Indian Geotechnical Journal, Vol. 25 (2), 1995

A Review of Twenty Years of Geotechnical Studies at Na'ur Landslide No. 4 in Jordan

by

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Introduction

This paper is concerned with a comprehensive review of geotechnical investigation of a major and historical landslide which occurred in Jordan along the road between Amman and the Dead Sea through Na'ur and Adasiyeh towns. The landslide is known as Na'ur Landslide No. 4. This landslide is about 600 m in length (Fig. 1). The paper includes a brief summary of hydrology, geomorphology, geology and engineering geology of the landslide area. Results of stability analyses were performed and given together with recommendations for stabilizing the area.

The constructed road that passed through Na'ur landslide No. 4 in

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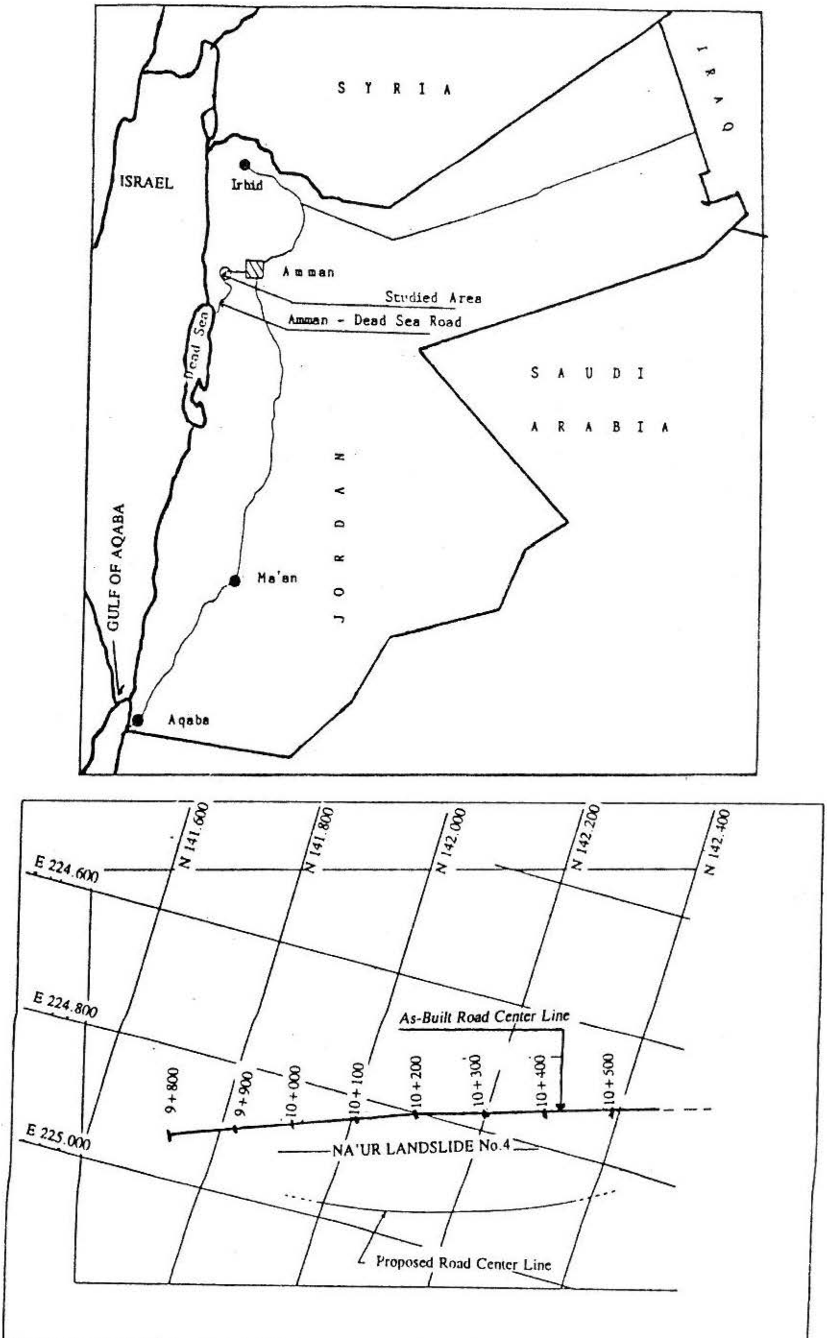

FIGURE 1 Location Map Showing Amman Dead Sea Road, and Studied Area (Na'ur Landslide No.4), and the As-Built and Recommended Alignments.

