TWO CASE STUDIES RELATED TO THE LANDSLIDE REMEDIAL DESIGNS

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SYNOPSIS

In this paper, case studies are presented investigating two landslides took place at different sections of Gerede-Ankara and Ankara Peripheral Motorway due to the presence of high landslide-potential conditions. Firstly, the data obtained from the surface geology studies, borings, laboratory and monitoring works are summarized. Then the geotechnical profiles and landslide mechanism are determined making use of this information. Soil strength parameters controlling the stability are found by utilizing back calculation method on the failure modes developed. Cases are presented using these parameters within the landslide remedial design project. The importance of the instrumentation and monitoring including surface benchmarks, inclinometers and piezometers on the optimization of the remedial design is also emphasized.

1. INTRODUCTION

At different sections of the Gerede-Ankara and Ankara Peripheral Motorway which is presently under construction, it has been obligatory to pass from the regions with high landslide potential. Subsoil investigations, design and instrumental monitoring works in the scope of the remedial design projects of such two landslides took place are within the content of this article. The motorway is being constructed by the Enka-Bechtel J.V. and the Engineer is the KSWK in the name of the Highway General

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Directorate (KGM). The subject remedial design projects of these landslides in this article are prepared by Zetaş Earth Technology Corp. under the supervision of the senior author.

The first landslide named as Buğralar Failure covered in the article took place at the east slope of the viaduct located at Km 63+400 within the Gerede-Ankara section of the motorway, in May 1990. Between May 1990 and July 1992 and later, the design and the construction works have been continued. The second landslide took place on a cut section near Km 154 of the Peripheral Motorway in terms of a sudden failure. For this landslide, although the subsoil investigations, design and the monitoring works which are essential for the design have been completed, construction and the monitoring works during the construction works are still being continued.

2. GEREDE-ANKARA AND ANKARA PERIPHERAL MOTORWAY Km 63+400 BUĞRALAR LANDSLIDE

2.1. Landslide Description

A large scale landslide took place on the east slope of the valley on the motorway alignment following an excavation for the Bugralar Bridge construction in May 1990 at Km 63+400 of Gerede-Ankara and Ankara Peripheral Motorway. Although a major part of the slipped material is removed, subdrains and a rock buttress at the lower elevation are constructed, the landslide progressed further to the upper elevations and finally leaned to the rock buttress as of July 1992. Therefore, additional measures had to be taken to stabilize the landslide. These measures are discussed within the scope of this paper.

2.2. Subsoil Conditions and the Mechanism of the Movement

The subsoil conditions are determined by the site and subsoil investigations. At the landslide location, there is colluvium on top, claystone (CLS) at the lower elevations of the slide area At the higher elevations tuff-claystone (TCL) and below, claystone exist as the formations underlying the top colluvium (Qc) as could be seen in Figure 1. The thickness of the colluvium is less at the upper elevations. The slide is known to be along the colluvium/claystone interface (slip surface #a) at the lower elevations. However, slides may take place along the colluvium/tuff-claystone (slip surface #b) or tuff-claystone/claystone interfaces (slip surface #c) at the upper elevations. As a result, separate stability analyses are performed for these three different slip surfaces at each section. Circular slide surfaces are

FIGURE 1 KM 63+400 BUGRALAR LANDSLIDE - GEOTECHNICAL CROSS-SECTION

SECTION (1)

