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BACK ANALYSIS TECHNIQUE FOR SLOPE STABILIZATION WORKS OF EMBANKMENT LANDSLIDE DUE TO FOUNDATION INSTABILITY

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ABSTRACT

This paper deals with the overall stability of an embankment foundation failure that lies at km 40+700 of the new Irbid-Amman Highway in Jordan. Slope stability back analysis was carried out for the slope to assess the conditions at time of failure, and estimate most representative shear strength parameters of foundation materials. Slope stability analysis was also carried out for proposed remedies.

Probabilistic seismic hazard analysis was carried out for the landslide site. Peak Ground Acceleration (PGA) value of 0.2 g was estimated for design. This corresponds to a 90% probability of non-exceedence in a 50 year design life of the highway. Pseudo-static slope stability analysis was also carried out.

The study concluded that the landslide movement occurred within the foundation colluvium material. It resulted primarily from the excessive load of the embankment and excess piezometric pressures generated within the slope.

The most feasible remedial measure to stabilize the landslide area was removal of existing failed embankment down to the top of sandstone layer, and reconstruction (using imported free-drainage rockfill) of a split level embankment together with the construction of surface and subsurface drainage system. These measures were successfully implemented in the field.

Key words: earthfill, earthquake, landslide, pore pressure, safety factor, slope stability (**IGC**: E6/H9)

INTRODUCTION

This study pertains to the geological and geotechnical study made for the area of a landslide at Station 40+700 along Irbid-Jerash-Amman Road which, presently, is under construction (Fig. 1). The study area is about 270 m in length extending from Grid N 181050 to N 181320 along the proposed road alignment (Fig. 2). Major embankment failure, however, necessitated interrupting the construction works of the new road until an appropriate solution for this landslide is found.

The recently proposed alignment of the road must pass through the study area in order to satisfy grade and curvature requirements. This study presents the results of the geotechnical investigation of the landslide zone and the proposed scheme of alignment routed through the landslide zone.

PURPOSE OF STUDY

In order to develop information to reinstate the road along the proposed alignment the investigation consisted of the following:—

1. Geotechnical and geological mapping of the vicinity of landslide area.
2. Characteristics of soil, rock and groundwater regime through a programme of rotary drilling with 8 Continuous sampling and laboratory testing.
3. Laboratory testing programme which was designed to measure undrained and drained strength, effective stress parameters of undisturbed and remoulded samples of typical soil deposits at the site, as well as classification and index tests. Unconfined compressive strength tests were performed on representative rock specimens.
4. Analysis of the field and laboratory data which was then interpreted to describe and evaluate the engineering characteristics of the soil, rock and groundwater regime.
5. Evaluation of stability conditions at the landslide area and presentation of recommendations for the stabilization of the sliding area and for the construction of a safe road.

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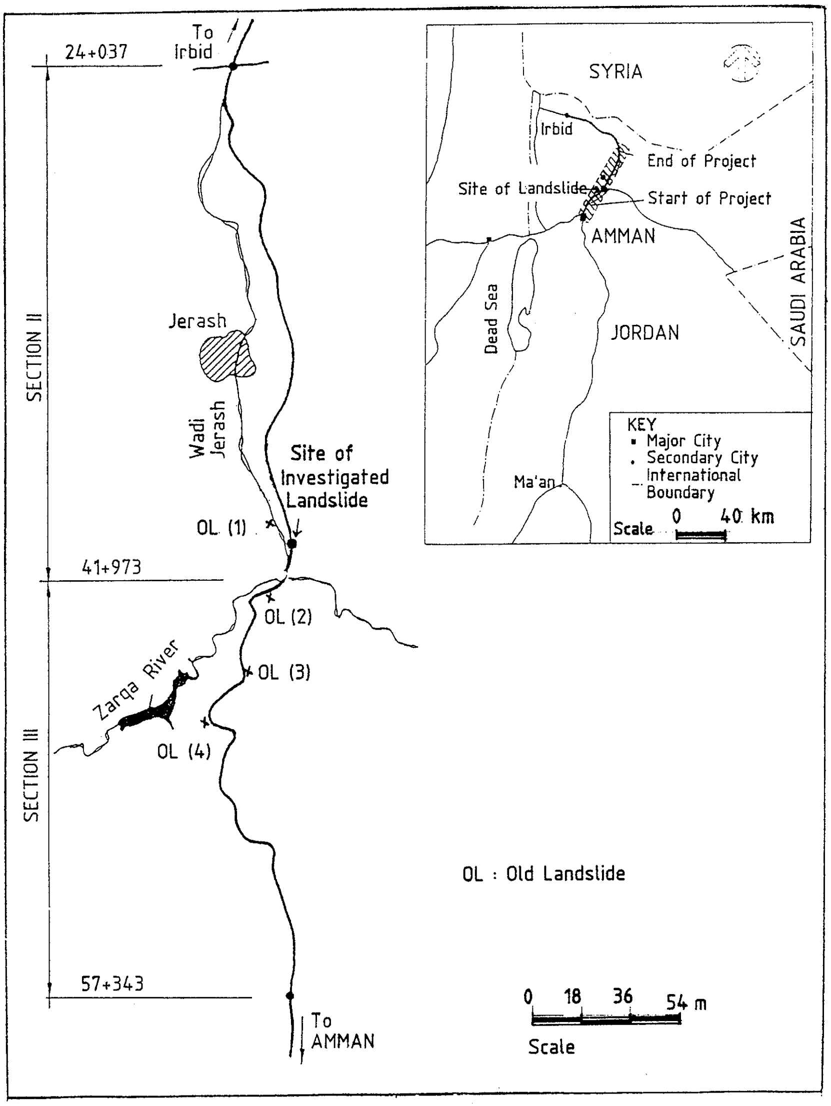

Fig. 1. Location map of Irbid-Jerash-Amman highway and site of landslide at station 40+700

This paper presents results of the study and includes recommendations for the appropriate remedial measures to stabilize the sliding area and build a safe road.

GEOMORPHOLOGY

The study area lies within the geological province of mountain ridge and northern highland, east of the Jordan Rift Valley. The development of the Rift Valley has led to continual rejuvenation of streams draining from the high ground down to the Dead Sea, thus giving rise to steep slopes and continual active wadi erosion.

The landslide at Station 40+700 occupies the western slope of a wadi running approximately south-north. This slope is characterized by a chaotic, stepped profile having a very steep slope. It descends by a heterogeneous series of impersistent rock scarps. Gullies, screen and soil-rock flows. In places, the area is strewn with boulders.

The surface elevation of the boreholes varies from about El+292 near borehole B-1 to El+256 near borehole B-8. Most of the area is vegetated except for the scarps. A number of local water seepages and springs were observed in the study area. Figure 3 illustrates the geomorphologically important areas and their subdivisional zones commencing at the wadi to the east and moving upslope of the study area. The embankment area is defined as that extending from the wadi on the east up to the existing road elevation. The upper slope area is defined as the area west of the existing road.

GEOLOGY

Stratigraphy

The rocks outcropping in the vicinity of landslide at Station 40+700 consist of the variegated Kurnub Sandstone Group (K2) of Lower Cretaceous Age. Where rock and artificial fill are not exposed, a superficial cover of the weathering products is present.

The geological sequence of the area of the landslide is outlined in the following paragraphs:—