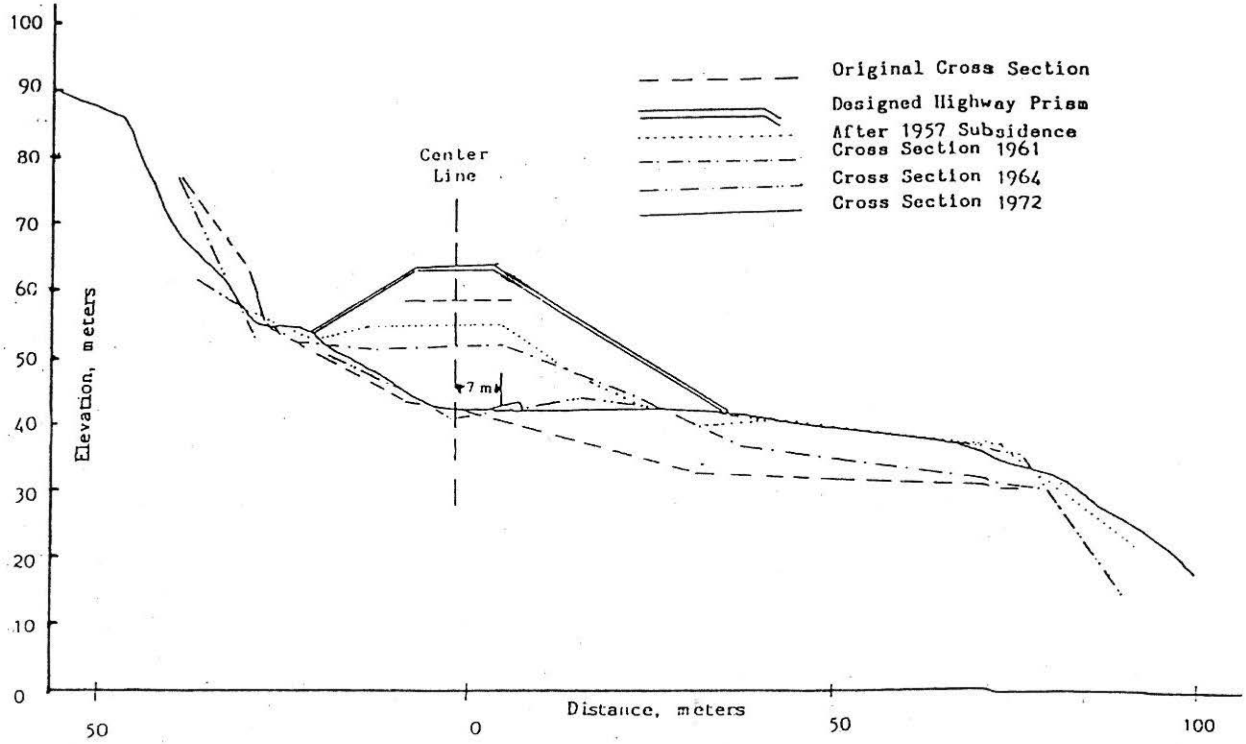

FIGURE 2 Cross Section of Different Subsidence of Na'ur Landslide No.4 During the Period (1957-1972).

1954 involved a proposed 20 m high embankment. In 1956, when the embankment was 14 m high and 200 m length, the embankment has subsided about 7 m. After reconstructing the embankment 14 m height, there was a renewal subsidence of several meters in 1957, repeated refilling eventually left the embankment at a height of 9 m and the road was opened for use in 1957. In February 1964, following a month of 141 mm of continuous heavy rainfall, the embankment suffered a 7 m vertical and 11 m horizontal slide, along a 150 m stretch. This incident resulted in the road being abandoned. Fig. 2 shows cross section of different subsidence at site of Na'ur Landslide No. 4.

Currently the Ministry of Public Work and Housing (MPWH) in Jordan, is constructing a new Highway passing through the area (Fig. 1). To avoid Na'ur landslide No. 4, a new alignment is selected. The new alignment (under construction) lies at a higher level than the area of the landslide, and has many curvatures which make it very difficult for the traffic to pass through, especially large trucks.