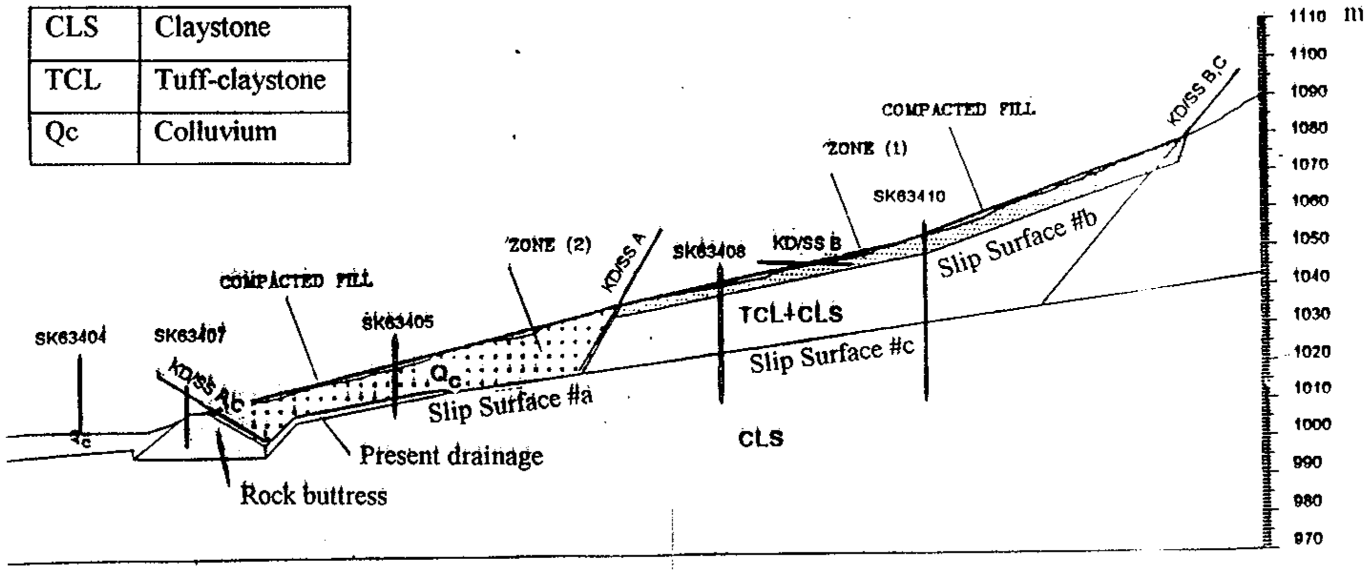


TABLE II. STABILITY ANALYSES SUMMARY TABLE - JULY 1992

SECTION	SAFETY FACTOR			NOTES
	#a	#b	#c	
1	0.89	0.75	1.33	landslide
2	1.43	0.82	1.49	landslide
3	1.12	0.92	1.58	landslide
1			1.34	After excavation 1&2
1			1.68/1.75*	After the first remedial design
2			1.92/2.02*	After the first remedial design
3			2.11/2.19*	After the first remedial design
1			0.94	EQ (0.2g)
1	Circular min. FS=2.36			After the first remedial design
1	Circular min. FS=1.27 (0.2g)			After the first remedial design

* Analysis with $\phi'=35°$ and $c'=0$ for the fill.

investigated in the computer utilizing "Modified Bishop" methodology. For the investigation of the noncircular slide surfaces by computer, "Janbu" methodology is adopted. The ground water level determined by the monitoring wells, subsoil conditions obtained by the borings and the effect of the already constructed subdrains are incorporated in the analyses. The average geotechnical parameters for the formations are summarized in Table I. Skempton (1964) suggested that the shear strength can be determined by the peak friction angle, ϕ' prior to the landslide, and residual friction angle ϕ_r' after the landslide took place. Back calculation analysis for the landslide of May 1990 resulted in a friction angle of $\phi_r'=14°$ along the colluvium/claystone interface. The residual cohesion value, c_r' is taken as equal to zero.

TABLE I - GEOTECHNICAL PARAMETERS

Material	No	c_r' (kPA)	ϕ_r' (degree)	γ_n (kN/m^3)
Colluvium	-	0	14	19
Tuff-claystone	-	25	25	19
Claystone	6	50	25	19
Tuff-claystone/claystone	3	0	11	19
Compacted Earth Fill	1,2	25	25	20
Rock Fill	5	0	42	20
Uncompacted Rock Fill	4	0	40	20

2.3. Stability Analyses and the Proposed Remedial Design

Stability analyses are performed for the observed case as of July 1992, and the case after the implementation of the recommended remedial design (ZETAŞ, 1992A). Table 2 summarizes the results of the stability analyses for both cases and for three possible slide surfaces in the three different sections.

Stability analyses for the observed case as of July 1992, revealed that (see Figure 1) the Section 1 is the most critical one and the lower colluvial material slipped into the previously excavated area down to the bedrock elevations may further slip even over the rock buttress (slip surface #a) (FS=0.89). The analysis at three sections for the upper colluvium (slip surface #b) illustrate that the stability of the upper colluvium is also critical (FS=0.75-0.92) and the slide may progress to the upper elevations. A deeper instability along the tuff-claystone/claystone interface (slip surface #c) seems to be less probable (FS=1.33-1.58) compared to other shallower slip surfaces.

