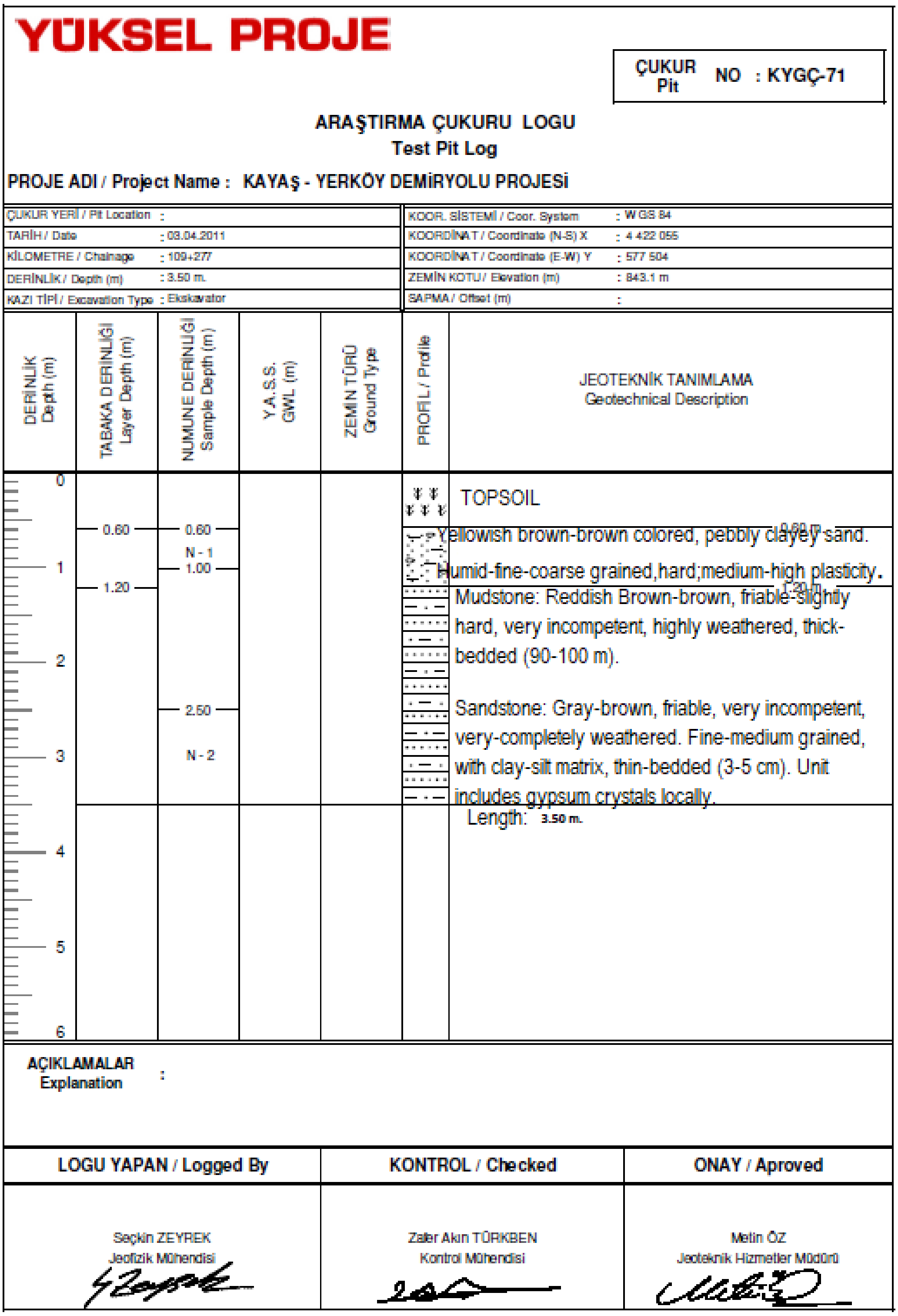

KAYAŞ-YERKÖY DEMİRYOLU PROJESİ
KUYU NO :KY-115
DERİNLİK :22.20-24.00m.
SANDIK NO :4/4
YUKSEL PROJE
KY-115
4/4
22.20
22.50
24.00


KAYAŞ-YERKÖY DEMİRYOLU PROJESİ
KUYU NO :KY-128
DERİNLİK :4.94-9.65 m
SANDIK NO :1/2
YÜKSEL PROJE
KY-128 1/2
4.94
7.50
9.65
6.00
9.00


KAYAŞ-YERKÖY DEMİRYOLU PROJESİ
KUYU NO :KY-128
DERİNLİK :9.65-12.00 m
SANDIK NO :2/2
YÜKSEL PROJE
KY-128 2/2
9.65
12.00


KAYAŞ-YERKÖY DEMİRYOLU PROJESİ
KUYU NO : KY-135
DERİNLİK : 6.12-10.50 m
SANDIK NO : 1/1
YÜKSEL PROJE
KY-135 1/1
6.12
7.50
9.00
10.50

C. TEST PIT LOGS



YÜKSEL PROJE

ÇUKUR Pit NO : KYGÇ-71

ARAŞTIRMA ÇUKURU LOGU
Test Pit Log

PROJE ADI / Project Name : KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

ÇUKUR YERİ / Pit Location	:	KOOR. SİSTEMİ / Coor. System	: WGS 84
TARİH / Date	: 03.04.2011	KOORDİNAT / Coordinate (N-S) X	: 4 422 055
KİLOMETRE / Chainage	: 109+277	KOORDİNAT / Coordinate (E-W) Y	: 577 504
DERİNLİK / Depth (m)	: 3.50 m.	ZEMİN KOTU / Elevation (m)	: 843.1 m
KAZI TİPİ / Excavation Type	: Ekskavator	SAPMA / Offset (m)	:

DERİNLİK Depth (m)	TABAKA DERİNLİĞİ Layer Depth (m)	NUMUNE DERİNLİĞİ Sample Depth (m)	Y.A.S.S. GWL (m)	ZEMİN TÜRÜ Ground Type	PROFİL / Profile	JEOTEKNİK TANIMLAMA Geotechnical Description
0						TOPSOIL
1	0.60	0.60 N - 1 1.00				0.60 m Yellowish brown-brown colored, pebbly clayey sand. Humid-fine-coarse grained,hard;medium-high plasticity.
2 3	1.20	2.50 N - 2				1.20 m Mudstone: Reddish Brown-brown, friable-slightly hard, very incompetent, highly weathered, thick-bedded (90-100 m). Sandstone: Gray-brown, friable, very incompetent, very-completely weathered. Fine-medium grained, with clay-silt matrix, thin-bedded (3-5 cm). Unit includes gypsum crystals locally.
4 5 6						Length: 3.50 m.

AÇIKLAMALAR
Explanation :

LOGU YAPAN / Logged By	**KONTROL / Checked**	**ONAY / Aproved**
Seçkin ZEYREK Jeofizik Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeoteknik Hizmetler Müdürü

YÜKSEL PROJE

ÇUKUR Pit	NO : KYAÇ-35

ARAŞTIRMA ÇUKURU LOGU
Test Pit Log

PROJE ADI / Project Name : KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

ÇUKUR YERİ / Pit Location	:	KOOR. SİSTEMİ / Coor. System	: WGS 84
TARİH / Date	: 03.04.2011	KOORDİNAT / Coordinate (N-S) X	: 4 422 157
KİLOMETRE / Chainage	: 113+032	KOORDİNAT / Coordinate (E-W) Y	: 581 069
DERİNLİK / Depth (m)	: 3.50 m.	ZEMİN KOTU / Elevation (m)	: 793.4 m
KAZI TİPİ / Excavation Type	: Ekskavator	SAPMA / Offset (m)	:

DERİNLİK Depth (m)	TABAKA DERİNLİĞİ Layer Depth (m)	NUMUNE DERİNLİĞİ Sample Depth (m)	Y.A.S.S. GWL (m)	ZEMİN TÜRÜ Ground Type	PROFİL / Profile	JEOTEKNİK TANIMLAMA Geotechnical Description
0						TOPSOIL
	0.30					0.30 m.
1						MUDSTONE Reddish Brown-claret red colored, friable-slightly hard, incompetent-very incompetent, highly-completely weathered. Discontinuities are locally coated with MnO. Gypsum crystals are observed locally.
2		2.00				
		N - 1				
3						
						Length: 3.50 m.
4						
5						
6						

AÇIKLAMALAR Explanation :

LOGU YAPAN / Logged By	KONTROL / Checked	ONAY / Aproved
Seçkin ZEYREK Jeofizik Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeoteknik Hizmetler Müdürü

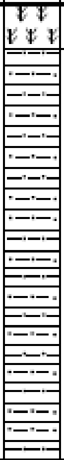
YÜKSEL PROJE

ÇUKUR Pit	NO : KYAÇ-42

ARAŞTIRMA ÇUKURU LOGU
Test Pit Log

PROJE ADI / Project Name : KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

ÇUKUR YERİ / Pit Location	:	KOOR. SİSTEMİ / Coor. System	: WGS 84
TARİH / Date	: 06.04.2011	KOORDİNAT / Coordinate (N-S) X	: 4 415 420
KİLOMETRE / Chainage	: 121+210	KOORDİNAT / Coordinate (E-W) Y	: 584 959
DERİNLİK / Depth (m)	: 4.00 m.	ZEMİN KOTU / Elevation (m)	: 704.66 m
KAZI TİPİ / Excavation Type	: Ekskavator	SAPMA / Offset (m)	:

DERİNLİK Depth (m)	TABAKA DERİNLİĞİ Layer Depth (m)	NUMUNE DERİNLİĞİ Sample Depth (m)	Y.A.S.S. GWL (m)	ZEMİN TÜRÜ Ground Type	PROFİL / Profile	JEOTEKNİK TANIMLAMA Geotechnical Description
0						TOPSOIL
	0.40					0.40 m.
1						MUDSTONE
	1.50	1.50				Reddish Brown-claret red colored, soft-friable-slightly hard, incompetent-very incompetent, highly-completely weathered.
2		N - 1				
		2.50				Gypsum crystals are observed locally within the unit. Upper parts (0.50-1.10m) are totally weathered.
3						
4						Length: 4.00 m.
5						
6						

AÇIKLAMALAR
Explanation :

LOGU YAPAN / Logged By	KONTROL / Checked	ONAY / Aproved
Seçkin ZEYREK Jeofizik Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeoteknik Hizmetler Müdürü

YÜKSEL PROJE

ÇUKUR Pit NO : KYGÇ-87

ARAŞTIRMA ÇUKURU LOGU
Test Pit Log

PROJE ADI / Project Name : KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