Since 1958, different researchers and many firms such : Reuf (1964); Saket (1970); Harris-Western and Arabtech and Associates (1972); Saket (1975); Geotechnical Engineering and Materials Testing Co. (1983, 1987 and 1989); Arup and Partners (1987); and Parsons Brinckerhoff (1987), investigated this landslide upon request by the MPWH. This paper summarizes and compare results of all these investigations. Moreover, the possibility of reconstructing the old failed embankment is addressed and analyzed from a geotechnical point of view and compared economically to the implemented solution of the road realignment.

Surface Hydrology

The average annual precipitation (measured over a 50 years period) in the study area is 482 mm. This rainfall is usually concentrated in the period between November and May. The annual precipitation during the period between 1942 and 1990 is illustrated in Fig. 3. There are no perennial flows in the wadis at the landslide area. The discharge is limited to the seasonal rainfall.

Precipitation water penetrates through the joints in the bedrock and becomes perched on the subsurface clay horizons, therefore numerous springs can be observed between Na'ur and Adasiyeh. Most of them are dry in summer which confirms the rather limited extent of the aquifers.

The drainage of the study area is mainly towards the west and the

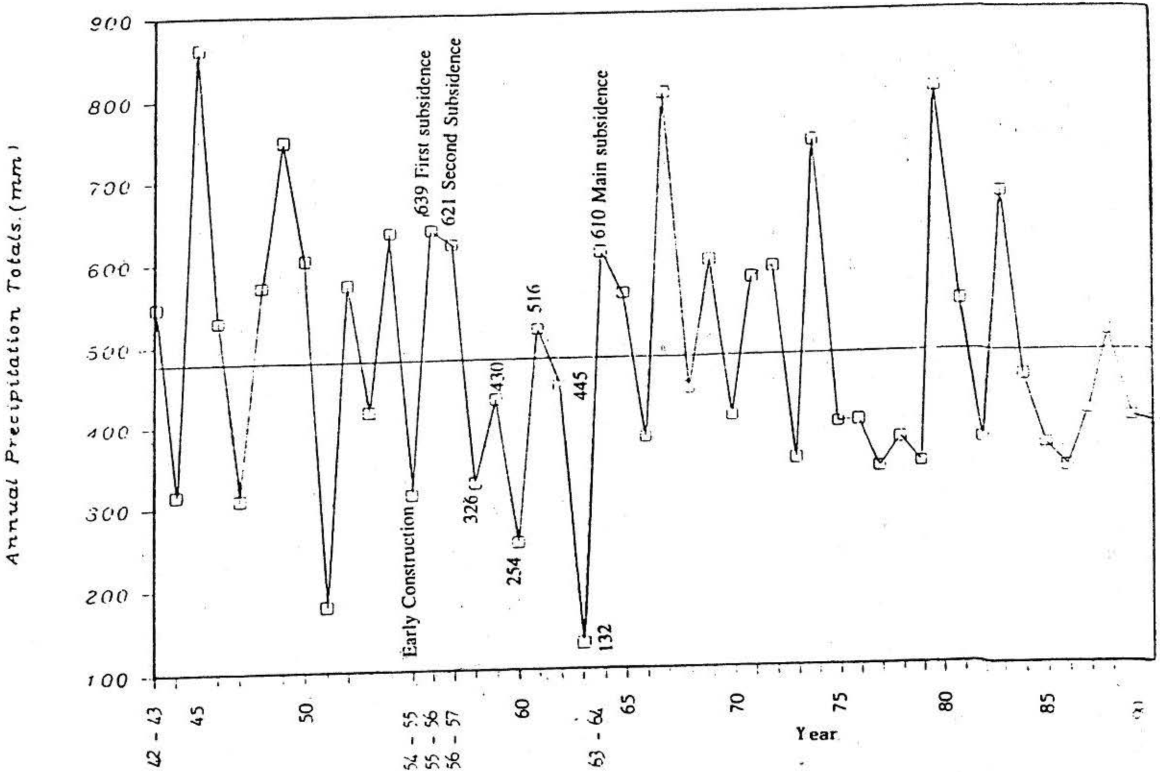

FIGURE 3 Rainfall Records at Na'ur Station

wadis are tributaries of Wadi El Kafrein. The slope is incised by several valleys and gullies directed towards the main landslide area and then drain into a relatively major seasonal stream of steep gradient (Umariya Valley).