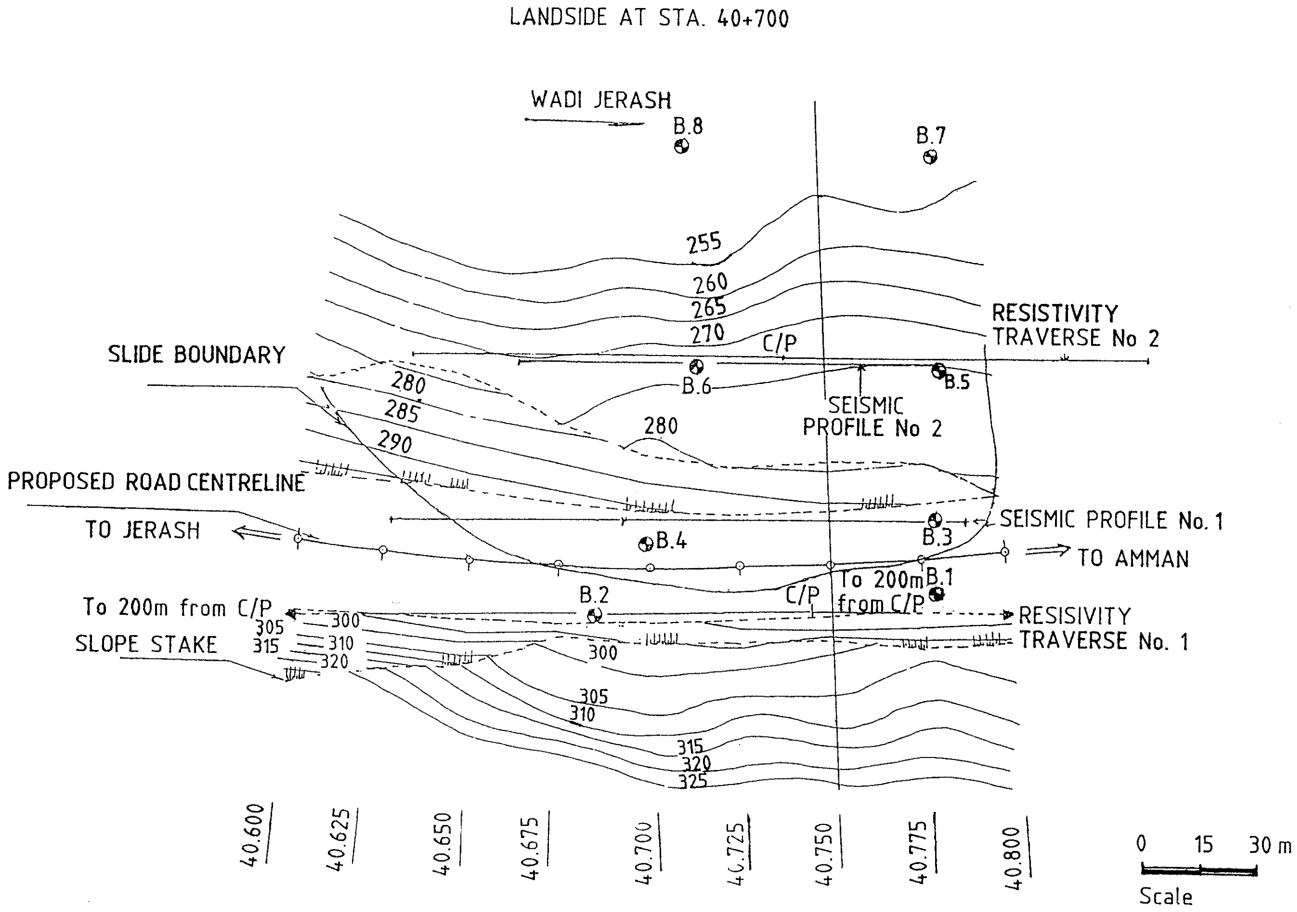

Fig. 2. Detailed location map of landslide at station 40+700 showing location of boreholes

1. Lower Cretaceous, Kurnub Sandstone Group
   Albian, Subeihi Formation K2.

This sandstone unit is the oldest rock unit in the study area. It is characterized by a thin ferruginous cap layer and a transition from mostly red-violet sandstone to greenish gypsiferous sandy shale. The sandstone unit is mostly violet, brown, yellowish, and white cross-bedded. It consists of fine to coarse sands with varying degrees of friability with occasional piping and cavernous effects.

The following are the main features of the Kurnub sandstones:

* Lateral and vertical lithological variations.
* Alternating sand, shale, clay and clayey sand layers.
* Presence of thin-To thick-bedded clayey layers and lenses.
* Presence of high deformation features represented by fracturing, jointing and local faulting.

2. Quaternary-Recent, Surficial Cover

The surficial cover consists of rock and soil materials which have accumulated through the processes of weathering, erosion, mass movements and artificial filling. These include the unconsolidated heterogeneous colluvial and alluvial materials covering the slope as well as the talus material on top of the bedrock.

Structural Geology

The landslide area is bounded to the north by an east-west trending anticline; to the west by the steep hillside of the Kurnub sandstones and to the east by Wadi Jerash.

The area within the described boundaries lies on the southern limb of a major open anticline. The dip of the strata is clearly seen in the rear sandstone cut located at the western boundary. This outcrop appears to be in situ, and is used as a geological reference for the remainder of the outcrops. Shale layers are also exposed in this cut.

The regional dip of the strata of the rear scarp is 10 to 15 degrees to the south, slightly in favour of stability. Kurnub sandstones at the landslide area is obscured by the old debris and artificial fill.

Along much of the wadi and at the toe of the landslide, however, the debris, which covers the sandstone, is being actively eroded. It is probable, therefore, that earlier landslides have occurred during periods of erosion and down cutting of the Kurnub Sandstone. More recent natural or man-made alterations to the environment are also responsible for reactivating landslide movements.

Joint Surveying

Rock outcrops observed above the road alignment were the subject of a joint survey study. It should be noted that data collected has been limited by accessibility

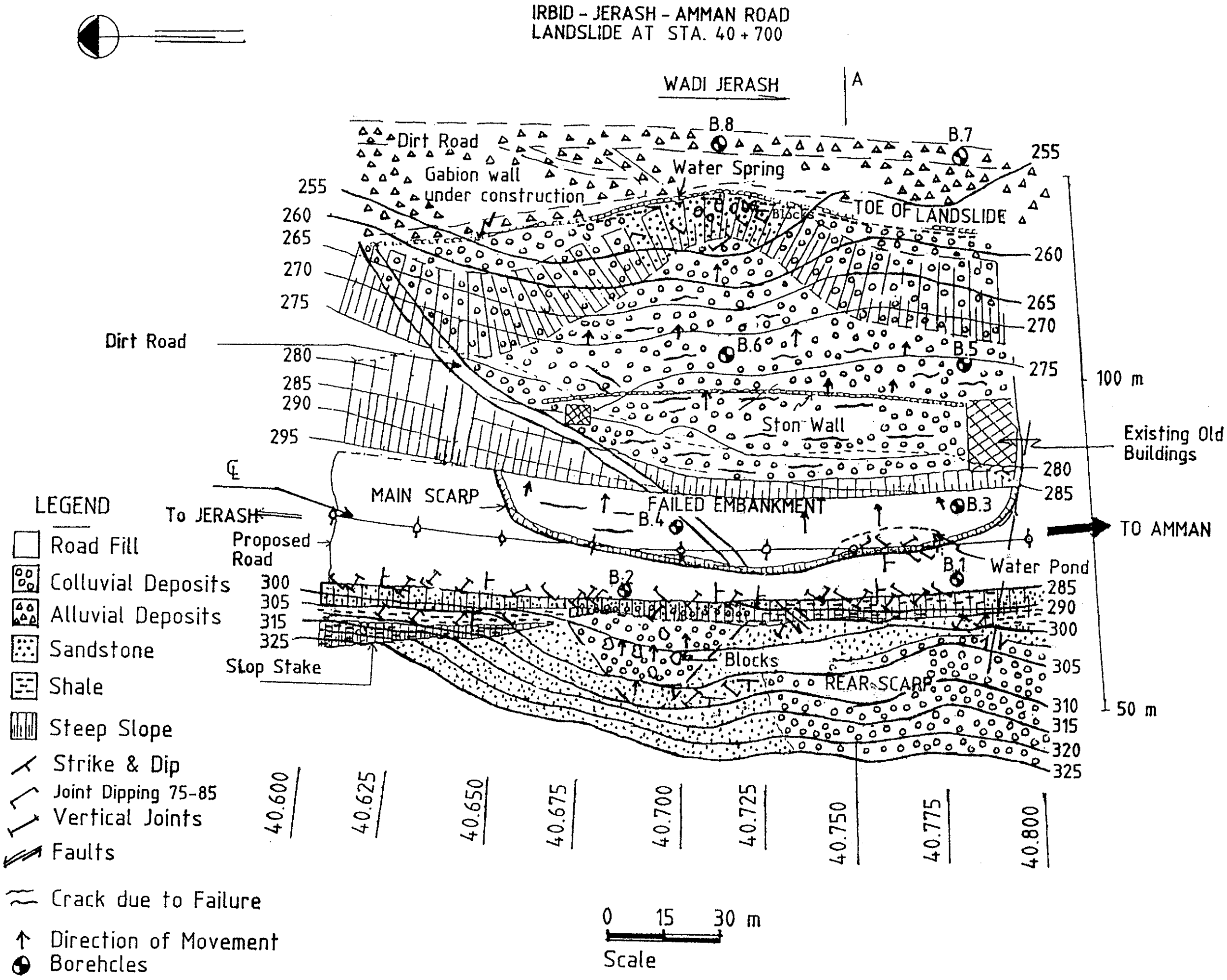

Fig. 3. Geotechnical map

to the exposed faces. Full "scan line" joint surveys were not possible. Two main vertical joint sets were identified, namely 160 and 230 degrees.

FIELD INVESTIGATION

The field investigation was conducted between August 1, 1992 and September 12, 1992 and consisted of the following: (1) eight sample borings (B-1 through B-8) were drilled to depths ranging from 20 m to 50 m, and (2) seismic and resistivity profiles were conducted along selected lines.