As of July 1992, landslide hindered the safety of the already completed constructions for the purpose of stabilizing the area and might have progressed further to the upper elevations. As a result, stability of the slide area should be secured by implementation of the appropriate remedial measures.

Recommended remedial works are, the removal of the lower parts of the landslide area defined with the area slipped in May 1990 up to the elevations where the colluvium gets thinner and tuff-claystone is encountered, down to the claystone bedrock (CLS) or to the previously constructed drainage layers. In addition, the slide area at the upper elevations are recommended to be excavated minimum 5.0 m or down to the tuff-claystone (TCL) for the purpose of removing the colluvial material. Finally, the complete excavated area should be filled with compacted material to the natural topographical levels. Additional subdrains are recommended in the lower excavated area before filling. The proposed construction scheme is summarized below:

- Excavate 5.0 or down to TCL at upper elevations (Zone 1, see Figure 1)
- Excavate down to bedrock CLS at lower elevations (Zone 2). Keep continuous record of probable movements with movement benchmarks installed at Zone 1.
- Construct subdrains.
- Fill the excavated parts with compacted material in a short period.
- Construct surface drains.

Stability analysis for the geometry after the excavation of the lower area (Zone 2) gives a factor of safety of FS=1.34 for the slip surface #c. The analysis for the observed case as of July 1992 (before the excavation) yield FS=1.33 for the same slip surface. As a result analysis show that the excavation of Zone 2 does not make the stability worse than the present stability. Nevertheless, it is recommended to take movement records of the upper TCL during lower excavation (Zone 2) and during filling operations with movement benchmarks and carry out the remedial works with care.
The stability analysis yielded FS=1.68-2.11 under static and FS=0.94 under earthquake (0.2g) conditions which is satisfactory considering the conservatism in the method of analysis and assumptions,

after the completion of the recommended remedial works (for slip surface#c). Other slip surfaces will not be critical since all the material is replaced with compacted fill. In addition, the circle giving the minimum factor of safety is searched with simplex optimization technique and FS=2.36 for static and FS=1.27 for earthquake (0.2g) conditions are determined.

Comparison of the factor of safety values for the observed case and after the remedial works illustrates that stability condition is significantly improved with recommended remedial works and the stability of the final condition is sufficient. Subsequently, the implementation of the design works are initiated.

At this stage, during the progress of the excavation works and construction of the drainage system, it is proposed to keep the temporary rock buttress between Zone 1 and Zone 2 and also to avoid some portion of the Zone 1 backfilling by the Engineer. Additionally, it is found essential in terms of stability to construct another buttress in front of the present one.

As the last stage of the design works with consideration of the construction monitoring data, stability analyses are performed for seven (7) different backfilling configurations for both static and earthquake (0.2g) conditions. The geotechnical parameters and the numbers assigned to formations in the analyses are given in Table 1. BUGR C, BUGR E and BUGR F types provided satisfactory safety factors (FS) in term of stability (see Figure 2). BUGR F type was selected to be applied in the final design since this configuration required the least amount of earthfill material. Table III summarizes the stability analysis results for different slip surfaces and backfilling configurations. The design is implemented in the site and it has been determined by instrumental monitoring that the stability is achieved.

3. GEREDE-ANKARA AND ANKARA PERIPHERAL MOTORWAY Km 154+650 LANDSLIDE

3.1. Landslide Description

A 300 meter-long section of the motorway in deep cut was completed in June 1993 within Gerede-Ankara and Ankara Peripheral Motorway. A sudden landslide on the 30 meter-high left slope of the cut area took place on July 19, 1993. The landslide movement was occurred following the final excavation reaching the motorway platform level. The slip surface reached up to the 5000 m^3-ASKI water tank