The relationship between annual rainfall and slipping is clear. The early stage of embankment construction was in 1955, a dry year with an annual rainfall of just about 300 mm. When the failures occurred in 1956 and 1957 the amount of annual rainfall was over 600 mm and 500 mm, respectively. The period when the road was in use coincided with several dry years in succession, terminated by the wetter years of 1963 and 1964, when failures reoccurred again. It is clear therefore that the old landslide was capable of supporting an embankment under dry conditions.

Geomorphology

The study area lies within the mountain ridge, east of the Jordan rift valley. The development of the rift valley had led to continual rejuvenation of streams draining the highlands to the Dead Sea, thus giving rise to steep sloes and continual active wadi erosions.

Na'ur Landslide No. 4 occupies the western slope of the wadi running approximately South-North, this slope is characterized by a chaotic, stepped profile having a vertical or undercut rear scarps. It is descent by the heterogeneous series of impersistent rock scraps, gullies, scree and soil-rock flows with the scarps becoming less frequent towards the south. In places, the area is strewn with boulders.

The study area can be geomorphologically divided into different subareas showing various topographic stages, and different landscapes.

From the west to the east, the area can be divided into the following geomorphologic subareas (Fig. 4) :

Subarea 1 : This is the site of the limestone and marl slide forming a major gully.

Subarea 2 : It represents the rear limestone scrap which is vertical to undercut rock face of limestone and marl.

Subarea 3 : Immediately below the rear limestone scarp, an upper ledge is encountered. it consists of very rough, irregular terrain strewn with boulders and blocks up to 8 m × 6 m × 6 m in size. These blocks and boulders are underlain by marl.

Subarea 4 : It comprises foundered limestone outcrops.

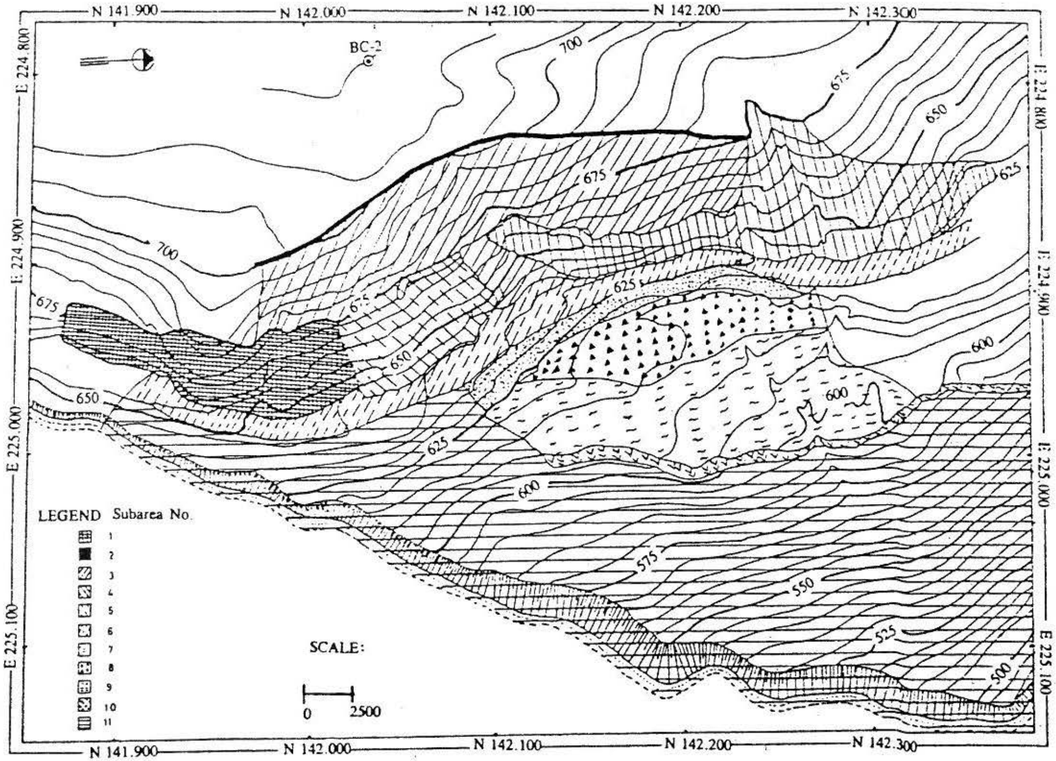

FIGURE 4 Geomorphological Map of Na'ur Landslide No.4.