Eight sample borings designated B-1 through B-8 were drilled within the study area at the locations shown in Fig. 2. The logs of individual borings are given in Ref. 1. The sample borings were drilled to investigate subsurface soil and rock stratigraphy, obtain representative samples of soil and rock for testing, and investigate subsurface groundwater conditions. The borings were drilled with a Bomag B-100 drill rig, using the air flush rotary method of drilling. In order to obtain bulk samples of fill and talus deposits, some borehole sections were advanced by drilling without air and using a 5.5-in. single-core barrel. Samples of the materials encountered in the borings were recovered using 412 double-tube rock core barrel, 2-in. split barrel sampler, and 5.5-in. single core barrel sampler. Samples were taken continuously throughout the borings depths in each borehole. Percent recovery and Rock Quality Designation (RQD) values are shown on the boring logs. Standard Penetration Tests (SPT) and shear vane tests were performed in general accordance with the equipment and procedures recommended by ASTM. Samples from the boreholes were classified in the field. The samples were then sealed against moisture changes, where relevant, boxed and transported to the laboratory.

LABORATORY TESTING

Classification Tests

Water content and dry unit weight were determined as routine portions of unconfined, triaxial and direct shear tests. Natural water contents were also determined for many other samples recovered from the borings. Liquid and plastic limits were determined on selected specimens from the sample borings in order to evaluate soil plasticity and aid in soil classification.

The grain size characteristics of many samples were determined by sieving and hydrometer analyses. In addition, the percentage of material finer than the No. 200 sieve was also determined for several samples.

Strength Tests

The shear strength properties of the near-surface and subsurface materials were investigated by unconsolidated-undrained direct shear tests, unconfined compression tests, unconsolidated-undrained triaxial compression tests and consolidated-undrained triaxial tests with pore pressure measurements conducted on undisturbed and remoulded samples. The peak and residual strength parameters were determined. Compression tests were conducted on rock specimens recovered from the boreholes to assess their strength properties. A summary of the mean results of the laboratory testing carried out, grouped in accordance with the classification of the various strata, is given in Table 1.

SUBSURFACE CONDITIONS

Stratigraphy

Although localized variations in soil and rock quality and character occur within the subsurface strata, the borings made for this study disclosed general trends in subsurface conditions. The talus and the fill were especially difficult to sample and characterize because of their lack of homogeneity. Subsurface conditions at the site are depicted graphically by the generalized stratigraphic profile presented on (Fig. 4). This profile indicates that the materials can be divided into four generalized strata described below. The engineering properties of each layer are presented in Table 2.

Stratum I.

This stratum is represented by the failed embankment and covers the area along the proposed road. The predominant material present within this stratum is man-placed fill of light brown to yellow clay, silty clay,

Table 1. Summary of laboratory data

Material description	Water content (%)	Wet unit weight, γ (kN/m^3)	Dry unit weight, γ_d (kN/m^3)	Liquid limit (%)	Plastic limit (%)	PI[(1)] (%)	UCS[(2)] (MPa)	Grading		
								% Clay	% Silt	% Sand
Fill (Stratum I)	8(17)	21.7(2)	20.0(2)	24(5)	19(5)	5(5)	2.1(2)			
Fill (Colluvium) (Stratum I)	11(12)	21.7(2)	19.9(1)	24(6)	16(6)	9(6)	0.8(2)	15(1)	31(1)	54(1)
Colluvium (Stratum II)	12(39)	21.2(8)	18.9(8)	25(22)	18(11)	11(11)	0.9(8)	31(8)	31(8)	38(8)
Alluvium (Stratum II)	11(8)	21.1(2)	19.0(2)	27(2)	17(2)	11(2)	0.6(2)			
Weathered Rock (Shale) (Stratum III)	18(70)	20.8(10)	17.8(10)	29(37)	19(37)	14(37)	2.6(9)	28(16)	48(16)	24(16)
Weathered Rock (Sandstone) (Stratum IV)	28(125)	21.0(7)	18.0(7)	18(64)	15(11)	9(11)	6.8(11)	9(3)	38(3)	53(3)

Material description	Direct shear-UU				Direct shear-CD				Triaxial compression-CU			
	Water content (%)	Dry unit weight, γ_d (kN/m^3)	c (kPa)	ϕ (deg)	Water content (%)	Dry unit weight, γ_d (kN/m^3)	c (kPa)	ϕ (deg)	Water content (%)	Dry unit weight, γ_d (kN/m^3)	c (kPa)	ϕ (deg)
Colluvium (Stratum II)	16(5)	18.7(5)	54(5)	12(5)					17(3)*	17.7(3)*	30(3)*	17(3)*
Weathered Rock (Shale) (Stratum III)	17(15)	18.4(15)	44(15)	8(15)	17(1)	18.5(1)	25(1)	22.0(1)	17(5)	17.8(5)	23(5)	21(5)
Weathered Rock (Sandstone) (Stratum IV)	17(5)	18.0(5)	28(5)	23(5)								

(1) PI=Plasticity Index, (2) UCS=Unconfined Compressive Strength
Mean values shown with number of test in brackets.
All shear tests on remoulded samples except where marked*, where undisturbed samples were tested.

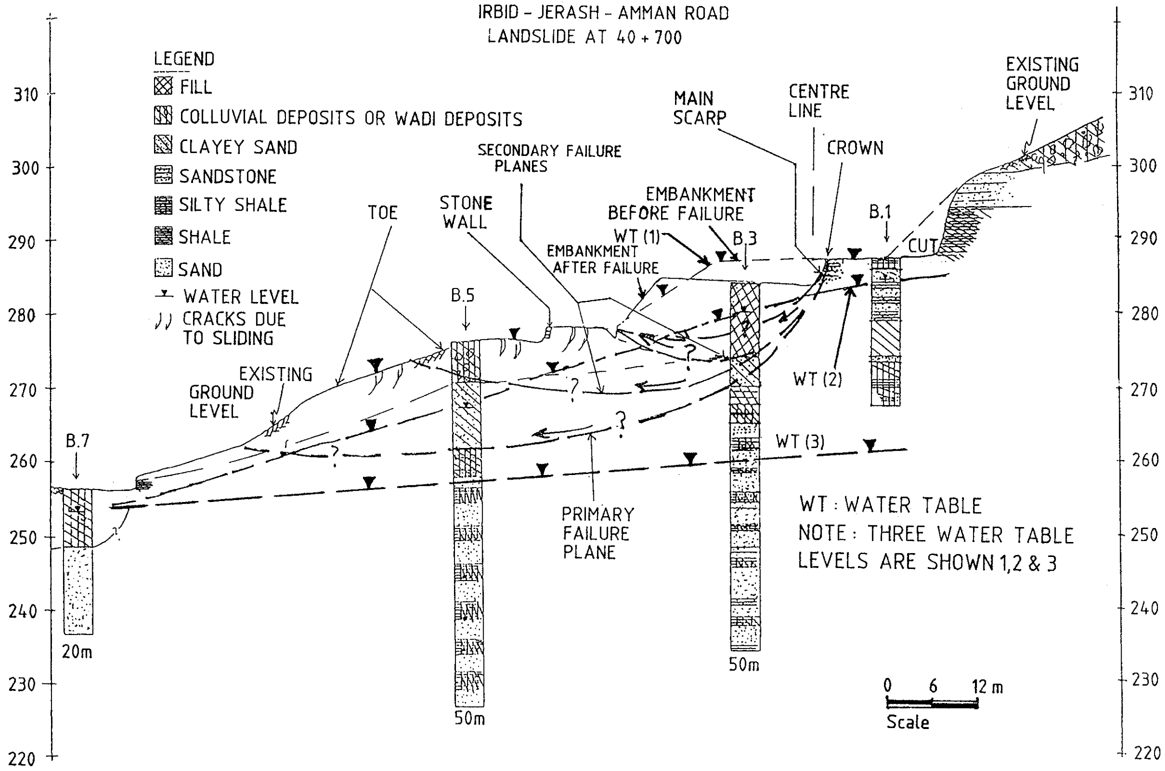

Fig. 4. General stratigraphic profile, configuration of failure planes and piezometric water elevations considered in stability analysis

Table 2. Depth of groundwater in boreholes

Borehole No.	Depth of groundwater, m	
	During drilling (August 1 to August 30, 1992)	On September 19, 1992
B-1	5.00	2.7
B-2	20.50	8.0
B-3	15.00	3.6
B-4	17.00	5.4
B-5	—	8.8
B-6	—	7.5
B-7	3.00	3.0
B-8	1.00	0.0

and sandy clay with gravels and boulders of limestone and sandstone. This stratum has no significant structure or texture. The thickness of this top stratum varies from zero to about 10 m below existing ground levels. The maximum thickness of this stratum is about 10 m in boreholes B-3 and B-4. It is our understanding that the fill material was obtained from borrow pits or areas and/or cuts within the vicinity of the site.