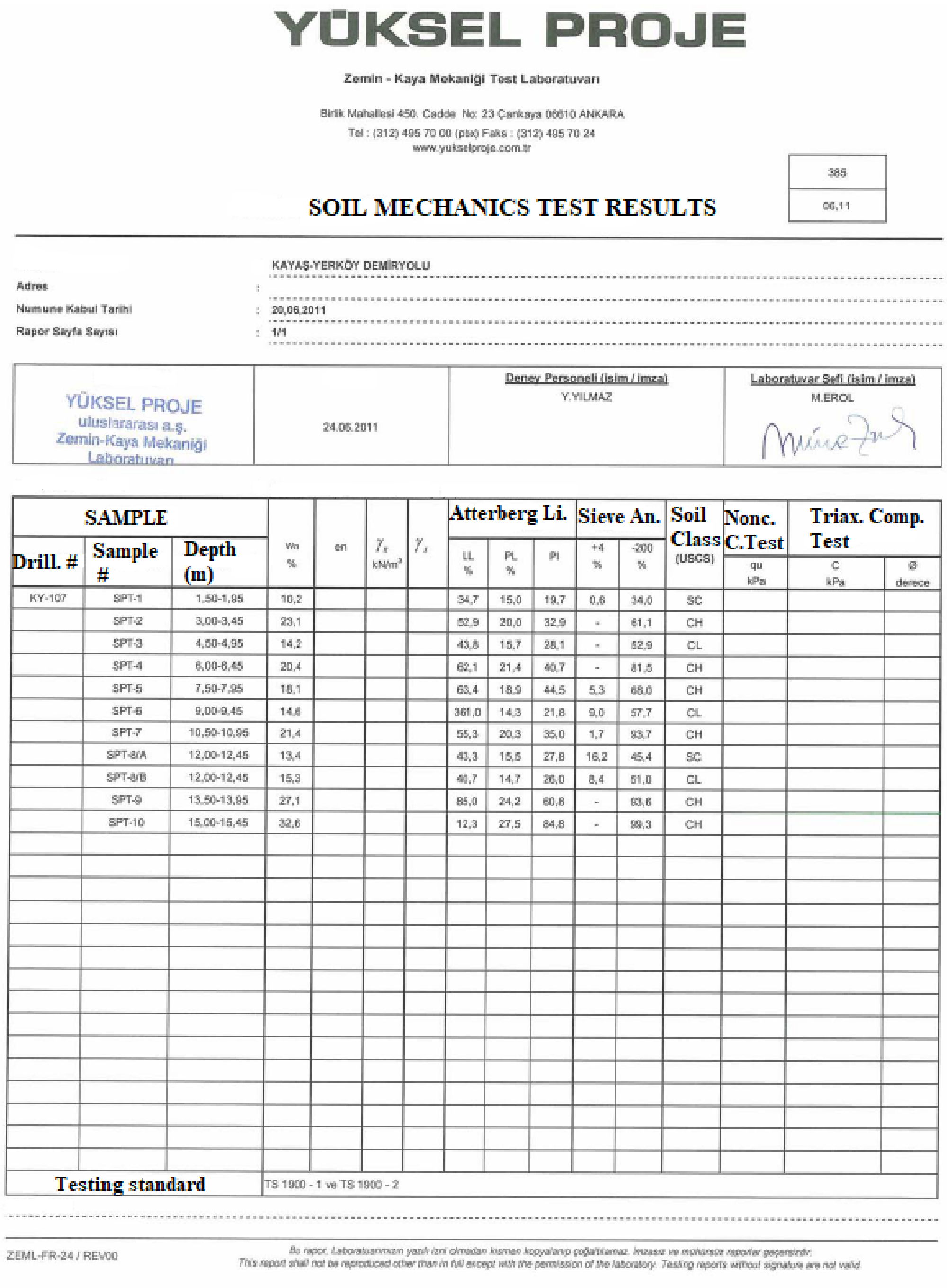
ÇUKUR YERİ / Pit Location	:	KOOR. SİSTEMİ / Coor. System	: WGS 84
TARİH / Date	: 07.04.2011	KOORDİNAT / Coordinate (N-S) X	: 4 410 673
KİLOMETRE / Chainage	: 128+380	KOORDİNAT / Coordinate (E-W) Y	: 590 314
DERİNLİK / Depth (m)	: 4.00 m.	ZEMİN KOTU / Elevation (m)	: 663.3 m
KAZI TİPİ / Excavation Type	: Ekskavator	SAPMA / Offset (m)	:

DERİNLİK Depth (m)	TABAKA DERİNLİĞİ Layer Depth (m)	NUMUNE DERİNLİĞİ Sample Depth (m)	Y.A.S.S. GWL (m)	ZEMİN TÜRÜ Ground Type	PROFİL / Profile	JEOTEKNİK TANIMLAMA Geotechnical Description
0 1 2 3 4 5 6	0.40	1.50 N - 1 2.50	2.10 m			TOPSOIL 0.40 m. MUDSTONE Reddish brown-claret red colored, soft-friable-slightly hard, incompetent-very incompetent, highly-completely weathered. Local gypsum crystals in the unit and MnO coatings within discontinuity surfaces are observed. Upper parts (0.50-1.10m) are totally weathered. Upper levels (0.40-1.90 m) are completely weathered. **ÇUKUR SONU: 4.00 m.**

AÇIKLAMALAR Explanation :

LOGU YAPAN / Logged By	KONTROL / Checked	ONAY / Aproved
Seçkin ZEYREK Jeofizik Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeoteknik Hizmetler Müdürü

D. LABORATORY DATA ON EXPLORATORY DRILLING SAMPLES



YÜKSEL PROJE

Zemin - Kaya Mekaniği Test Laboratuvarı

Birlik Mahallesi 450. Cadde No: 23 Çankaya 06610 ANKARA
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06,11

SOIL MECHANICS TEST RESULTS

KAYAŞ-YERKÖY DEMİRYOLU

Adres :
Numune Kabul Tarihi : 20,06,2011
Rapor Sayfa Sayısı : 1/1

YÜKSEL PROJE uluslararası a.ş. Zemin-Kaya Mekaniği Laboratuvarı	24.06.2011	Deney Personeli (isim / imza) Y.YILMAZ	Laboratuvar Şefi (isim / imza) M.EROL

SAMPLE			Wn %	en	γ_s kN/m³	γ_s	Atterberg Li.			Sieve An.		Soil Class (USCS)	Nonc. C.Test	Triax. Comp. Test	
Drill. #	Sample #	Depth (m)					LL %	PL %	PI	+4 %	-200 %		qu kPa	C kPa	Ø derece
KY-107	SPT-1	1,50-1,95	10,2				34,7	15,0	19,7	0,6	34,0	SC			
	SPT-2	3,00-3,45	23,1				52,9	20,0	32,9	-	61,1	CH			
	SPT-3	4,50-4,95	14,2				43,8	15,7	28,1	-	52,9	CL			
	SPT-4	6,00-6,45	20,4				62,1	21,4	40,7	-	81,5	CH			
	SPT-5	7,50-7,95	18,1				63,4	18,9	44,5	5,3	68,0	CH			
	SPT-6	9,00-9,45	14,6				361,0	14,3	21,8	9,0	57,7	CL			
	SPT-7	10,50-10,95	21,4				55,3	20,3	35,0	1,7	93,7	CH			
	SPT-8/A	12,00-12,45	13,4				43,3	15,5	27,8	16,2	45,4	SC			
	SPT-8/B	12,00-12,45	15,3				40,7	14,7	26,0	8,4	51,0	CL			
	SPT-9	13,50-13,95	27,1				85,0	24,2	60,8	-	93,6	CH			
	SPT-10	15,00-15,45	32,6				12,3	27,5	84,8	-	99,3	CH			
Testing standard	TS 1900 - 1 ve TS 1900 - 2														

ZEML-FR-24 / REV00

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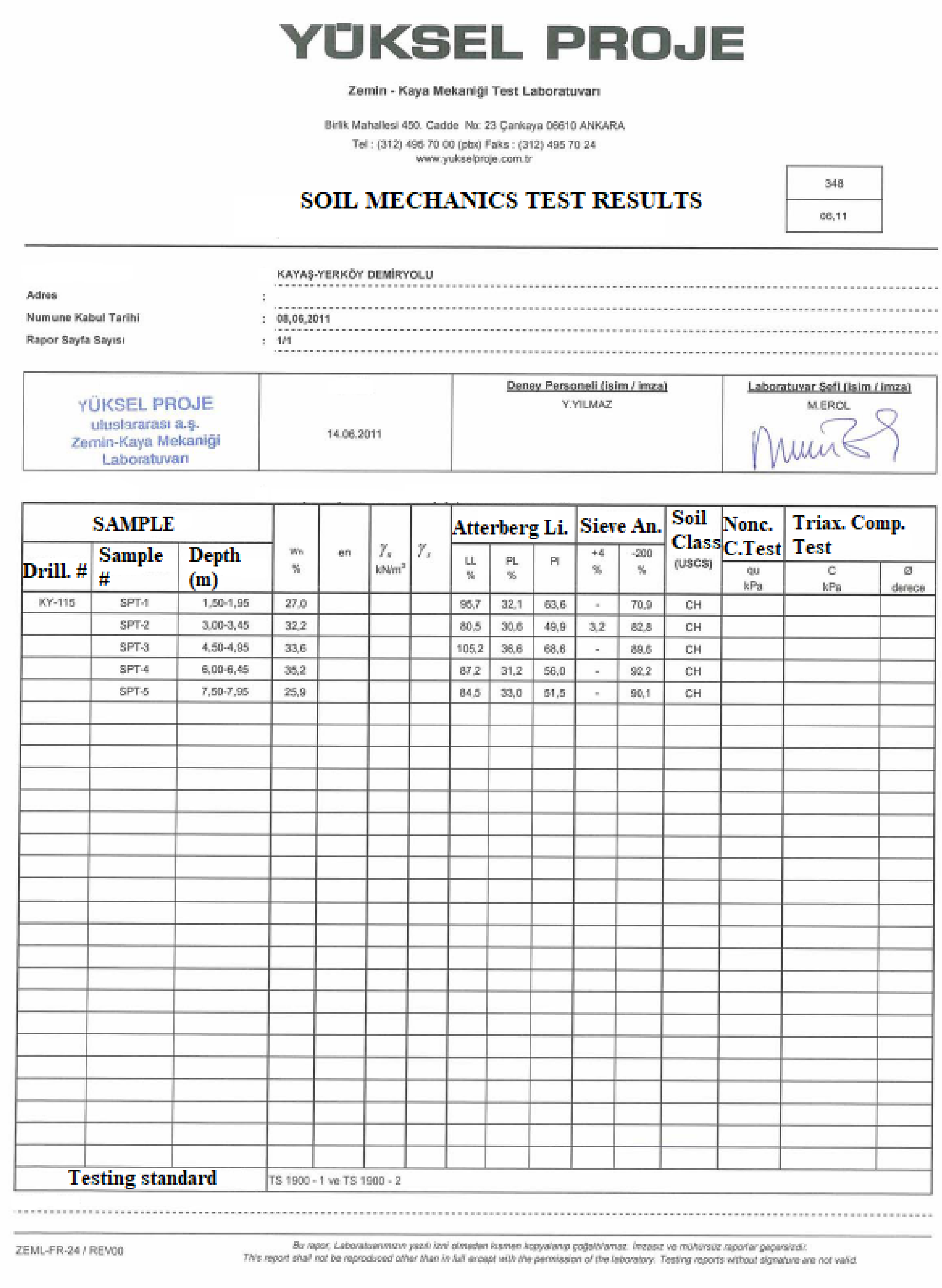

YÜKSEL PROJE

Zemin - Kaya Mekaniği Test Laboratuvarı

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06,11

SOIL MECHANICS TEST RESULTS

KAYAŞ-YERKÖY DEMİRYOLU

Adres :

Numune Kabul Tarihi : 08,06,2011

Rapor Sayfa Sayısı : 1/1

		Deney Personeli (isim / imza)	Laboratuvar Şefi (isim / imza)
YÜKSEL PROJE uluslararası a.ş. Zemin-Kaya Mekaniği Laboratuvarı	14.06.2011	Y.YILMAZ	M.EROL

SAMPLE							Atterberg Li.			Sieve An.		Soil Class	Nonc. C.Test	Triax. Comp. Test	
Drill. #	Sample #	Depth (m)	Wn %	en	γ_n kN/m³	γ_s	LL %	PL %	PI	+4 %	-200 %	(USCS)	qu kPa	C kPa	Ø derece
KY-115	SPT-1	1,50-1,95	27,0				95,7	32,1	63,6	-	70,9	CH			
	SPT-2	3,00-3,45	32,2				80,5	30,6	49,9	3,2	82,8	CH			
	SPT-3	4,50-4,95	33,6				105,2	36,6	68,6	-	89,6	CH			
	SPT-4	6,00-6,45	35,2				87,2	31,2	56,0	-	92,2	CH			
	SPT-5	7,50-7,95	25,9				84,5	33,0	51,5	-	90,1	CH			
Testing standard			TS 1900 - 1 ve TS 1900 - 2												