FIGURE 2. KM 63+400 LANDSLIDE - GEOTECHNICAL CROSS-SECTION
FINAL CASE

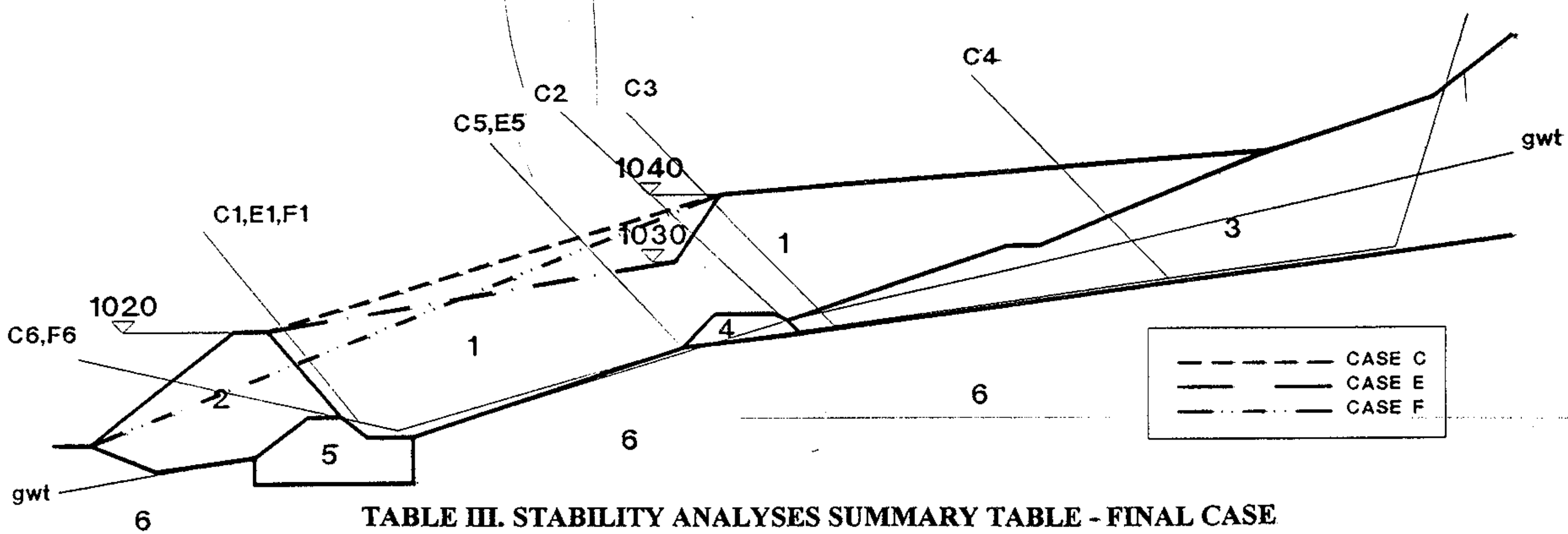


TABLE III. STABILITY ANALYSES SUMMARY TABLE - FINAL CASE

Slip surface no	Static/EQ	BUGR Case C		BUGR Case E		BUGR Case F	
		Analysis	FS	Analysis	FS	Analysis	FS
1	Static	BUGR1	3.0			BUGR1	2.2
	EQ	BUGR1	1.3	BUGR1E	1.1	BUGR1	1.3
2	Static	BUGR2	2.4				
	EQ	BUGR2	1.0				
3	Static	BUGR3	2.2				
	EQ	BUGR3	1.0				
4	Static	BUGR4	1.5				
	EQ						
5	Static	BUGR5	2.7				
	EQ	BUGR5	1.1	BUGR5E	0.9		
6	Static	BUGR6	2.8			BUGR6	2.3
	EQ	BUGR6	1.2			BUGR6	1.0

which is located 300 meters behind the toe of the excavation slope. Therefore, the landslide also hindered the safety of the ASKI water tank. Different formations of cracks indicated the presence of secondary movements within the sliding mass.

3.2. Subsoil Conditions and the Mechanism of the Movement

Subsoil conditions were further determined by a detailed subsoil investigation program after the initiation of the landslide, including borings, inclinometers and the installation of the PVC standpipe and surface movement benchmarks. The main formation of the landslide area is determined as andesitic agglomerate. Within this formation clayey zones were encountered. The ground water elevation found during the borings was high and it raised to the ground level at the first berm. Therefore in order to monitor the ground water table, it is determined to install piezometers at proper locations.

Back calculation analysis performed for the selected most probable slip surface resulted in an interface friction angle of $\phi_r'=9°$ considering that the exit part of the slide surface in andesitic agglomerate has an internal friction angle of $\phi'=30°$. On the other hand, if the exit part was considered to have the same interface friction properties, then the interface friction angle for the whole surface comes out to be $\phi_r'=11°$. Table IV summarizes the geotechnical parameters used in the analyses.