Stratum II.

This stratum covers most of the site below the road level. The predominant material present is naturally deposited fill either on the hillside (colluvial) or at the bottom of the wadi (alluvial) and consists of a heterogeneous mixture of clay, silt, sand, gravels and boulders. The thickness of this stratum varies between 5 m and 8 m.

Stratum III.

This stratum underlies either stratum I or stratum II and extends to a maximum depth of 15 m below existing ground level in boreholes B-5 and B-6. This stratum consists of talus of natural deposits of clayey silty sand with a composition similar to the weathered siltstone of the underlying stratum IV. This stratum exhibits occasional structure and texture. The thickness of this stratum varies between 4 m and 9 m.

Stratum IV.

This stratum underlies either stratum II or stratum III and extends to the maximum depths explored in the borings. This stratum consists of bedrock of interbedded layers of sandstones and shales of the K2 formation. The Kurnub Sandstone Formation was encountered in all boreholes. The natural deposits of this stratum are generally strong, and relatively incompressible, especially when compared to the fill and talus deposits of strata I and II. The layers of this stratum are fissured and jointed, and are often filled with gypsum, or clay deposits. It appears that local faulting and folding have also disturbed the natural deposits in some areas.

The Kurnub sandstones are characterized by the following features:

—lateral and vertical lithological variations,
—presence of alternating of sand, shale, clay and sandy clay layers.
—presence of thin-to thick-bedded clayey layers and lenses.
—high deformation represented by fracturing jointing and some local faulting.

Engineering Properties

Engineering properties can be classified in three general groups related to four main types of materials as follows:—

* Group 1 comprises Stratum I and Stratum II.
* Group 2 comprises Stratum III.
* Group 3 comprises Stratum IV.

A summary of engineering properties is given in Table 2.

GROUNDWATER CONDITIONS

During drilling, free flowing groundwater was encountered in all borings at the depths shown in Table 2. Natural groundwater levels are below the borehole depths. The water encountered in the borings could have resulted from percolation of surface water into the formation or some temporary groundwater flow because of the extremely heavy rains.

GEOTECHNICAL APPRAISAL

It was apparent from the outset of the geotechnical mapping that the area was the site of earlier instability. The formation of the wadi provided lateral freedom for regressive failures of the colluvial deposits onto the interbedded sandstone and shale sequence.

The failure mechanisms within the area above the road are essentially superficial as a result of the lithological and structural characteristics of the sandstones and shales, the steepness of the valley sides and ingress of water.

Recent movements observed above the road are also superficial and have no significant effect on the safety of the road. Natural sub-aerial erosion processes have acted to form the sandstone and shale debris (scree) slope above the road. Bare unvegetated areas suggest that these processes are active at the present. Gully erosion on a limited scale is also evident in this area.

In order to control these processes, the following measures should be adopted:

a. Surface water run-off over the rear scarp into the landslide area should be prevented.
b. Water draining from the upper slope through erosion gullies should be intercepted to prevent erosion and reduce the ingress of water into the slope.
c. Bare areas should be vegetated with indigenous species of shrubs and grass. The following remarks can be made concerning the failed man-made embankment:

i. The embankment was sited on sloping ground probably with no benches or steps keyed into the slope.
ii. The embankment was sited on ground comprising colluvial deposits and probably landslide debris.
iii. Available evidence does not suggest a deep-seated failure extending much below the talus/bedrock boundary that existed prior to construction.
iv. The most likely location of the earthen embankment slip surface is about 10-m to 14-m depth in the vicinity of borings B-3 and B-4.
v. The embankment failure occurred as the embankment was nearly completed. This suggests that some saturated zones existed (possibly within the embankment itself) and that excess pore water pressures developed in the soils within these zones as the total stress increased to the weight of the embankment fill.
vi. The arrangement for water drainage appears to have given opportunity for water to enter the ground and to have an adverse effect on the shear strength parameters of certain zones, either within the embankment or at the interface between fill and natural original ground.
vii. Since the man-made embankment has failed, it is definite that a failure zone exists and that the soils within this zone have strength values significantly less than peak strength and probably tending towards residual at large strain values.
viii. The soil strength along any existing failure surface is less than the strength at the time of failure because of remoulding. The design analysis should incorporate the reduced strength along the slip surface.
ix. Dynamic soil characteristics were not investigated as part of this study. The design analysis should consider potential natural hazards. Among these hazards are the effects of earthquakes which generally increase the driving forces on embankments and slopes and tend to decrease the soil strength or its deformation modulus.

Seismic Design Conditions

The Irbid-Amman highway is situated in a region that is not far from seismically active faults with a major contribution from the Dead Sea fault, a world class strike-slip fault. Therefore, it is important to assess seismic hazard at the landslide site to evaluate slope stability.

Peak Ground Acceleration (PGA) at the bedrock level with 90% probability of not being exceeded in an economical life time of 50 years was calculated based on the line source model incorporated in the computer program

FRISK (McGuire, 1978). The line sources in Jordan and the vicinity were identified to include nine distinctive seismic sources. The pertinent parameters of each time source, such as the b-parameter in the Gutenberg-Richter formula, the annual rate λ_4 and the upper bound magnitude m_1, were determined from two sets of seismic data: the historical earthquake records and the instrumentally recorded earthquake data (from 1900-Dec. to 1991 A.D.). For a 50 year design life of a road cutting the landslide site, a value of 0.2 g was estimated to be the maximum Peak Ground Acceleration (PGA) with 90% probability of not being exceeded.

EVALUATION OF STABILITY CONDITIONS

Stability Analyses

Geometrical configuration of the landslide is shown on Fig. 4. The crown, main scarp and the toe of the landslide were clearly observed at the site. The ground movement below the road may be defined as a rotational circular to non-circular landslide. Possible failure planes along which landsliding may have occurred are shown on Figs. 4 and 5. Basically primary and secondary failure planes were identified.

Failure occurred mainly through colluvial deposits (Fig. 4). Colluvial deposits contain evidence of old landslides in failure planes throughout their mass. Furthermore, only residual shear strength properties can be mobilized along these planes and the non-cohesive nature of these properties make them extremely sensitive to the affect of pore water pressure. Also, because of the method of deposition, such planes are normally oriented parallel to the existing slope. In summary, therefore, in situ colluvial materials are, at best, metastable and make poor foundation materials.

Following the recommendations of Skempton (1964, 1977, 1985), the presence of failure planes within a colluvial mass and where there has been sufficient shearing deformations as a result of the presence of an older landslide, requires the use of residual shear strength parameters when analyzing the limiting stability of such materials. When a colluvial material goes from a peak to a residual strength state, it experiences a complete loss of the cohesive component of shear strength (i.e. $c'=0$) and a considerable reduction (perhaps by 50%) of the frictional component.

Stability analysis was carried out using two-dimensional limit equilibrium slope stability analysis methods incorporated in the computer program STAP5M (Siegel, 1975). Analysis was carried out using two well known methods, the simplified Bishop and the Janbu methods. The Janbu method considers non-circular failure surfaces, but in the simplified Bishop method, non-circular failure surfaces are replaced by an equivalent number of circular surfaces. These two methods give reasonably precise factors of safety (Boutrup, 1977).

In the simplified Bishop method, overall moment equilibrium and vertical force equilibrium are satisfied. However, for individual slices, neither moment nor horizontal force equilibrium is satisfied. The forces on the sides of each slice are assumed to be in a horizontal direction. This assumption implies that there is no friction between two slices. Although equilibrium conditions are not completely satisfied, the method is, nevertheless, a satisfactory procedure and is recommended for stability analysis where the failure surface can be approximated by a circle.

In Janbu's method, both force and moment equilibrium are satisfied. The location of the interslice normal force, or the line of thrust, must be arbitrarily assumed. For cohesionless soils, the line of thrust should be selected at or very near the lower third point. For cohesive soils, the line of thrust should be located above the lower third point in a compressive zone (passive condition) and somewhat below it in an expansive zone (active condition). Janbu method is applicable to failure surfaces of any shape.

Further details on theoretical development and governing equilibrium equations of simplified Bishop and Janbu methods of slope stability analysis can be obtained from Huang (1983).

The level of the water table at the time of failure is unknown. However, analysis was performed for three conditions of groundwater level (Fig. 4): (1) at ground surface (fully saturated condition) (identified in Fig. 4 as WT (1)); (2) intermediate (partially saturated condition) (identified in Fig. 4 as WT (2)); and (3) below failure surface (dry condition) (identified in Fig. 4 as WT (3)).

Strength parameters of the colluvium are those given in Table 2. These were obtained from limited strength tests due to the difficulty in obtaining undisturbed samples from colluvium.