ZEML-FR-24 / REV00

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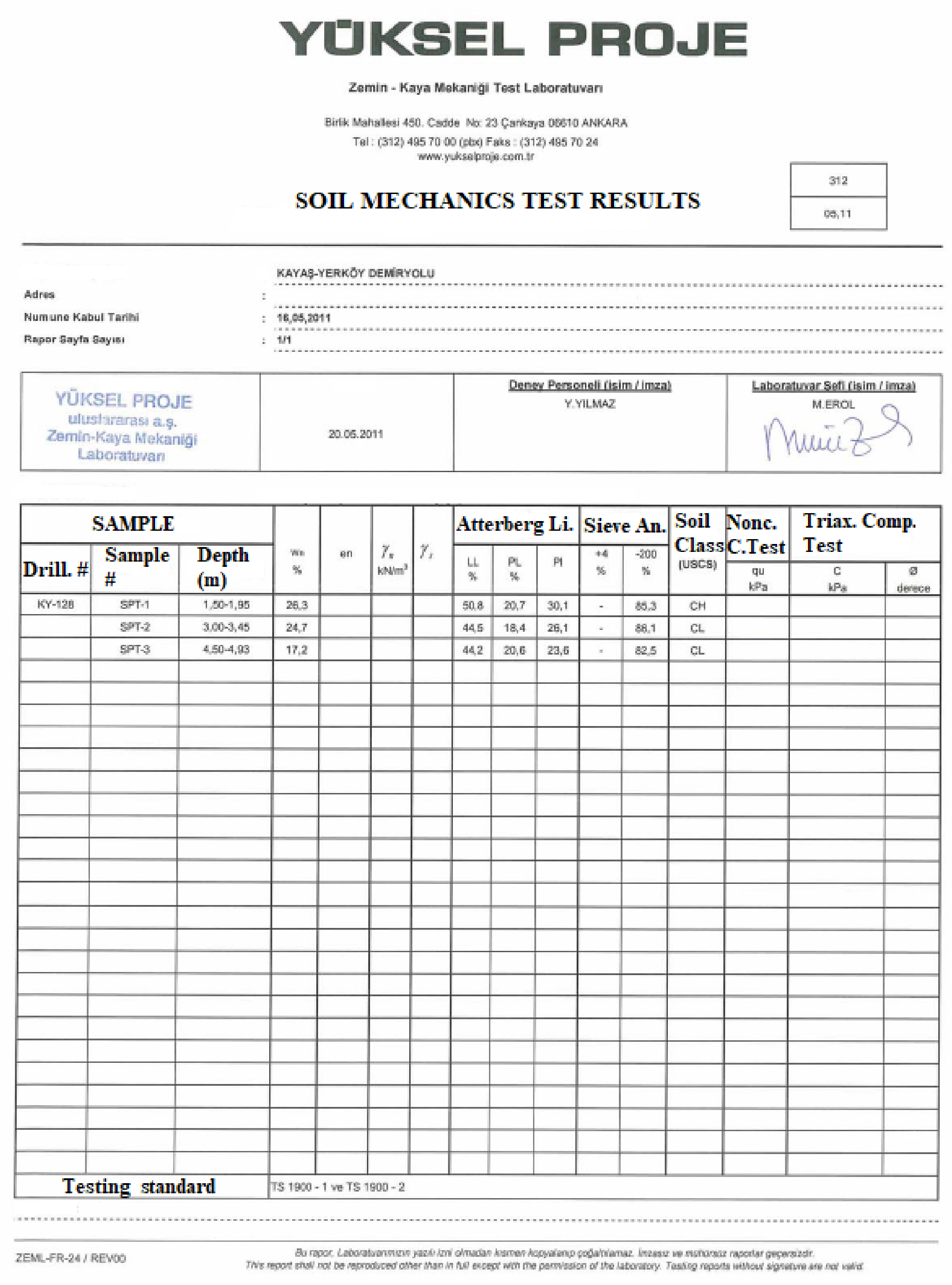

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SOIL MECHANICS TEST RESULTS

KAYAŞ-YERKÖY DEMİRYOLU
Adres :
Numune Kabul Tarihi : 16,05,2011
Rapor Sayfa Sayısı : 1/1

		Deney Personeli (isim / imza)	Laboratuvar Şefi (isim / imza)
YÜKSEL PROJE uluslararası a.ş. Zemin-Kaya Mekaniği Laboratuvarı	20.06.2011	Y.YILMAZ	M.EROL

SAMPLE			Wn %	en	γ_n kN/m^3	γ_s	Atterberg Li.			Sieve An.		Soil Class (USCS)	Nonc. C.Test	Triax. Comp. Test	
Drill. #	Sample #	Depth (m)					LL %	PL %	PI	+4 %	-200 %		qu kPa	c kPa	Ø derece
KY-128	SPT-1	1,50-1,95	26,3				50,8	20,7	30,1	-	85,3	CH			
	SPT-2	3,00-3,45	24,7				44,5	18,4	26,1	-	86,1	CL			
	SPT-3	4,50-4,93	17,2				44,2	20,6	23,6	-	82,5	CL			
Testing standard			TS 1900 - 1 ve TS 1900 - 2												

ZEML-FR-24 / REV00

Bu rapor, Laboratuarımızın yazılı izni olmadan kısmen kopyalanıp çoğaltılamaz. İmzasız ve mühürsüz raporlar geçersizdir.
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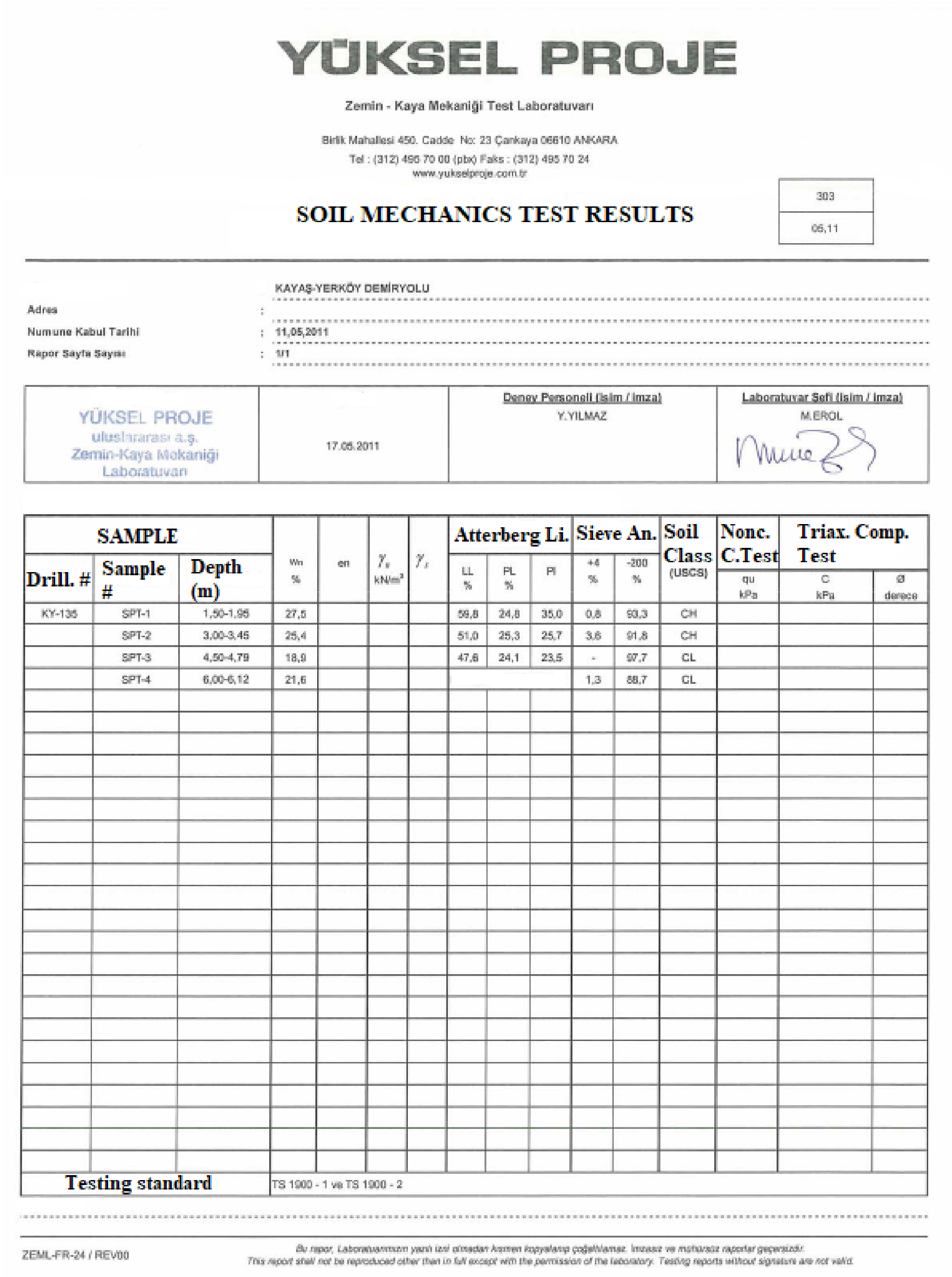

YÜKSEL PROJE

Zemin - Kaya Mekaniği Test Laboratuvarı

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SOIL MECHANICS TEST RESULTS

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05,11

KAYAŞ-YERKÖY DEMİRYOLU
Adres :
Numune Kabul Tarihi : 11,05,2011
Rapor Sayfa Sayısı : 1/1

YÜKSEL PROJE uluslararası a.ş. Zemin-Kaya Mekaniği Laboratuvarı	17.05.2011	Deney Personeli (İsim / İmza) Y.YILMAZ	Laboratuvar Şefi (İsim / İmza) M.EROL

SAMPLE			W_n %	en	γ_n kN/m³	γ_s	Atterberg Li.			Sieve An.		Soil Class (USCS)	Nonc. C.Test	Triax. Comp. Test	
Drill. #	Sample #	Depth (m)					LL %	PL %	PI	+4 %	-200 %		qu kPa	C kPa	Ø derece
KY-136	SPT-1	1,50-1,95	27,5				59,8	24,8	35,0	0,8	93,3	CH			
	SPT-2	3,00-3,45	25,4				51,0	25,3	25,7	3,6	91,8	CH			
	SPT-3	4,50-4,79	18,9				47,6	24,1	23,5	-	97,7	CL			
	SPT-4	6,00-6,12	21,6							1,3	88,7	CL			
Testing standard			TS 1900 - 1 ve TS 1900 - 2												

ZEML-FR-24 / REV00

Bu rapor, Laboratuvarımızın yazılı izni olmadan kısmen kopyalanıp çoğaltılamaz. İmzasız ve mühürsüz raporlar geçersizdir.
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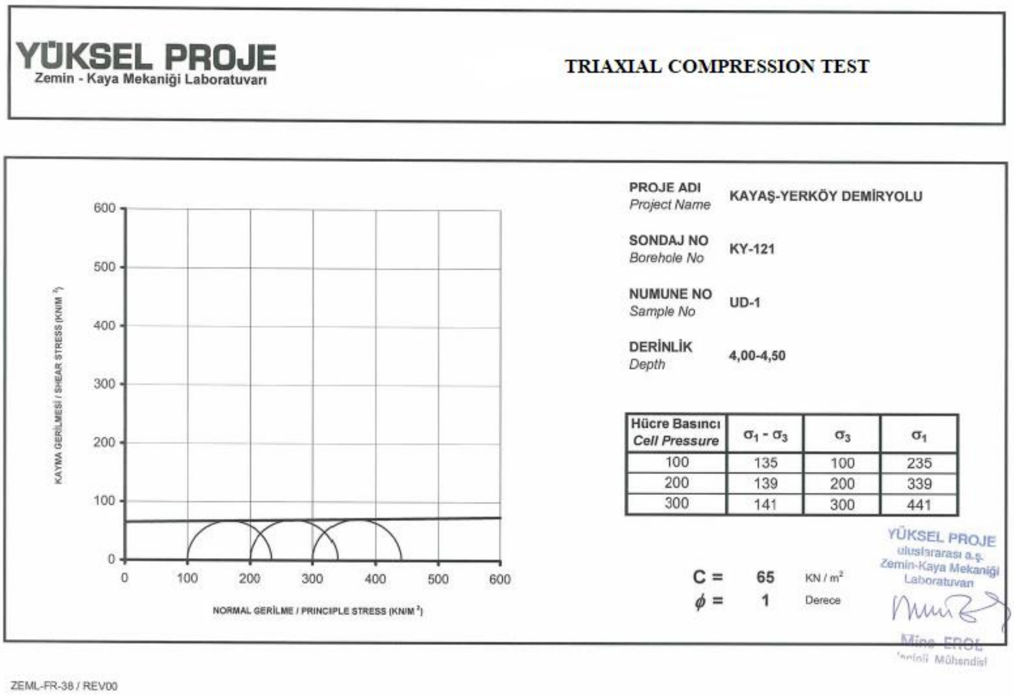
E. LABORATORY DATA ON UNDISTURBED SAMPLES (UD)