TABLE IV. GEOTECHNICAL PARAMETERS

Explanation	c_r' (kPa)	ϕ' (degree)	γ_n (kN/m^3)
Andesitic agglomerate	0	30	19
Andesitic agglomerate/Claystone interface	0	9	-
Andesitic agglomerate/Clay interface	0	9	-

Instrumental monitoring works during the investigation and design stages consisted of inclinometers and surface benchmarks. The inclinometers are utilized to determine the depth of the sliding surface. For instance, the readings with time obtained from INC1 (see Figure 3), indicate that there is a movement at the elevation of 929 m as could be seen in Figure 4. Surface benchmark readings supported the inclinometers. Graphs developed from the surface benchmarks indicated the progress of the landslide in

FIGURE 3. KM 154+650 LANDSLIDE - GEOTECHNICAL CROSS-SECTION

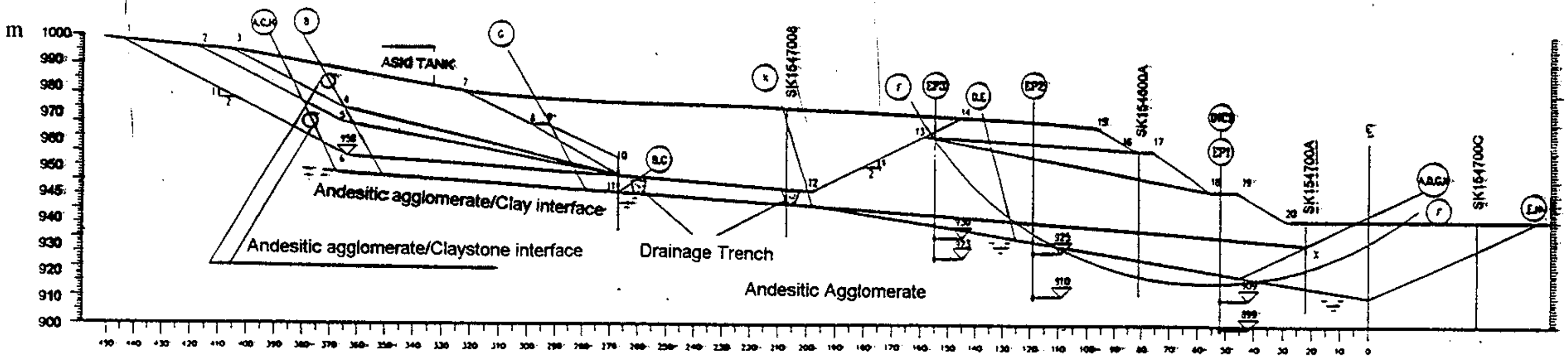
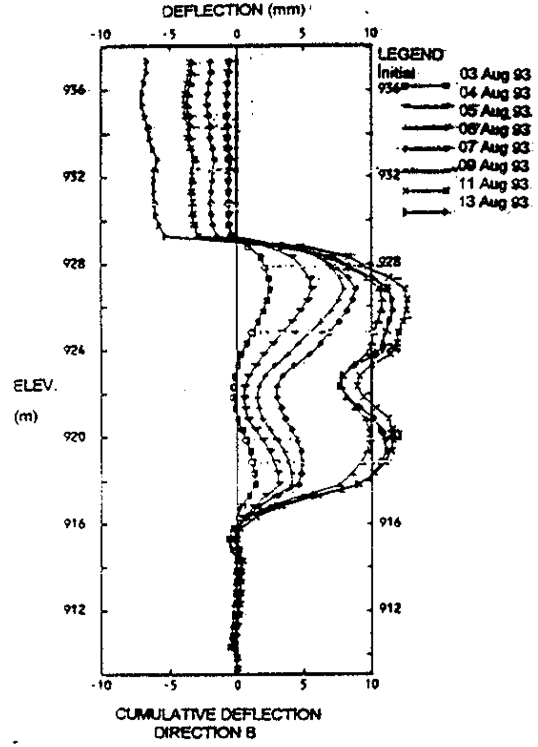
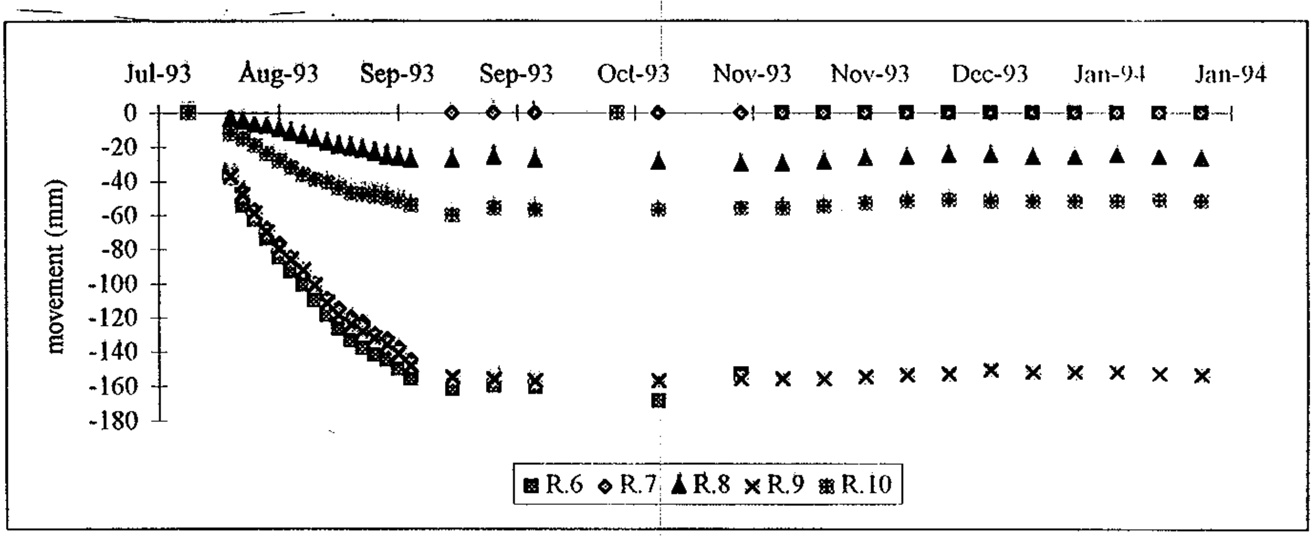