In view of the uncertainly associated with the determination of appropriate strength parameters for the colluvial materials, sensitivity analyses were carried out via back analyses assuming that the failure has occurred when the factor of safety was 1.0, to obtain representative soil parameters of the colluvial material.

Determining soil strength by back analysis avoids many of the problems associated with laboratory testing. This technique is an effective method of accounting for important factors that may not be well represented in laboratory tests, such as the structural fabric of the soil, the influence of fissures on the strength of the soil, and the effects of pre-existing shear planes within the soil mass.

Saito (1980) and Chandler (1977), pointed out that reliable values of c' and ϕ' can be obtained through back analysis (using both circular and noncircular slip surfaces) if the pore pressure used in the analyses reflect accurately the conditions at time of failure. Janbu (1996) used back analysis technique for evaluating effective shear strength parameters for several failures in cuts, slopes and embankments. Janbu (1996) concluded that using such technique it is actually possible to determine the complete state of limit equilibrium behind slopes, independent of a priori knowledge of the shear strength.

The critical sliding failure surface was found to be the one that connects the crest and the toe observed in the field and identified as the primary failure plane in Fig. 4. Tables 3 through 5 summarize the results of slope stability analyses. Moreover, Figs. 5 and 6 show the variation of calculated values of effective cohesion (c') with assumed values of effective angle of friction (ϕ') that give a factor of safety of 1.0 for three conditions of ground water level for the Bishop and Janbu methods, respectively. Also shown on each Figure are the sets of c' and ϕ' values obtained from consolidated undrained (CU) triaxial strength tests.

Since the colluvial deposits contain evidence of old landslides, it is reasonable to use residual shear strength parameters. As, discussed previously, colluvial deposits have a residual cohesion $c'=0$. Considering this fact, back analysis give a factor of safety equal to 1.0 for a residual angle of friction of 20.0°, assuming a partially saturated condition which is believed to be the prevalent condition at the time of failure. Therefore, it is believed that the most reasonable shear strength parameters for colluvial deposits are: $c'=0$ and $\phi'=20.0°$. These are identified in Figs. 5 and 6 as design parameters. It is obvious that these design strength parameters are more

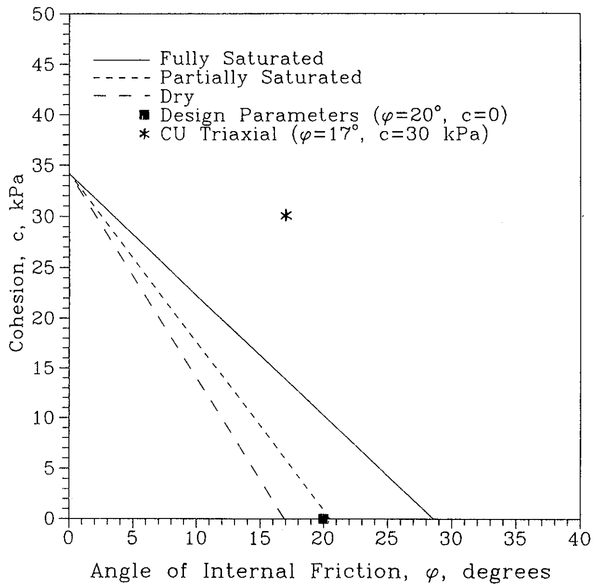

Fig. 5. Results of slope stability back analysis using Bishop method at different conditions of water table

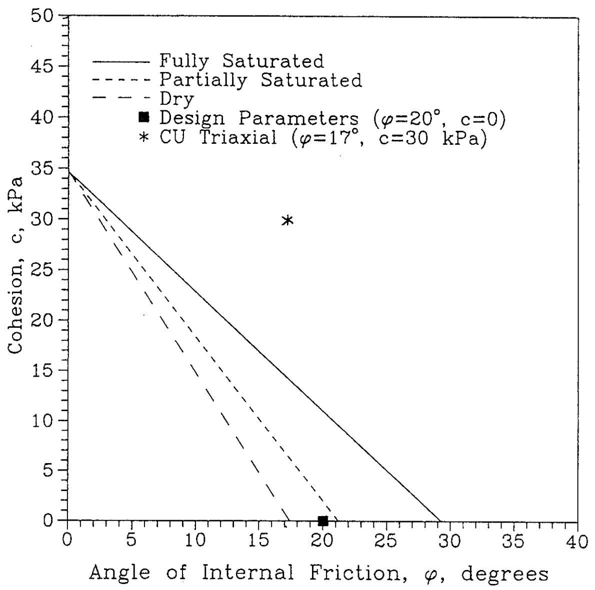

Fig. 6. Results of slope stability back analysis using Janbu method at different conditions of water table

Table 3. Results of slope stability analysis at dry condition

Source of data	Shear strength parameters		Factor of safety (FS)		Difference in FS between two methods (%)
	ϕ (deg)	c (kPa)	Modified Janbu	Simplified Bishop	
CU. triaxial test (Peak)	17	30	1.875	1.936	3.0
Back analysis	17.4	0	1.001	1.031	2.9
	0	34.7	1.000	1.022	2.2
	16.9	0	0.970	1.000	3.0
	0	34.2	0.971	1.001	3.0
Design	20	0	1.162	1.198	3.0

Table 4. Results of slope stability analysis at partially saturated condition

Source of data	Shear strength parameters		Factor of safety (FS)		Difference in FS between two methods (%)
	ϕ (deg)	c (kPa)	Modified Janbu	Simplified Bishop	
CU. triaxial test (Peak)	17	30	1.795	1.857	3.3
Back analysis	21.2	0	1.000	1.041	3.7
	0	34.7	1.000	1.022	2.2
	20.53	0	0.962	1.000	3.8
	0	34.2	0.982	1.000	2.0
Design	20	0	0.959	0.995	3.6

Table 5. Results of slope stability analysis at fully saturated condition

Source of data	Shear strength parameters		Factor of safety (FS)		Difference in FS between two methods (%)
	ϕ (deg)	c (kPa)	Modified Janbu	Simplified Bishop	
CU. triaxial test (Peak)	17	30	1.520	1.570	2.9
Back analysis	29.3	0	1.000	1.028	2.8
	0	34.7	1.000	1.018	1.8
	28.6	0	0.977	1.000	2.3
	0	34.2	0.980	1.000	2.0
Design	20	0	0.652	0.670	2.2

reasonable than those obtained from laboratory strength tests.

Analysis showed that the presence of excess pore water pressure within the slope materials has a marked effect on their stability. Furthermore, results have indicated that the difference in the factors of safety evaluated using the Janbu and Bishop methods of analysis is between 1.8% and 3.87%, with factors of safety obtained from the latter method being higher than the ones from the former for similar conditions.

Considering design shear strength parameters, the factor of safety for the present ground situation (after failure) is 1.66 at almost dry conditions and 1.1 with a high groundwater level. Therefore, any rise in the groundwater level or the reconstruction of the failed embankment to the original road level will increase the driving forces and will decrease the factor of safety to less than 1.0, and failure will reoccur.

A 0.2 g PGA value further reduces the factor of safety by about 30% at dry conditions and by about 50% at partially saturated and saturated conditions. Based on these results instability would also have occurred at this site if a moderate earthquake had taken place even in a dry condition.

REMEDIAL MEASURES

In order to reinstate the road along the proposed alignment, five separate concepts of road design and other remedial measures are considered. They are as follows;—

1. Improving the embankment:
   This implies the removal of about 70000 m^3 of the existing embankment to a depth of about 20 m below existing ground level, which is the top of the sandstone layer. The temporary side slopes of such excavation should not exceed 1-horizontal to 1-vertical. Imported free draining rockfill has then to be placed at the location of the excavated fill as shown on Fig. 7. The factor of safety in this case with the groundwater level at ground surface is about 2.0. Considering design earthquake loading (i.e. PGA = 0.2 g), the factor of safety is reduced to 1.0. According to the codes of practice for design of geotechnical engineering structures, a factor of safety of 1.0 is acceptable if earthquake loading is considered.

2. Anchored pile wall:
   This implies the construction of a support system through a network of piles and anchors to retain the failed part of the embankment. The support system consists of about 150 piles of about 40 cm in diameter which extend to an average depth of 25 m. The heads of the piles are to be tied horizontally by about 150 anchors which extend to about 25 m into the sandstone bedrock underneath the proposed road as shown on Fig. 8.