YÜKSEL PROJE
Zemin - Kaya Mekaniği Laboratuvarı

TRIAXIAL COMPRESSION TEST

PROJE ADI *Project Name* KAYAŞ-YERKÖY DEMİRYOLU

SONDAJ NO *Borehole No* KY-121

NUMUNE NO *Sample No* UD-1

DERİNLİK *Depth* 4,00-4,50

Hücre Basıncı *Cell Pressure*	$\sigma_1 - \sigma_3$	σ_3	σ_1
100	135	100	235
200	139	200	339
300	141	300	441

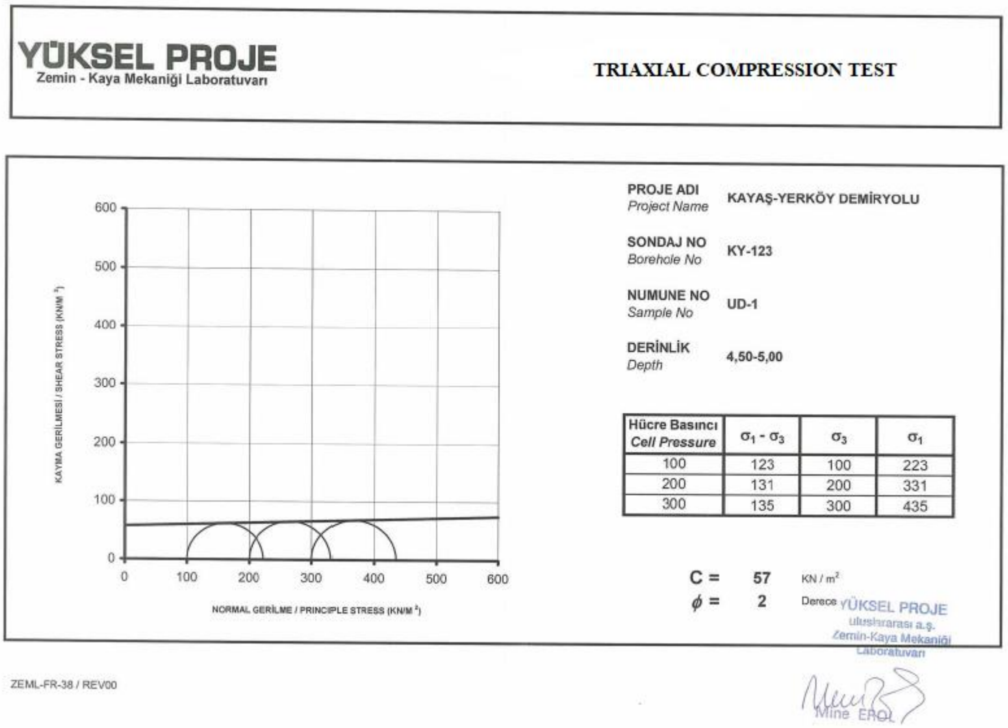
C = 65 KN / m²

ϕ = 1 Derece

YÜKSEL PROJE
uluslararası a.ş.
Zemin-Kaya Mekaniği
Laboratuvarı

Mine EROL
Jeoloji Mühendisi

ZEML-FR-38 / REV00



YÜKSEL PROJE
Zemin - Kaya Mekaniği Laboratuvarı

TRIAXIAL COMPRESSION TEST

PROJE ADI *Project Name* KAYAŞ-YERKÖY DEMİRYOLU

SONDAJ NO *Borehole No* KY-123

NUMUNE NO *Sample No* UD-1

DERİNLİK *Depth* 4,50-5,00

Hücre Basıncı *Cell Pressure*	$\sigma_1 - \sigma_3$	σ_3	σ_1
100	123	100	223
200	131	200	331
300	135	300	435

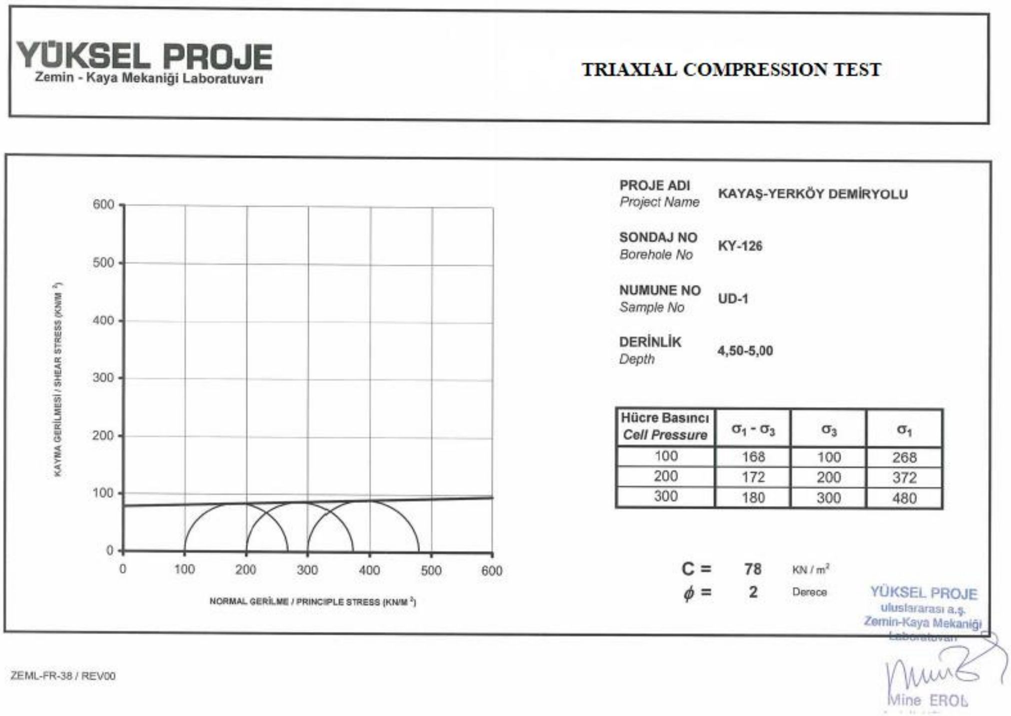
C = 57 KN / m²

ϕ = 2 Derece

YÜKSEL PROJE
uluslararası a.ş.
Zemin-Kaya Mekaniği
Laboratuvarı

ZEML-FR-38 / REV00

Mine EROL



YÜKSEL PROJE
Zemin - Kaya Mekaniği Laboratuvarı

TRIAXIAL COMPRESSION TEST

PROJE ADI *Project Name*: **KAYAŞ-YERKÖY DEMİRYOLU**

SONDAJ NO *Borehole No*: **KY-126**

NUMUNE NO *Sample No*: **UD-1**

DERİNLİK *Depth*: **4,50-5,00**

Hücre Basıncı *Cell Pressure*	$\sigma_1 - \sigma_3$	σ_3	σ_1
100	168	100	268
200	172	200	372
300	180	300	480