FIGURE 4. INCLINOMETER 1 READINGS



time. As a short term measure based on the first available data it was proposed to perform some initial excavation at the top part of the landslide area in October 1993. A typical example is given in Figure 5. The surface benchmarks showed the tendency of slide to slowdown and stop after the excavation proposed in October 1993.

FIGURE 5. SURFACE BENCHMARK READINGS



3.3. Stability Analyses and the Proposed Remedial Design

Numerous stability analyses are performed for the present topography and for various regrading excavation configurations (ZETAŞ, 1994). Table V summarizes the analyses performed. The excavation geometry for each case is illustrated in Figure 3.

Back calculation analysis for the present case (defined by the points 1-2-3-7-14-15-16-17-18-19-20) to achieve a factor of safety of unity (FS=1.0) for surface A-A yields $\phi=9°$ (K154A) for the andesitic agglomerate/clay-claystone interface under present high ground water conditions determined by the borings. Analysis through A-A for the present topography but for completely drained condition yield FS=1.547 (K154DRY) under static conditions and FS=0.966 (K154DR1) for earthquake coefficients of k=0.1 (Çetinkaya, Durgunoğlu et al., 1993). This means that, if complete drainage of the landslide is somewhat achieved, the present geometry (natural) will satisfy the static factor of safety and will be very close to earthquake factor of safety without any excavation. Analyses for the other slip surfaces yielded factor of safety values close to unity meaning that intermediate smaller size slides are also probable. Therefore, for such intermediate slide surfaces (partial slide mobilization) a somewhat less friction value is expected to be applicable within the andesitic agglomerate. A separate back-calculation is carried out to find the averaged friction angle value along the complete surface. This analysis (K154PN3) yielded average $\phi_r'=15°$ and some analyses are performed with this averaged friction value.