3. Shifting the whole road into the hillside:
   This implies the relocation of the proposed road by cutting back into the hillside to a distance of about

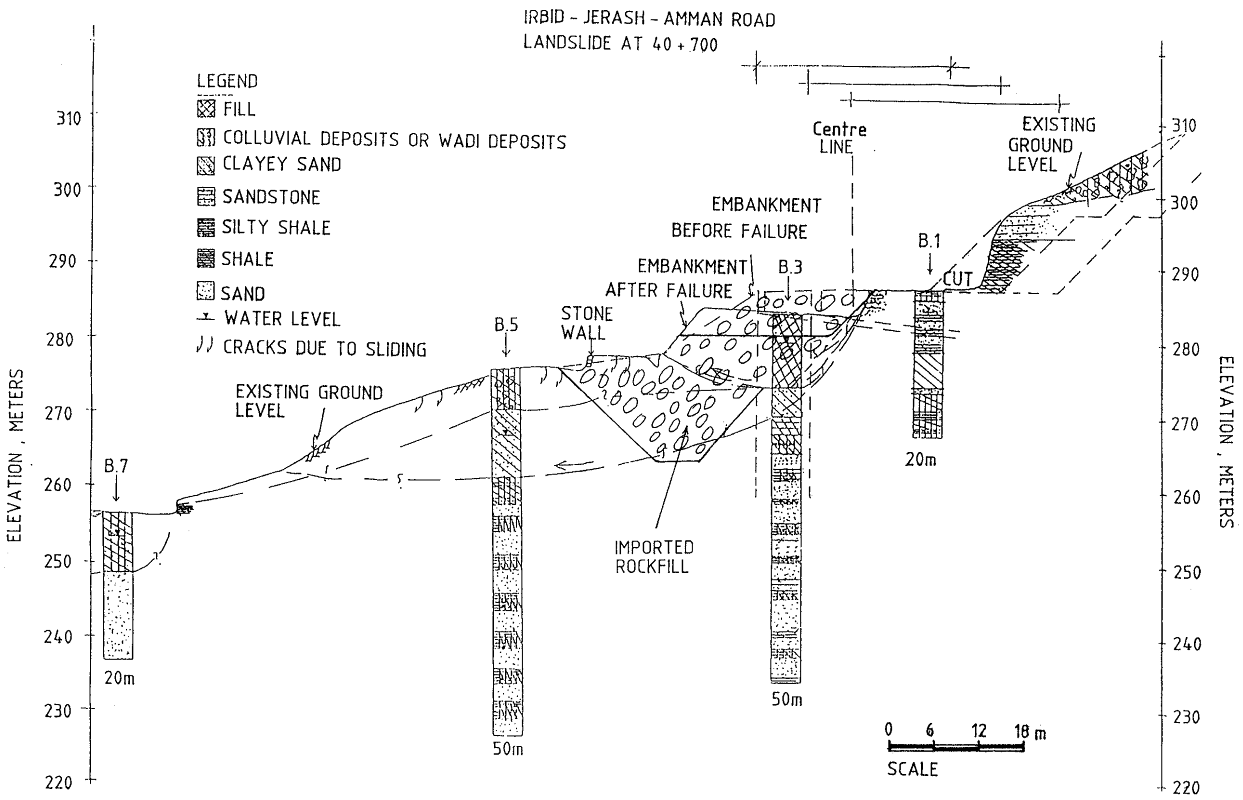

Fig. 7. Section of improved embankment using imported rockfill

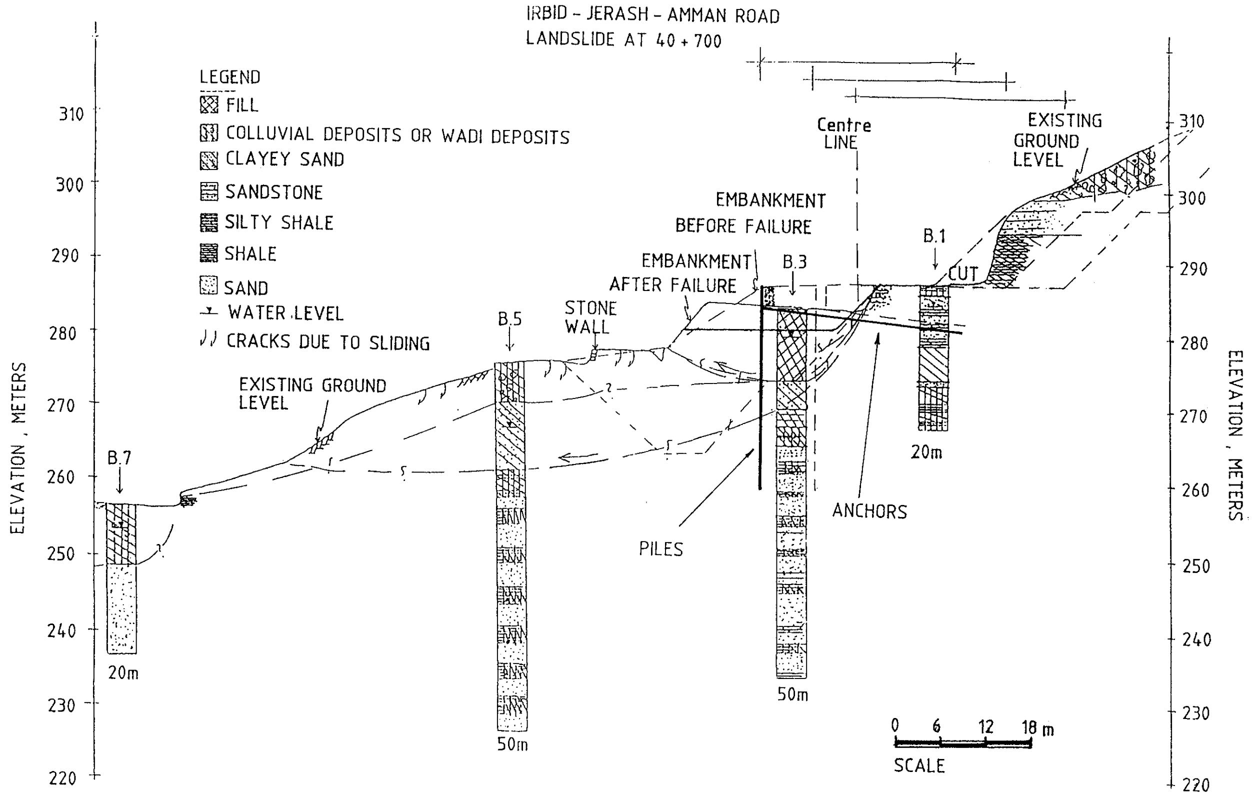

Fig. 8. Section of supported embankment using anchored pile wall

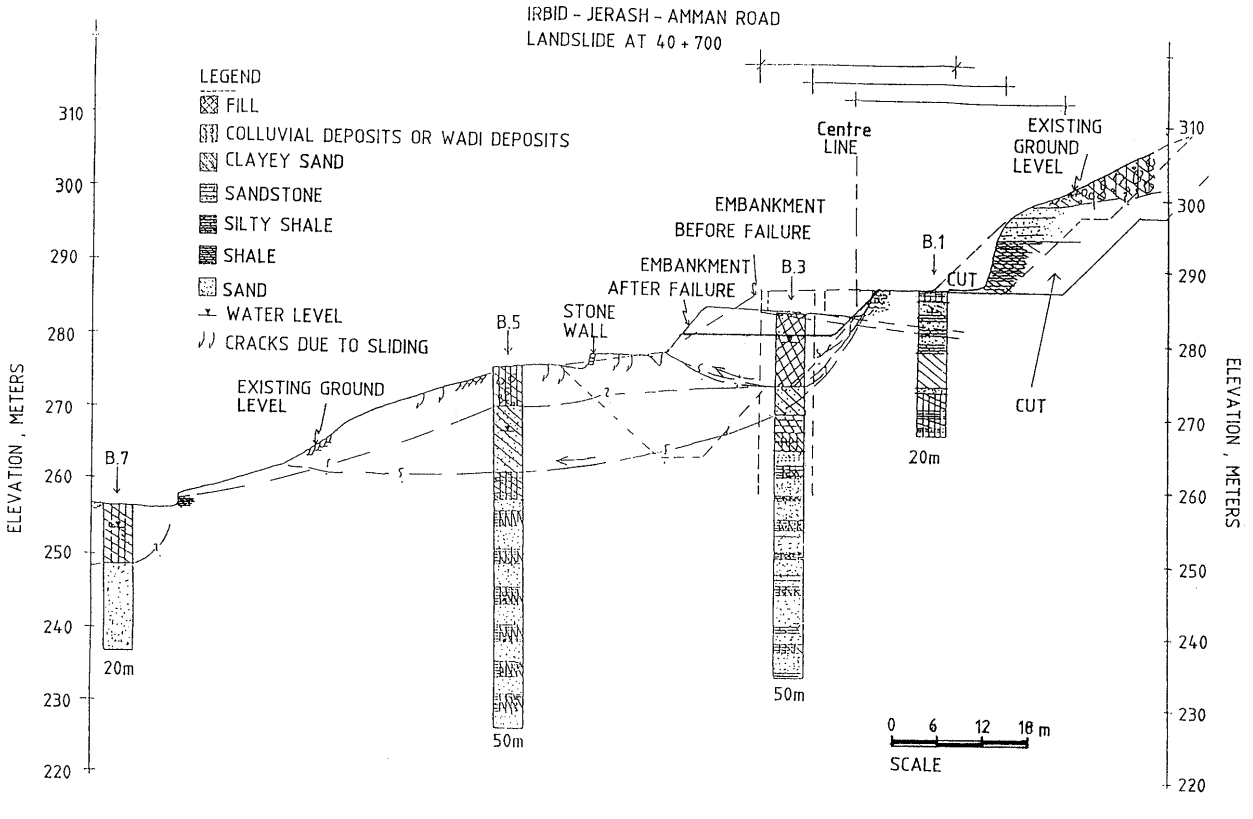

Fig. 9. Section of shifted road into cut

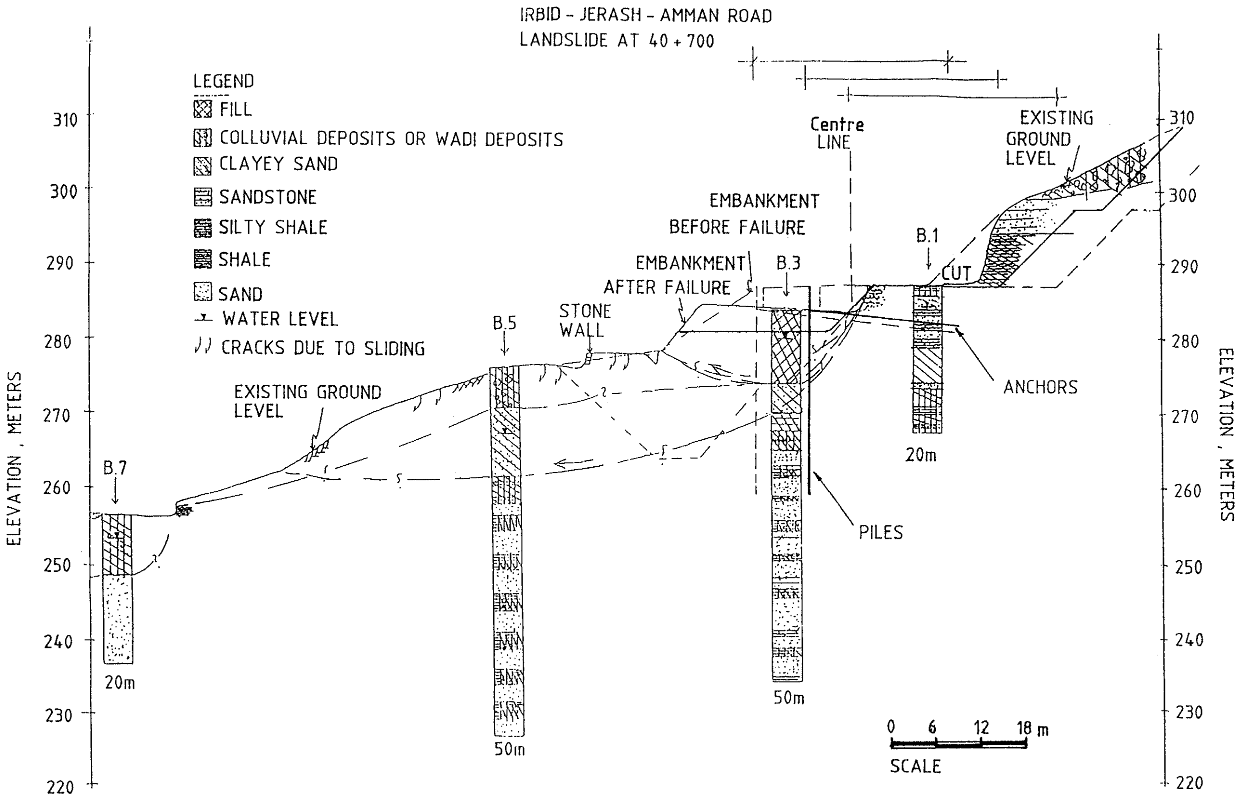

Fig. 10. Section of supported embankment using anchored pile wall system together with shifting in cut

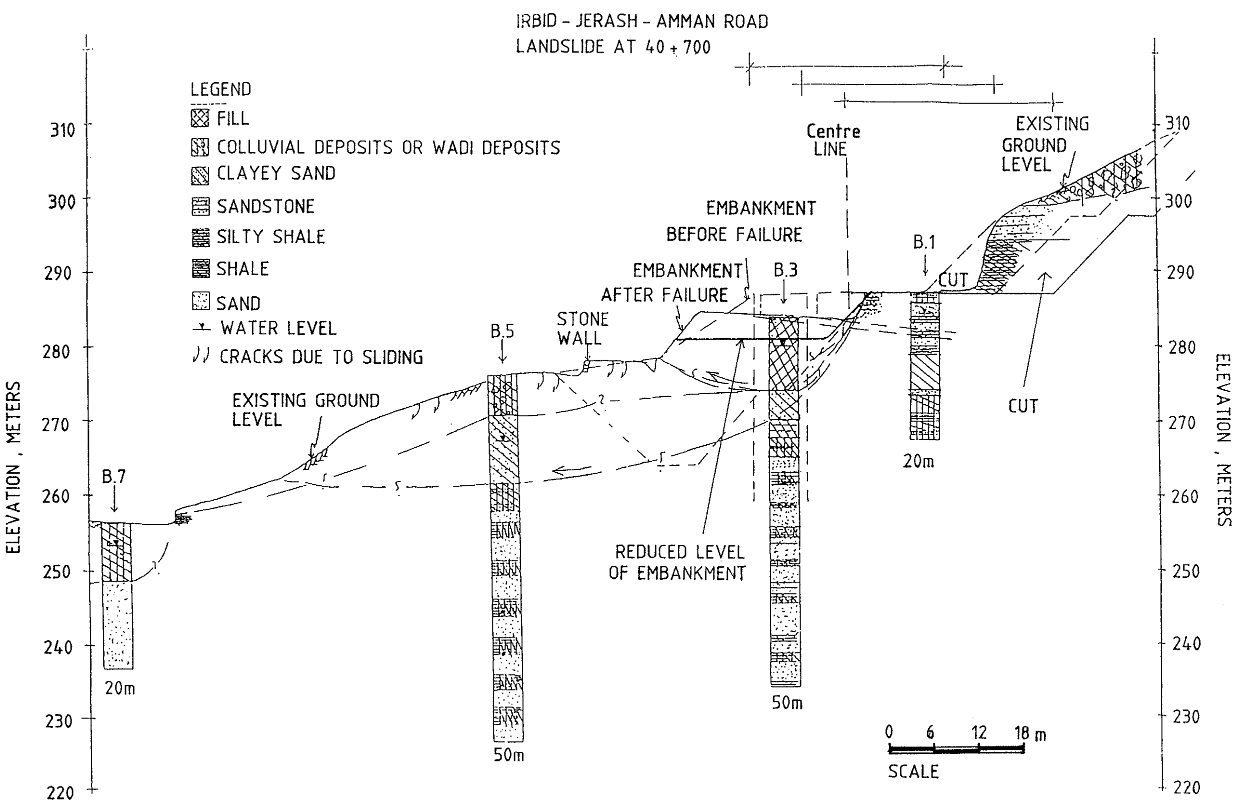

Fig. 11. Section of lowered embankment

15 m to the west. The road will then be placed totally on cut. The failed embankment will be outside the zone of construction activities as shown on Fig. 9. The side slopes of the permanent cuts should be no steeper than 1-horizontal to 1-vertical to a maximum height of 6 m. Benches of 4 m wide should be provided. The shale layer at the bottom of the cut should be protected by a caprock of sandstone.

4. Shifting the road partly into the hillside:
This implies the relocation of the proposed road by cutting back into the hillside to a distance of about 7 m to the west. This means that the proposed road will be placed partly in cut and partly on fill. This also implies the bracing of the failed part of the embankment by a system of piles and anchors similar to the system mentioned in 2, above, but of less extent, as shown on Fig. 10.

5. This alternative consists of constructing a one-level or split-level embankment by reducing the height of embankment by about 6 m below the original proposed road level as shown on Fig. 11. This alternative implies removing the top layers of the existing embankment and, hence reducing the driving forces and increasing the factor of safety. The factor of safety in this case with full saturation is about 1.30. Considering design earthquake loading, this value is reduced to about 0.65. If this method is adopted, it is not a safe one under earthquake loading. Moreover, additional geotechnical information is needed about the sections of the road which are going to be affected by this arrangement. Inclinometers and piezometers have to be installed below the road to monitor the stability of the road during operation. If the road is found to be unstable, this method may then be ameliorated by replacing the existing embankment by imported free-draining rockfill as in 1 above. However, if 1 is adopted together with this method the resulting factor of safety with full saturation condition is 2.70. Considering design earthquake loading, this value is reduced to about 1.35. It is evident that, if geometrical considerations are within tolerable limits, this combined alternative is the most feasible.