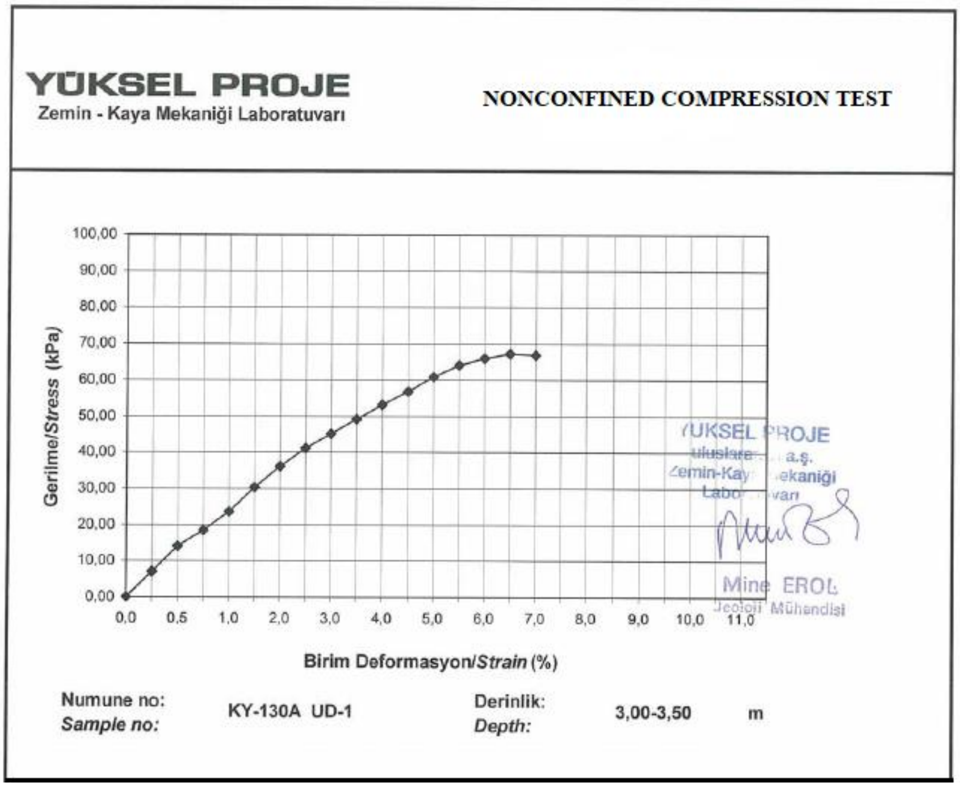
C = **78** KN / m²

ϕ = **2** Derece

YÜKSEL PROJE
uluslararası a.ş.
Zemin-Kaya Mekaniği
Laboratuvarı

Mine EROL

ZEML-FR-38 / REV00



YÜKSEL PROJE
Zemin - Kaya Mekaniği Laboratuvarı

NONCONFINED COMPRESSION TEST

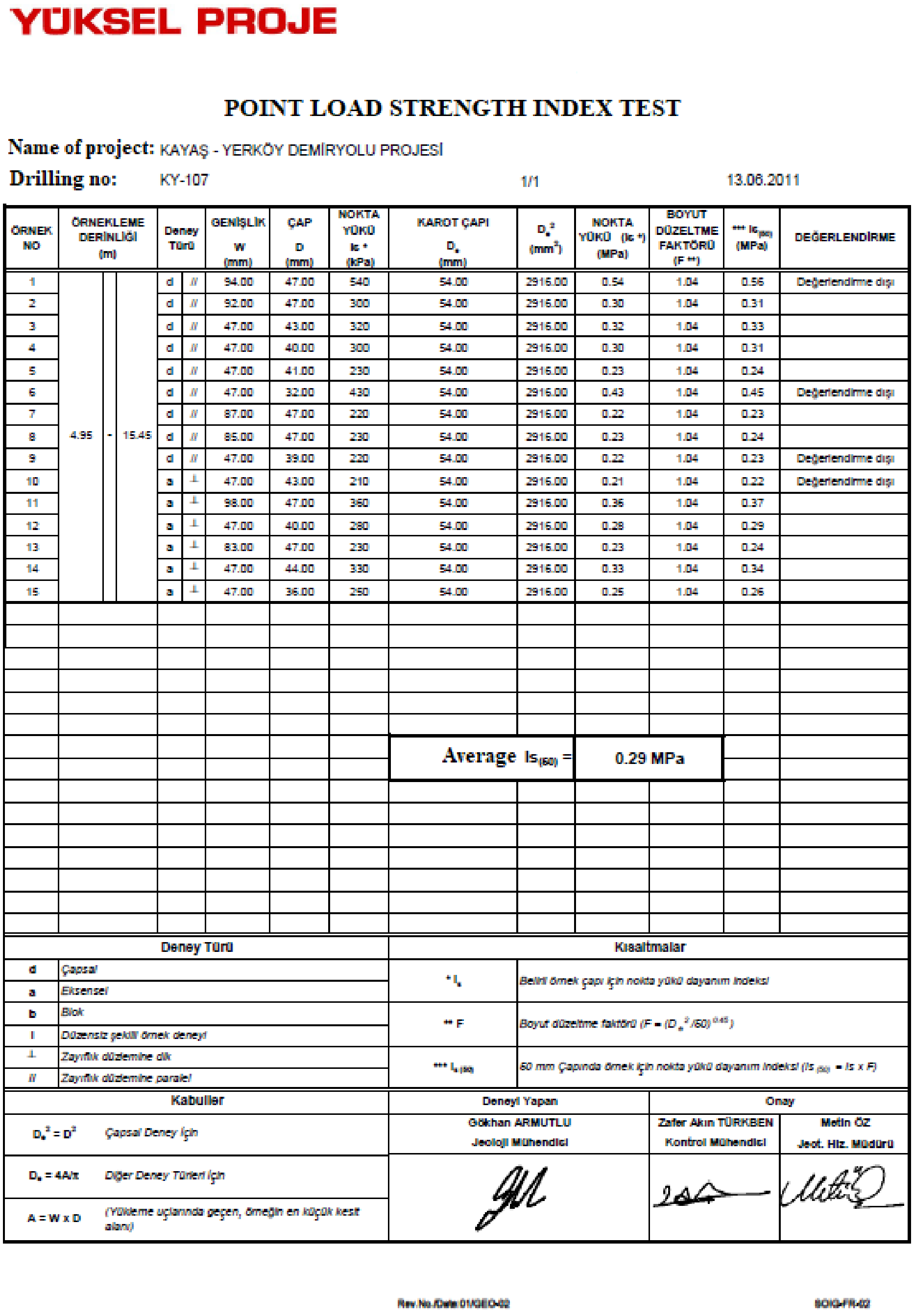
YÜKSEL PROJE
uluslararası a.ş.
Zemin-Kaya Mekaniği
Laboratuvarı

Mine EROL
Jeoloji Mühendisi

Numune no: *Sample no:* **KY-130A UD-1**

Derinlik: *Depth:* **3,00-3,50 m**

F. POINT LOAD TEST DATA ON EXPLORATORY DRILLING



YÜKSEL PROJE

POINT LOAD STRENGTH INDEX TEST

Name of project: KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

Drilling no: KY-107 1/1 13.06.2011

ÖRNEK NO	ÖRNEKLEME DERİNLİĞİ (m)	Deney Türü		GENİŞLİK W (mm)	ÇAP D (mm)	NOKTA YÜKÜ Ic * (kPa)	KAROT ÇAPI D_e (mm)	D_e^2 (mm^2)	NOKTA YÜKÜ (Ic *) (MPa)	BOYUT DÜZELTME FAKTÖRÜ (F **)	*** $Ic_{(50)}$ (MPa)	DEĞERLENDİRME
1	4.95 - 15.45	d	//	94.00	47.00	540	54.00	2916.00	0.54	1.04	0.56	Değerlendirme dışı
2		d	//	92.00	47.00	300	54.00	2916.00	0.30	1.04	0.31	
3		d	//	47.00	43.00	320	54.00	2916.00	0.32	1.04	0.33	
4		d	//	47.00	40.00	300	54.00	2916.00	0.30	1.04	0.31	
5		d	//	47.00	41.00	230	54.00	2916.00	0.23	1.04	0.24	
6		d	//	47.00	32.00	430	54.00	2916.00	0.43	1.04	0.45	Değerlendirme dışı
7		d	//	87.00	47.00	220	54.00	2916.00	0.22	1.04	0.23	
8		d	//	85.00	47.00	230	54.00	2916.00	0.23	1.04	0.24	
9		d	//	47.00	39.00	220	54.00	2916.00	0.22	1.04	0.23	Değerlendirme dışı
10		a	⊥	47.00	43.00	210	54.00	2916.00	0.21	1.04	0.22	Değerlendirme dışı
11		a	⊥	98.00	47.00	360	54.00	2916.00	0.36	1.04	0.37	
12		a	⊥	47.00	40.00	280	54.00	2916.00	0.28	1.04	0.29	
13		a	⊥	83.00	47.00	230	54.00	2916.00	0.23	1.04	0.24	
14		a	⊥	47.00	44.00	330	54.00	2916.00	0.33	1.04	0.34	
15		a	⊥	47.00	36.00	250	54.00	2916.00	0.25	1.04	0.26	

Average $Is_{(50)}$ = 0.29 MPa

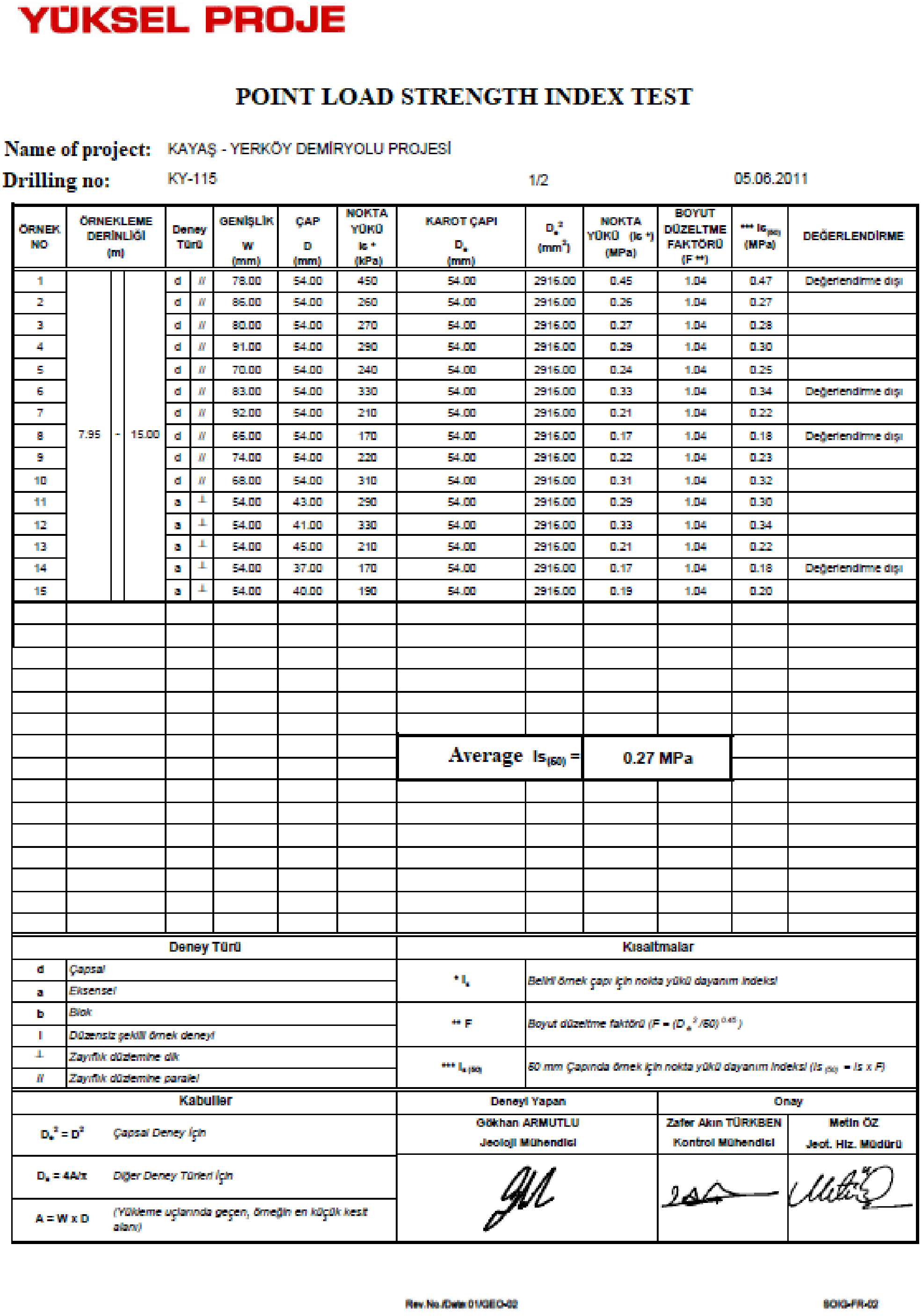
Deney Türü	
d	*Çapsal*
a	*Eksensel*
b	*Blok*
I	*Düzensiz şekilli örnek deneyi*
⊥	*Zayıflık düzlemine dik*
//	*Zayıflık düzlemine paralel*

Kısaltmalar	
* I_s	*Belirli örnek çapı için nokta yükü dayanım indeksi*
** F	*Boyut düzeltme faktörü* ($F = (D_e^2/50)^{0.45}$)
*** $I_{s(50)}$	*50 mm Çapında örnek için nokta yükü dayanım indeksi* ($Is_{(50)} = Is \times F$)

Kabuller	
$D_e^2 = D^2$	*Çapsal Deney İçin*
$D_e = 4A/\pi$	*Diğer Deney Türleri İçin*
$A = W \times D$	*(Yükleme uçlarında geçen, örneğin en küçük kesit alanı)*

Deneyi Yapan	Onay	
Gökhan ARMUTLU **Jeoloji Mühendisi**	**Zafer Akın TÜRKBEN** **Kontrol Mühendisi**	**Metin ÖZ** **Jeot. Hiz. Müdürü**

Rev.No./Date 01/GEO-02 SOIG-FR-02



YÜKSEL PROJE

POINT LOAD STRENGTH INDEX TEST

Name of project: KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

Drilling no: KY-115 1/2 05.06.2011

ÖRNEK NO	ÖRNEKLEME DERİNLİĞİ (m)	Deney Türü		GENİŞLİK W (mm)	ÇAP D (mm)	NOKTA YÜKÜ Is * (kPa)	KAROT ÇAPI D_e (mm)	D_e^2 (mm^2)	NOKTA YÜKÜ (Is *) (MPa)	BOYUT DÜZELTME FAKTÖRÜ (F **)	*** $Is_{(50)}$ (MPa)	DEĞERLENDİRME
1	7.95 - 15.00	d	//	78.00	54.00	450	54.00	2916.00	0.45	1.04	0.47	Değerlendirme dışı
2		d	//	86.00	54.00	260	54.00	2916.00	0.26	1.04	0.27	
3		d	//	80.00	54.00	270	54.00	2916.00	0.27	1.04	0.28	
4		d	//	91.00	54.00	290	54.00	2916.00	0.29	1.04	0.30	
5		d	//	70.00	54.00	240	54.00	2916.00	0.24	1.04	0.25	
6		d	//	83.00	54.00	330	54.00	2916.00	0.33	1.04	0.34	Değerlendirme dışı
7		d	//	92.00	54.00	210	54.00	2916.00	0.21	1.04	0.22	
8		d	//	66.00	54.00	170	54.00	2916.00	0.17	1.04	0.18	Değerlendirme dışı
9		d	//	74.00	54.00	220	54.00	2916.00	0.22	1.04	0.23	
10		d	//	68.00	54.00	310	54.00	2916.00	0.31	1.04	0.32	
11		a	⊥	54.00	43.00	290	54.00	2916.00	0.29	1.04	0.30	
12		a	⊥	54.00	41.00	330	54.00	2916.00	0.33	1.04	0.34	
13		a	⊥	54.00	45.00	210	54.00	2916.00	0.21	1.04	0.22	
14		a	⊥	54.00	37.00	170	54.00	2916.00	0.17	1.04	0.18	Değerlendirme dışı
15		a	⊥	54.00	40.00	190	54.00	2916.00	0.19	1.04	0.20	