TABLE V. KM 154+650 LANDSLIDE STABILITY ANALYSES

	EXCAVATION	SLIP	GWT	Factor of Safety							
				STATIC	Run no.	k=0.075	Run no.	k=0.100	Run no.	k=0.200	Run no.
I	1-2-3-7-14-15-16-17-18-19-20	A-A	Present	1.018	K154A						
	PRESENT CASE	A-A	DRY	1.547	K154DRY			0.966	K154DR1	0.739	K154DRE
		D-D	Present	1.159	K154PN1						
		D-x-D*	Present	1.048	K154PN3						
		E-E	Present	1.269	K154PN2						
		F-F	nr=0.15	1.030	K154PNC						
		F-F	nr=0.05	1.180	K154PNC1						
II	1-7-8-9-10-11-12-13-16-17-18-										
	PILED WALL	C-C	Case I	1.332	K154T1S			0.981	K154T1L	0.764	K154T1E
	425,000m3										
III	1-7-11-12-13-16-17-18-19-20	A-A	Case I	1.510	K154XS	1.042	K154X1	0.953	K154X		
	430,000m3	A-A	Case III	1.442	K154XS1						
		A-A	Case II	1.322	K154XS2						
		C-C	Case I	0.913	K154XTS						
		C-C	Case I	1.099	K154XTZ						
IV	1-2-5-11-12-13-16-17-18-19-	A-A	Case I					1.062	K154M		
	635,000m3	B-B	Case I	1.579	K154MSX						
V	1-3-4-11-12-13-16-17-18-19-	A-x-A	Case I	1.394	K154LZS	0.908	K154LZ1	0.818	K154LZ		
	600,000m3	A-A	Case I	1.701	K154LS			1.030	K154L		
			Case III					0.981	K154L1		
			Case II					0.893	K154L2		
		B-B	Case I	1.255	K154LSX						
		H-H	Case I					1.111	K154LW1		
		D-D	Case I	2.102	K154LDS			1.451	K154LD		
		D-x-D*	Case I	1.754	K154LZX			1.117	K154LZW		
		D-x-D	Case I	1.518	K154LZY	1.106	K154LZM	1.018	K154LZD		
		E-E	Case I					1.487	K154LE		
		F-F	nr=0.05	1.360	K154RNC1	0.910	K154RNC2				
		F-F	DRY			0.970	K154RNC3				
		K-x-K	Case I	2.266	K154LZK			1.297	K154LZE		
VI	1-2-5-11-12-13-18-19-20	A-A	Case I							0.809	K154H1
	700,000m3	D-x-D*	Case I	1.859	K154HD						
VII	1-6-11-12-13-18-19-20	A-A	Case I							0.995	K154F1
	850,000m3	G-G	Case I	2.755	K154EGS					0.995	K154EGW
			Case II	2.277	K154ES1						
		D-D	Case I	2.264	K154GS2					1.154	K154S2E

CASE I : GWT dropped CASE II : GWT at CL at elev.930m CASE III : GWT at CL at elev.920m

* 15 degrees friction angle for whole slide surface

Although complete drainage of the area might satisfy the design factor of safety values, the size of the landslide (~30-35 m deep and ~300 m long) makes it almost impossible to intersect ground water with reasonable drainage measures (e.g. subhorizontal drains) for a complete drainage. Therefore, a deep excavation is considered to intersect the ground water level at the middle part of the landslide to reduce the driving-forces. Drainage of the upper end lower parts of this deep excavation is proposed to be done by 5.0m-deep drainage trenches.

Various regrading alternatives including the removal of ASKI tank or keeping it with a piled retaining structure satisfying the design safety factor values of the motorway are studied. Table V and Figure 3 present the analyzed regrading geometries, the excavation quantities for every alternative and the calculated factor of safety values for various slide surfaces under static and earthquake conditions. As a result of all analyses, the excavation geometry defined by the points 1-7-11-12-13-16-17-18-19-20 is selected for the final design.

In the design of remedial works for the purpose of monitoring the landslide during construction, a borehole with two electrical piezometers and another borehole with an inclinometer around ASKI tank are proposed and these monitoring devices are required to be installed prior to any further excavation work at the slide area. Therefore during the excavation works, stability of the system could have been observed continually and according to the results of the monitoring data, removal of the ASKI tank or keeping it with an additional retaining structure could be considered. The monitoring of the electrical piezometer data proposed for the construction stage is essential to assure that the drainage condition is achieved in accordance with the assumptions done for the stability analyses. In case, electrical piezometers show that the pore water pressures are higher than considered in the design, then additional drainage measures (e.g. subhorizontal drains, toe drainage) will be essential and could easily be adopted without any major difficulty. The remedial design is implemented in the site together with the monitoring scheme and it is been determined that the stability is achieved following the proposed excavation and construction of the drainage system.

4. CONCLUSIONS

It is common to face with various stability problems due to the unavoidable cuts in high landslide potential regions in the course of motorway construction. On these occasions, it is essential to model the movement and judge the failure mechanism correctly in order to achieve the design safety following the remediation. The two case studies are presented for this purpose emphasizing the role and the importance of the instrumentation and monitoring within the Gerede-Ankara and Ankara Peripheral Motorway project.

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