6. Drainage
In addition to any of the proposed methods adopted as remedial measures, the landslide area should be adequately drained. Such drainage should include measures to prevent ingress of water into the slope and erosion of the surface. It will be required to construct concrete lined interceptor ditches on the upper slope. All water should be discharged, through flexible drains, away from the landslide area. Furthermore, bare areas should be vegetated.

7. Precaution measures
The conditions of materials upslope of the proposed alignment are such that substantial cleaning, flattening and other works will be necessary to protect the new highway and its users. For safety the following points should be considered:—
   a. Fallen boulders litter much of the upper area. All such blocks in danger of falling should be removed.
   b. Natural sub-aerial erosion has produced loosened blocks on the sandstone rock slope. All such outcrops should be scaled down to remove loose blocks.
   c. If the carriageway design requires cutting back into the slope, consideration should be given to the consequences of the possibility of activating a retrogressive landslide by removing lateral support, particularly when the cut is located near the base of the shale outcrops.
   d. Where blasting is employed to cut the slopes, all blasts should be designed to minimize ground accelerations. Final faces should be formed using the smooth blasting technique in order to leave the slope in as good a condition as possible.

Table 6 shows a relative comparison between the different concepts of the road redesign. The evaluation of the countermeasures (i.e. design concepts) with respect to the different items listed in Table 6 (e.g. stability in fill, stability in cut, etc.) was made considering results of topographical and geological survey conducted in this study. The results of evaluation are given in linguistic terms (e.g. high, medium, low, etc.) in Table 6. For some items, factors of safety were obtained (as discussed previously) reflecting the impact of topographical, geological, and geotechnical survey results.

Therefore considering safety under static and earthquake loading and the state of practice of Geotechnical Engineering in Jordan, the most feasible alternative concept of road design and remedial measures to stabilize the landslide area is a combination of removal of existing failed embankment down to the top of sandstone layer (i.e. to a depth of about 20 m below existing ground level) and reconstruction using imported rockfill of a split level embankment by reducing the height of embankment by about 6 m below the original proposed road level together with the construction of surface and subsurface drainage system. Figure 12 shows a scheme of the drainage system. The resulting factor of safety with full saturation condition is 2.70. This value drops to 1.35 considering design earthquake loading. This option was successfully implemented in the field.

SUMMARY AND CONCLUSIONS

A geotechnical investigation was conducted for a landslide that had resulted in an embankment failure at Station 40+700 along the Irbid-Amman Highway. The principal findings developed by this study are summarized as follows:—

1. The embankment failure can be attributed to exces-

Table 6. Comparison between different alternative remediation concepts for road redesign

Item	Design concepts				
	Reconstruction entire embankment using a free drainage rockfill	Supporting failure part of embankment via anchored pile wall support system	Shifting the entire road into the hillside	Shifting the road partly into the hillside together with anchored pile wall support system	Reconstructing a split level embankment using a free-drainage rockfill
1. Interaction with nature	High	Low	High	Low	Low
2. Safety during and after construction	High	High	Low	High	High
3. Stability in cut	High	High	Low	High	High
4. Stability in fill	High	High	High	High	High
5. Total cost	High	High	Low	Medium	Low
6. Cost of maintenance after construction	Low	Low	High	Low	Low
7. Time of execution	Long	Short	Short	Short	Short
8. Earthquake effect	Low	High	Medium	High	Low

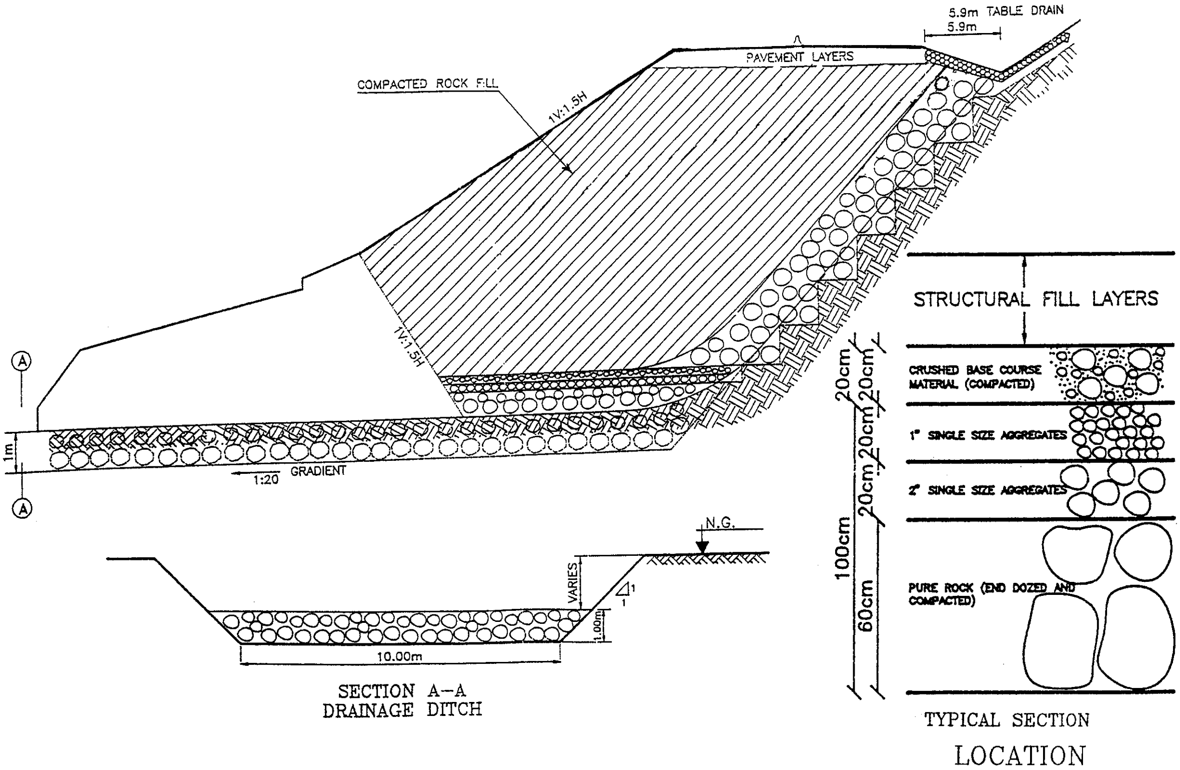

Fig. 12. Typical scheme of surface and subsurface drainage system accompanying remedial measure at station 40+700

sive load of embankment, improper placement procedures such as building of the embankment on colluvial deposits, and improper surface and subsurface drainage causing the ingress of water into the embankment material and underlying colluvium, therefore, resulting in excess pore water pressure.

2. The landslide movement occurred mainly within the colluvium deposits at the foundation of the embankment.
3. Colluvium deposits contain the evidence of old landslides. Only residual shear strength parameters were mobilized at time of failure with cohesion (c') $=0$. Back analysis resulted in mobilized effective residual shear strength parameters of cohesion (c') $=0$, and angle of friction (ϕ')$=20.0°$.
4. Probabilistic seismic hazard assessment of the landslide site using the computer program FRISK (McGuire, 1978) resulted in a Peak Ground Accelera-

tion (PGA) value of 0.2 g with a 90% probability of not-exceeding this value in 50 years.

5. Measures to stabilize unstable zones are discussed in this study along with specific drainage requirements. The following are five concepts of road design and remedial measures for the landslide area: (1) Improving the embankment by replacing the existing embankment by imported free-draining rockfill; (2) Construction of a support "anchored pile wall" system through a network of piles and anchors to retain the failed part of the embankment; (3) Shifting the road 15 m into the hillside by cutting back into the sandstone; (4) Shifting the road 7 m into the hillside partly by cutting back into the sandstone and partly by bracing the failed part of the embankment by a system of piles and anchors; and (5) Reducing the height of the embankment by 6 m below proposed road level. A one-level or split-level embankment scheme may be adopted.
6. The most feasible remedial measure to stabilize the landslide area is a combination of removal of existing failed embankment down to the top of sandstone layer (i.e. to a depth of about 20 m below existing ground level) and reconstruction (using imported free-drainage rockfill) of a split level embankment by reducing the height of embankment by about 6 m below the original proposed road level together with the construction of surface and subsurface drainage system. The resulting factor of safety with full saturation condition is 2.70. This value drops to 1.35 considering design earthquake loading. This option was successfully implemented in the field.

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