Average $Is_{(50)}$ = 0.27 MPa

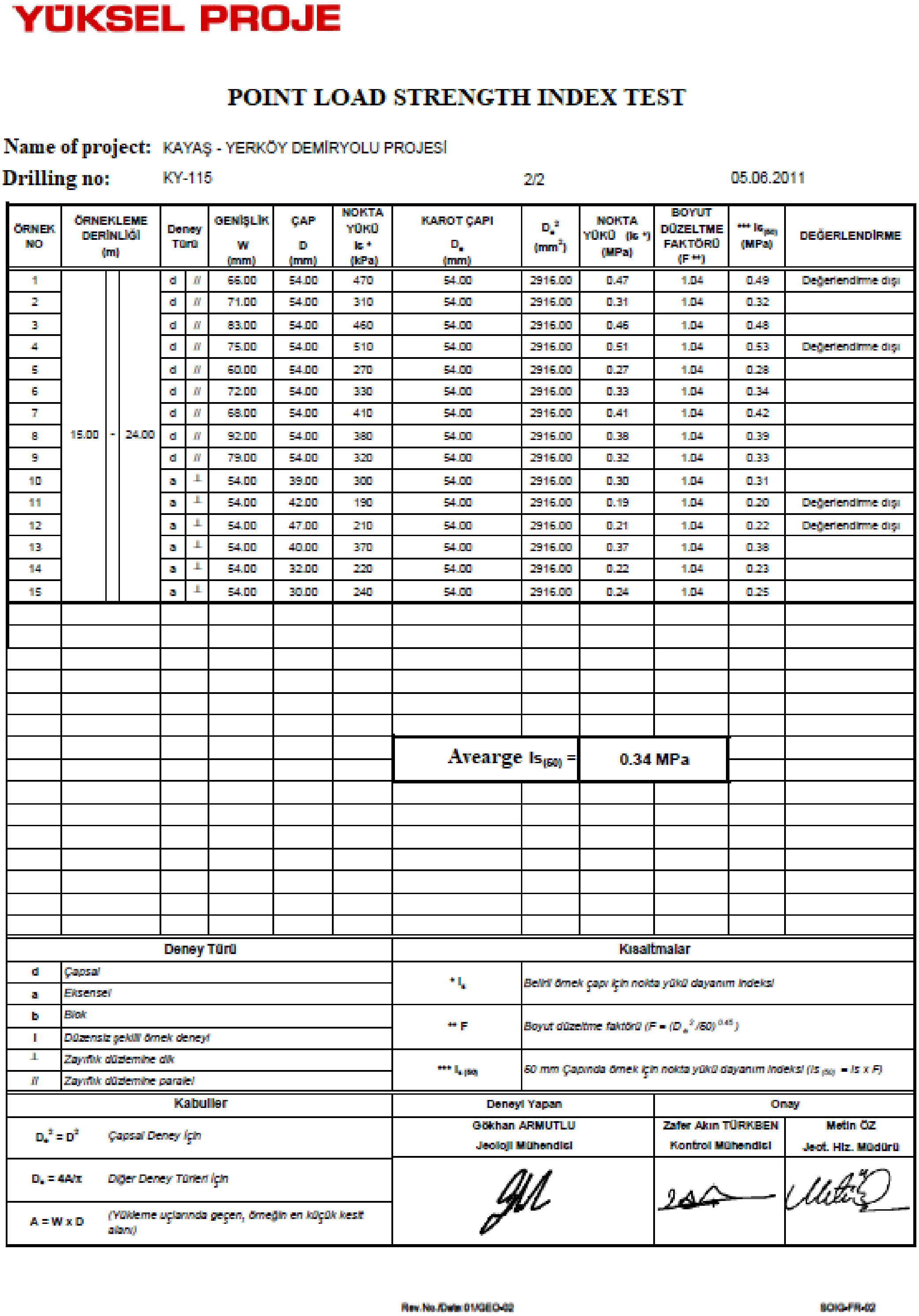
Deney Türü	
d	*Çapsal*
a	*Eksensel*
b	*Blok*
I	*Düzensiz şekilli örnek deneyi*
⊥	*Zayıflık düzlemine dik*
//	*Zayıflık düzlemine paralel*

Kısaltmalar	
* I_s	*Belirli örnek çapı için nokta yükü dayanım indeksi*
** F	*Boyut düzeltme faktörü* ($F = (D_e^2/50)^{0.45}$)
*** $I_{s(50)}$	*50 mm Çapında örnek için nokta yükü dayanım indeksi* ($Is_{(50)} = Is \times F$)

Kabuller	
$D_e^2 = D^2$	*Çapsal Deney İçin*
$D_e = 4A/\pi$	*Diğer Deney Türleri İçin*
A = W x D	*(Yükleme uçlarında geçen, örneğin en küçük kesit alanı)*

Deneyi Yapan	Onay	
Gökhan ARMUTLU Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeot. Hiz. Müdürü

Rev.No./Date 01/GEO-02 SOKG-FR-02



YÜKSEL PROJE

POINT LOAD STRENGTH INDEX TEST

Name of project: KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

Drilling no: KY-115 2/2 05.06.2011

ÖRNEK NO	ÖRNEKLEME DERİNLİĞİ (m)			Deney Türü		GENİŞLİK W (mm)	ÇAP D (mm)	NOKTA YÜKÜ Is * (kPa)	KAROT ÇAPI D$_e$ (mm)	D$_e^2$ (mm^2)	NOKTA YÜKÜ (Is *) (MPa)	BOYUT DÜZELTME FAKTÖRÜ (F **)	*** Is$_{(50)}$ (MPa)	DEĞERLENDİRME
1	15.00	-	24.00	d	//	66.00	54.00	470	54.00	2916.00	0.47	1.04	0.49	Değerlendirme dışı
2				d	//	71.00	54.00	310	54.00	2916.00	0.31	1.04	0.32	
3				d	//	83.00	54.00	460	54.00	2916.00	0.46	1.04	0.48	
4				d	//	75.00	54.00	510	54.00	2916.00	0.51	1.04	0.53	Değerlendirme dışı
5				d	//	60.00	54.00	270	54.00	2916.00	0.27	1.04	0.28	
6				d	//	72.00	54.00	330	54.00	2916.00	0.33	1.04	0.34	
7				d	//	68.00	54.00	410	54.00	2916.00	0.41	1.04	0.42	
8				d	//	92.00	54.00	380	54.00	2916.00	0.38	1.04	0.39	
9				d	//	79.00	54.00	320	54.00	2916.00	0.32	1.04	0.33	
10				a	⊥	54.00	39.00	300	54.00	2916.00	0.30	1.04	0.31	
11				a	⊥	54.00	42.00	190	54.00	2916.00	0.19	1.04	0.20	Değerlendirme dışı
12				a	⊥	54.00	47.00	210	54.00	2916.00	0.21	1.04	0.22	Değerlendirme dışı
13				a	⊥	54.00	40.00	370	54.00	2916.00	0.37	1.04	0.38	
14				a	⊥	54.00	32.00	220	54.00	2916.00	0.22	1.04	0.23	
15				a	⊥	54.00	30.00	240	54.00	2916.00	0.24	1.04	0.25	

Avearge Is$_{(50)}$ = 0.34 MPa

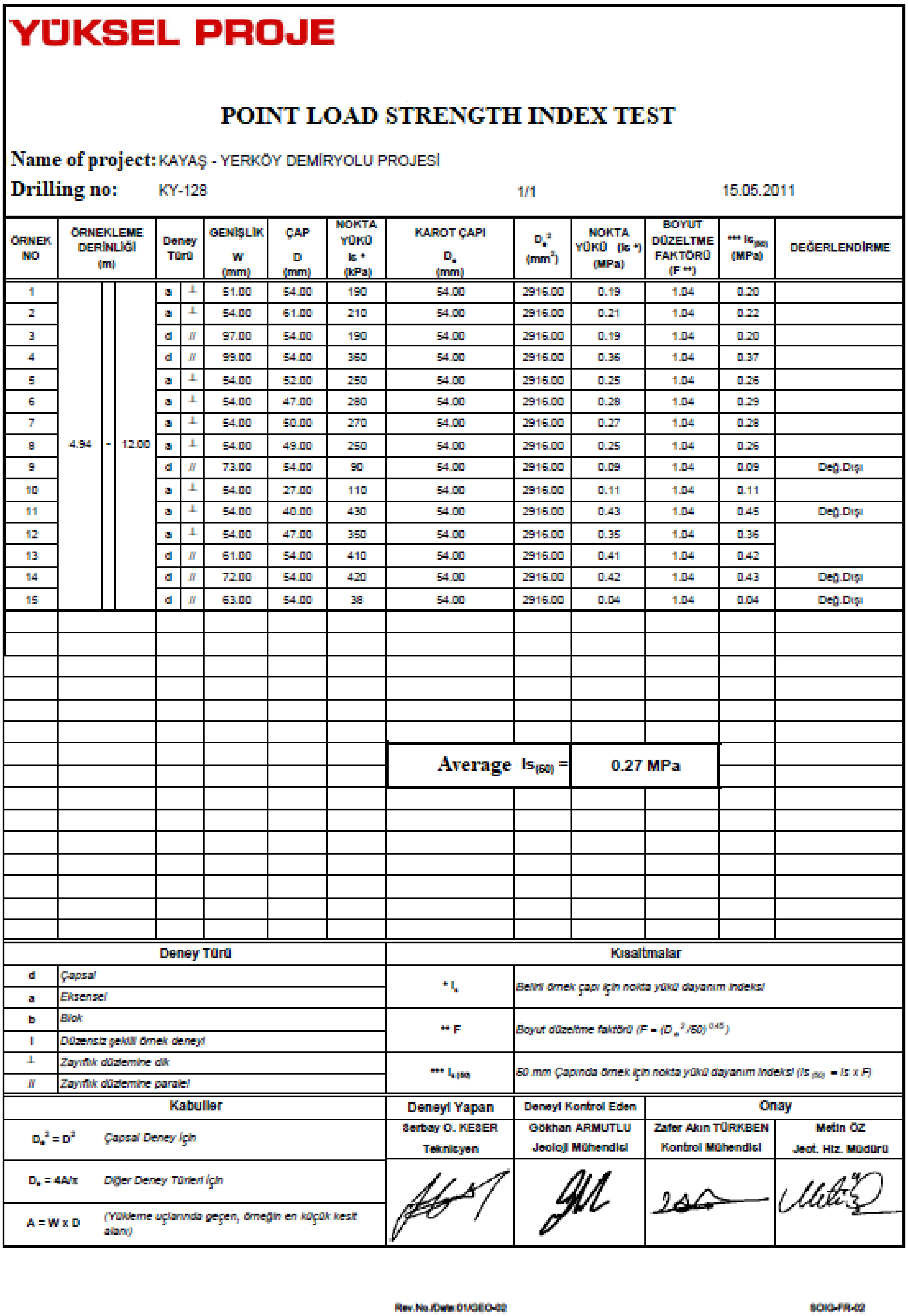
Deney Türü	
d	*Çapsal*
a	*Eksensel*
b	*Blok*
I	*Düzensiz şekilli örnek deneyi*
⊥	*Zayıflık düzlemine dik*
//	*Zayıflık düzlemine paralel*

Kısaltmalar	
* I$_s$	*Belirli örnek çapı için nokta yükü dayanım indeksi*
** F	*Boyut düzeltme faktörü ($F = (D_e^2/50)^{0.45}$)*
*** I$_{s(50)}$	*50 mm Çapında örnek için nokta yükü dayanım indeksi ($Is_{(50)} = Is \times F$)*

Kabuller	
$D_e^2 = D^2$	*Çapsal Deney İçin*
$D_e = 4A/\pi$	*Diğer Deney Türleri İçin*
$A = W \times D$	*(Yükleme uçlarında geçen, örneğin en küçük kesit alanı)*

Deneyi Yapan	Onay	
Gökhan ARMUTLU Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeot. Hiz. Müdürü

Rev.No./Date:01/GEO-02 SOIG-FR-02



YÜKSEL PROJE

POINT LOAD STRENGTH INDEX TEST

Name of project: KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

Drilling no: KY-128 1/1 15.05.2011

ÖRNEK NO	ÖRNEKLEME DERİNLİĞİ (m)	Deney Türü		GENİŞLİK W (mm)	ÇAP D (mm)	NOKTA YÜKÜ Is * (kPa)	KAROT ÇAPI D_e (mm)	D_e^2 (mm^2)	NOKTA YÜKÜ (Is *) (MPa)	BOYUT DÜZELTME FAKTÖRÜ (F **)	*** $Is_{(50)}$ (MPa)	DEĞERLENDİRME
1	4.94 - 12.00	a	⊥	51.00	54.00	190	54.00	2916.00	0.19	1.04	0.20	
2		a	⊥	54.00	61.00	210	54.00	2916.00	0.21	1.04	0.22	
3		d	//	97.00	54.00	190	54.00	2916.00	0.19	1.04	0.20	
4		d	//	99.00	54.00	360	54.00	2916.00	0.36	1.04	0.37	
5		a	⊥	54.00	52.00	250	54.00	2916.00	0.25	1.04	0.26	
6		a	⊥	54.00	47.00	280	54.00	2916.00	0.28	1.04	0.29	
7		a	⊥	54.00	50.00	270	54.00	2916.00	0.27	1.04	0.28	
8		a	⊥	54.00	49.00	250	54.00	2916.00	0.25	1.04	0.26	
9		d	//	73.00	54.00	90	54.00	2916.00	0.09	1.04	0.09	Değ.Dışı
10		a	⊥	54.00	27.00	110	54.00	2916.00	0.11	1.04	0.11	
11		a	⊥	54.00	40.00	430	54.00	2916.00	0.43	1.04	0.45	Değ.Dışı
12		a	⊥	54.00	47.00	350	54.00	2916.00	0.35	1.04	0.36	
13		d	//	61.00	54.00	410	54.00	2916.00	0.41	1.04	0.42	
14		d	//	72.00	54.00	420	54.00	2916.00	0.42	1.04	0.43	Değ.Dışı
15		d	//	63.00	54.00	38	54.00	2916.00	0.04	1.04	0.04	Değ.Dışı

Average $Is_{(50)}$ = 0.27 MPa

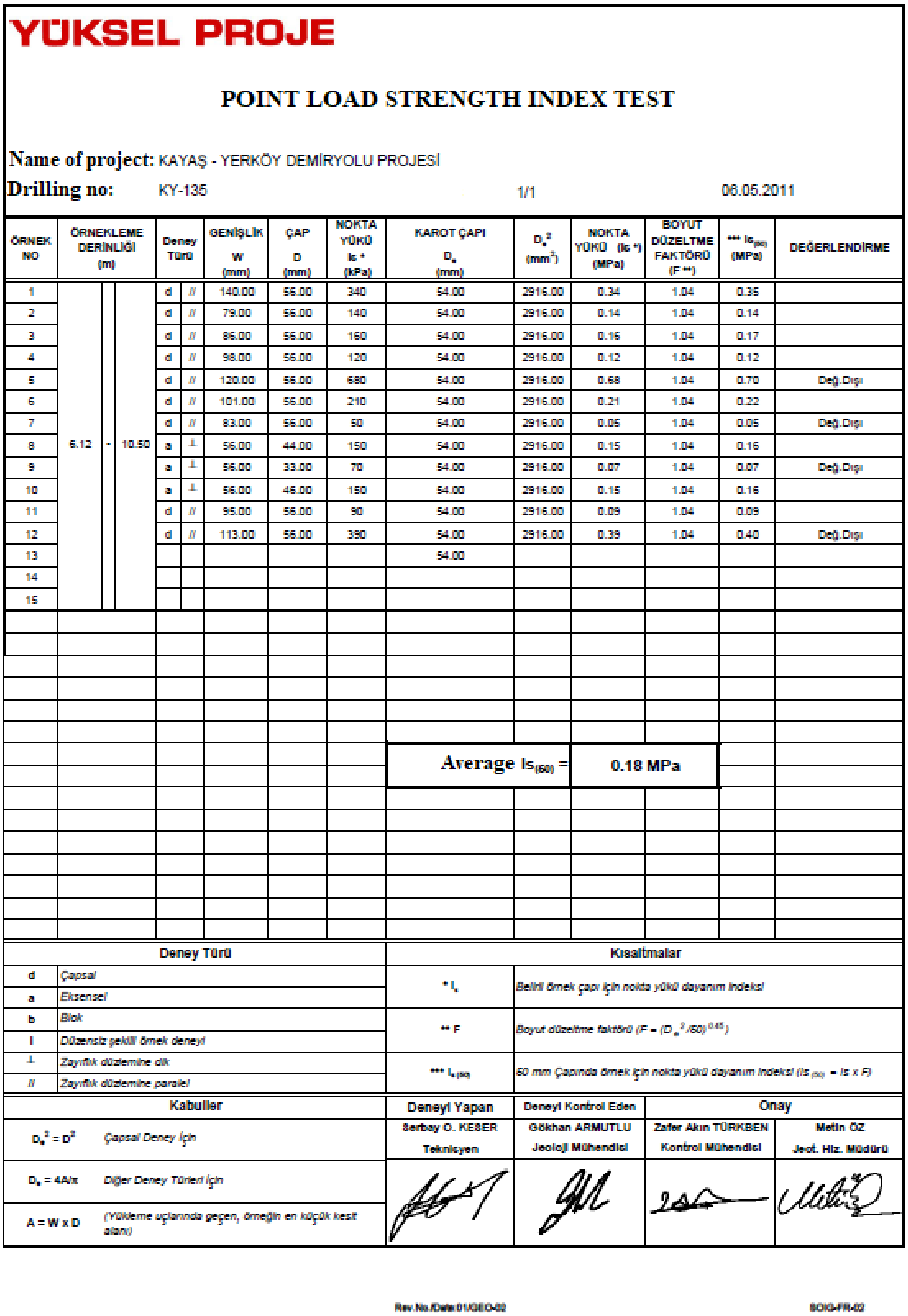
Deney Türü	
d	*Çapsal*
a	*Eksensel*
b	*Blok*
I	*Düzensiz şekilli örnek deneyi*
⊥	*Zayıflık düzlemine dik*
//	*Zayıflık düzlemine paralel*

Kısaltmalar	
* I_s	*Belirli örnek çapı için nokta yükü dayanım indeksi*
** F	*Boyut düzeltme faktörü* ($F = (D_e^2/50)^{0.45}$)
*** $I_{s(50)}$	*50 mm Çapında örnek için nokta yükü dayanım indeksi* ($Is_{(50)} = Is \times F$)

Kabuller	
$D_e^2 = D^2$	*Çapsal Deney İçin*
$D_e = 4A/\pi$	*Diğer Deney Türleri İçin*
$A = W \times D$	*(Yükleme uçlarında geçen, örneğin en küçük kesit alanı)*

Deneyi Yapan	Deneyi Kontrol Eden	Onay	
Serbay O. KESER Teknisyen	Gökhan ARMUTLU Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeot. Hiz. Müdürü

Rev.No./Date:01/GEO-02 SOKG-FR-02



YÜKSEL PROJE

POINT LOAD STRENGTH INDEX TEST

Name of project: KAYAŞ - YERKÖY DEMİRYOLU PROJESİ

Drilling no: KY-135 1/1 06.05.2011

ÖRNEK NO	ÖRNEKLEME DERİNLİĞİ (m)			Deney Türü		GENİŞLİK W (mm)	ÇAP D (mm)	NOKTA YÜKÜ Is * (kPa)	KAROT ÇAPI D_e (mm)	D_e^2 (mm²)	NOKTA YÜKÜ (Is *) (MPa)	BOYUT DÜZELTME FAKTÖRÜ (F **)	*** $Is_{(50)}$ (MPa)	DEĞERLENDİRME
1	6.12	-	10.50	d	//	140.00	56.00	340	54.00	2916.00	0.34	1.04	0.35	
2				d	//	79.00	56.00	140	54.00	2916.00	0.14	1.04	0.14	
3				d	//	86.00	56.00	160	54.00	2916.00	0.16	1.04	0.17	
4				d	//	98.00	56.00	120	54.00	2916.00	0.12	1.04	0.12	
5				d	//	120.00	56.00	680	54.00	2916.00	0.68	1.04	0.70	Değ.Dışı
6				d	//	101.00	56.00	210	54.00	2916.00	0.21	1.04	0.22	
7				d	//	83.00	56.00	50	54.00	2916.00	0.05	1.04	0.05	Değ.Dışı
8				a	⊥	56.00	44.00	150	54.00	2916.00	0.15	1.04	0.16	
9				a	⊥	56.00	33.00	70	54.00	2916.00	0.07	1.04	0.07	Değ.Dışı
10				a	⊥	56.00	46.00	150	54.00	2916.00	0.15	1.04	0.16	
11				d	//	95.00	56.00	90	54.00	2916.00	0.09	1.04	0.09	
12				d	//	113.00	56.00	390	54.00	2916.00	0.39	1.04	0.40	Değ.Dışı
13									54.00					
14														
15														

Average $Is_{(50)}$ = 0.18 MPa

Deney Türü	
d	*Çapsal*
a	*Eksensel*
b	*Blok*
I	*Düzensiz şekilli örnek deneyi*
⊥	*Zayıflık düzlemine dik*
//	*Zayıflık düzlemine paralel*

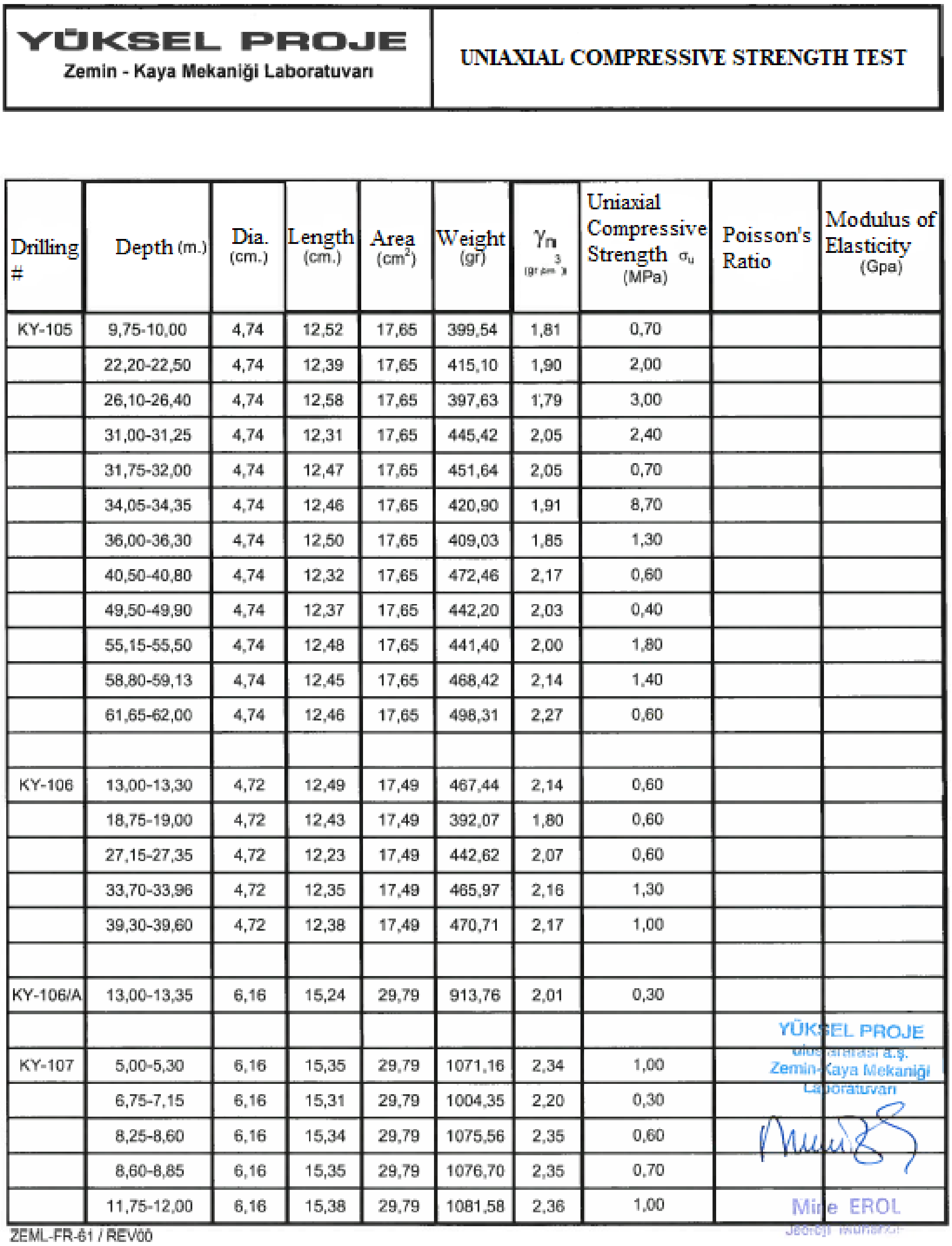
Kısaltmalar	
* I_s	*Belirli örnek çapı için nokta yükü dayanım indeksi*
** F	*Boyut düzeltme faktörü* ($F = (D_e^2/50)^{0.45}$)
*** $I_{s(50)}$	*50 mm Çapında örnek için nokta yükü dayanım indeksi* ($Is_{(50)} = Is \times F$)

Kabuller	
$D_e^2 = D^2$	*Çapsal Deney İçin*
$D_e = 4A/\pi$	*Diğer Deney Türleri İçin*
$A = W \times D$	*(Yükleme uçlarında geçen, örneğin en küçük kesit alanı)*

Deneyi Yapan	Deneyi Kontrol Eden	Onay	
Serbay O. KESER Teknisyen	Gökhan ARMUTLU Jeoloji Mühendisi	Zafer Akın TÜRKBEN Kontrol Mühendisi	Metin ÖZ Jeot. Hiz. Müdürü

Rev.No./Date:01/GEO-02 SOIG-FR-02

G. UNIAXIAL COMPRESSIVE TEST DATA ON EXPLORATORY DRILLING SAMPLES



YÜKSEL PROJE
Zemin - Kaya Mekaniği Laboratuvarı

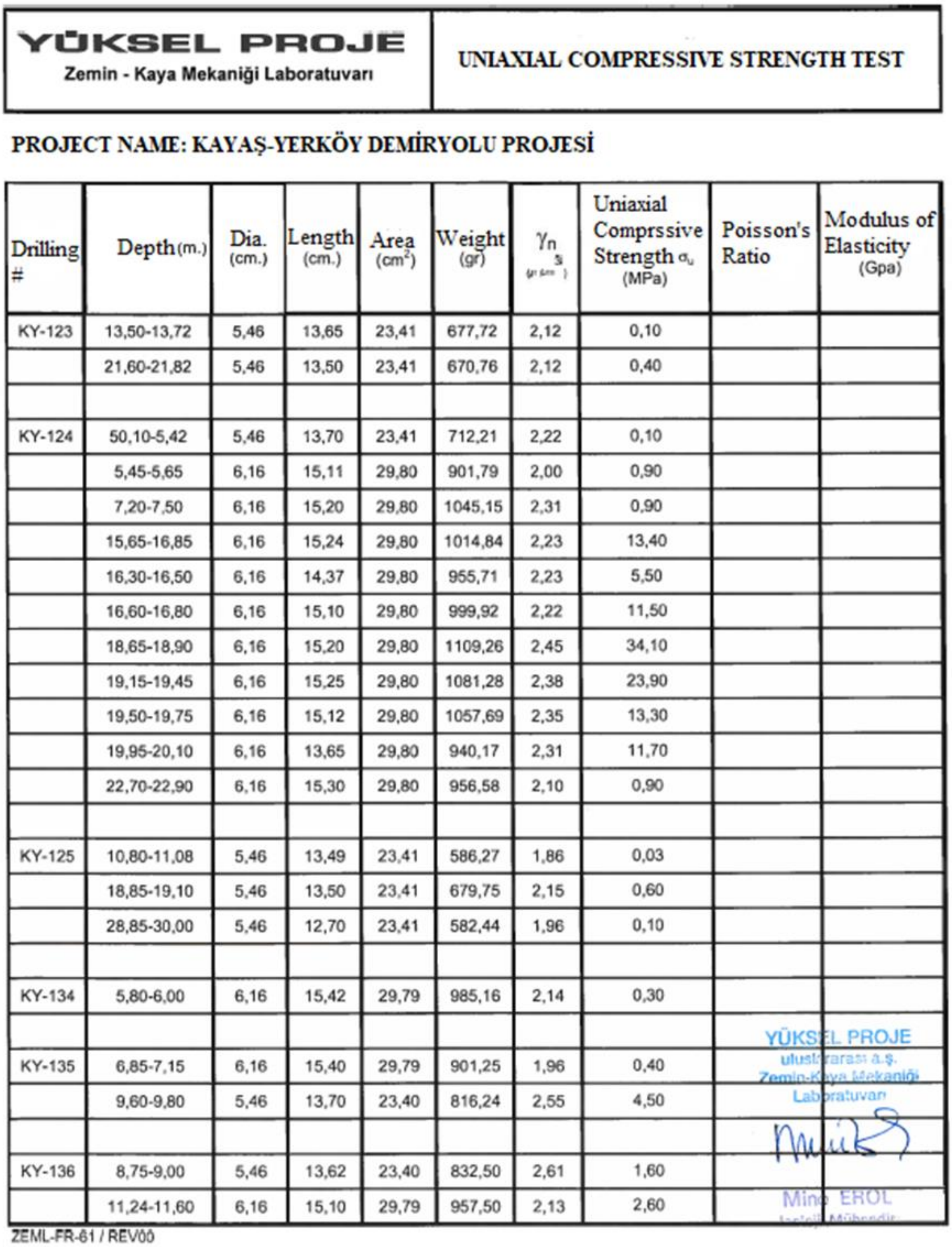
UNIAXIAL COMPRESSIVE STRENGTH TEST

Drilling #	Depth (m.)	Dia. (cm.)	Length (cm.)	Area (cm^2)	Weight (gr)	γ_n (gr/cm^3)	Uniaxial Compressive Strength σ_u (MPa)	Poisson's Ratio	Modulus of Elasticity (Gpa)
KY-105	9,75-10,00	4,74	12,52	17,65	399,54	1,81	0,70		
	22,20-22,50	4,74	12,39	17,65	415,10	1,90	2,00		
	26,10-26,40	4,74	12,58	17,65	397,63	1,79	3,00		
	31,00-31,25	4,74	12,31	17,65	445,42	2,05	2,40		
	31,75-32,00	4,74	12,47	17,65	451,64	2,05	0,70		
	34,05-34,35	4,74	12,46	17,65	420,90	1,91	8,70		
	36,00-36,30	4,74	12,50	17,65	409,03	1,85	1,30		
	40,50-40,80	4,74	12,32	17,65	472,46	2,17	0,60		
	49,50-49,90	4,74	12,37	17,65	442,20	2,03	0,40		
	55,15-55,50	4,74	12,48	17,65	441,40	2,00	1,80		
	58,80-59,13	4,74	12,45	17,65	468,42	2,14	1,40		
	61,65-62,00	4,74	12,46	17,65	498,31	2,27	0,60		
KY-106	13,00-13,30	4,72	12,49	17,49	467,44	2,14	0,60		
	18,75-19,00	4,72	12,43	17,49	392,07	1,80	0,60		
	27,15-27,35	4,72	12,23	17,49	442,62	2,07	0,60		
	33,70-33,96	4,72	12,35	17,49	465,97	2,16	1,30		
	39,30-39,60	4,72	12,38	17,49	470,71	2,17	1,00		
KY-106/A	13,00-13,35	6,16	15,24	29,79	913,76	2,01	0,30		
KY-107	5,00-5,30	6,16	15,35	29,79	1071,16	2,34	1,00		
	6,75-7,15	6,16	15,31	29,79	1004,35	2,20	0,30		
	8,25-8,60	6,16	15,34	29,79	1075,56	2,35	0,60		
	8,60-8,85	6,16	15,35	29,79	1076,70	2,35	0,70		
	11,75-12,00	6,16	15,38	29,79	1081,58	2,36	1,00		

ZEML-FR-61 / REV00

YÜKSEL PROJE
Uluslararası A.Ş.
Zemin-Kaya Mekaniği
Laboratuvarı

Mine EROL
Jeoloji Mühendisi



YÜKSEL PROJE
Zemin - Kaya Mekaniği Laboratuvarı

UNIAXIAL COMPRESSIVE STRENGTH TEST

PROJECT NAME: KAYAŞ-YERKÖY DEMİRYOLU PROJESİ

Drilling #	Depth (m.)	Dia. (cm.)	Length (cm.)	Area (cm^2)	Weight (gr)	γ_n (gr/cm^3)	Uniaxial Comprssive Strength σ_u (MPa)	Poisson's Ratio	Modulus of Elasticity (Gpa)
KY-123	13,50-13,72	5,46	13,65	23,41	677,72	2,12	0,10		
	21,60-21,82	5,46	13,50	23,41	670,76	2,12	0,40		
KY-124	50,10-5,42	5,46	13,70	23,41	712,21	2,22	0,10		
	5,45-5,65	6,16	15,11	29,80	901,79	2,00	0,90		
	7,20-7,50	6,16	15,20	29,80	1045,15	2,31	0,90		
	15,65-16,85	6,16	15,24	29,80	1014,84	2,23	13,40		
	16,30-16,50	6,16	14,37	29,80	955,71	2,23	5,50		
	16,60-16,80	6,16	15,10	29,80	999,92	2,22	11,50		
	18,65-18,90	6,16	15,20	29,80	1109,26	2,45	34,10		
	19,15-19,45	6,16	15,25	29,80	1081,28	2,38	23,90		
	19,50-19,75	6,16	15,12	29,80	1057,69	2,35	13,30		
	19,95-20,10	6,16	13,65	29,80	940,17	2,31	11,70		
	22,70-22,90	6,16	15,30	29,80	956,58	2,10	0,90		
KY-125	10,80-11,08	5,46	13,49	23,41	586,27	1,86	0,03		
	18,85-19,10	5,46	13,50	23,41	679,75	2,15	0,60		
	28,85-30,00	5,46	12,70	23,41	582,44	1,96	0,10		
KY-134	5,80-6,00	6,16	15,42	29,79	985,16	2,14	0,30		
KY-135	6,85-7,15	6,16	15,40	29,79	901,25	1,96	0,40		
	9,60-9,80	5,46	13,70	23,40	816,24	2,55	4,50		
KY-136	8,75-9,00	5,46	13,62	23,40	832,50	2,61	1,60		
	11,24-11,60	6,16	15,10	29,79	957,50	2,13	2,60		

YÜKSEL PROJE
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Zemin-Kaya Mekaniği
Laboratuvarı

Mine EROL
Jeoloji Mühendisi

ZEML-FR-61 / REV00