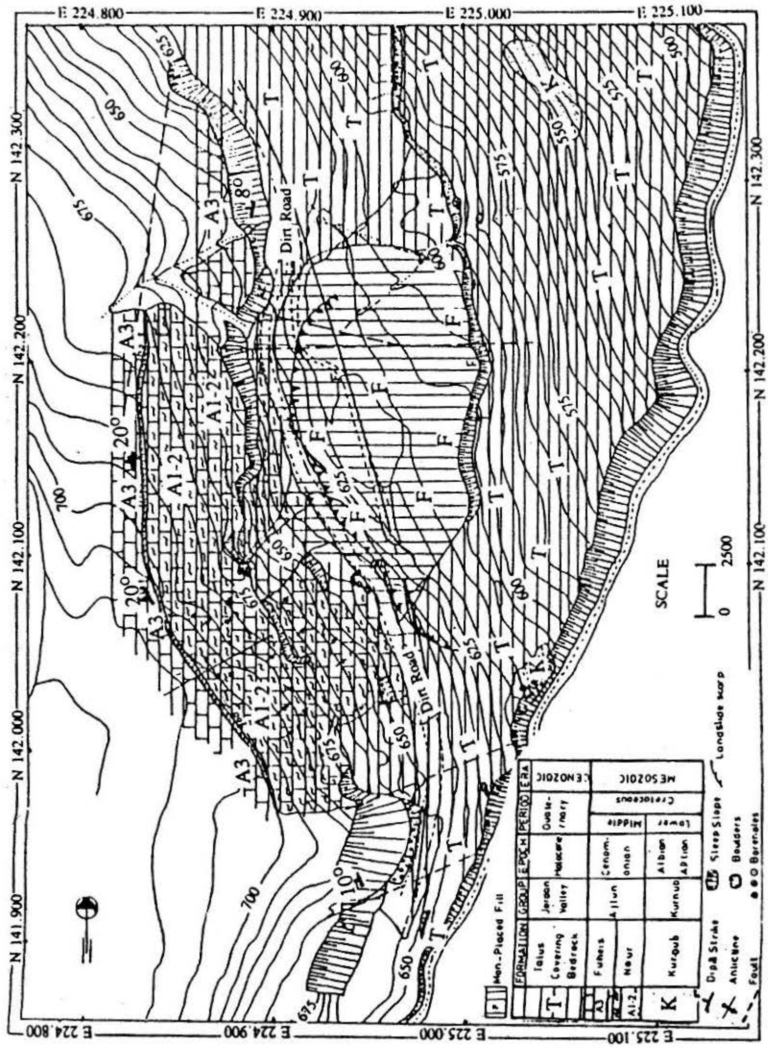

FIGURE 5 Geological Map of Na'ur Landslide No.4

Subarea 5 : This subarea represents a steep slope of marl and limestone debris.

Subarea 6 : It is relatively flat and encompasses the access road which traverses the landslide.

Subarea 7 : This subarea slope steeply eastward and consists of limestone blocks and boulders with marl.

Subarea 8 : A relatively flat area. It is the upper surface of the failed embankment material.

Subarea 9 : A gently slopping area *consisting of the failed embankment* fill. It has irregularly hummocky ground with steps and terraces up to 2 m in height.

Subarea 10 : This subarea consists of a very steep, largely bare, unstable slope of marl and limestone fill strewn with boulders. The toe of the failed embankment lies within this erosion subarea. Erosion of this toe may cause continued movement of the embankment fill.

Subarea 11 : The toe of the embankment overlies this subarea which consists of the steep slope of marl and limestone debris with large foundered limestone blocks.

Geology

Stratigraphy

In the vicinity of Na'ur landslide No. 4, the outcropping rocks consists of alternating limestone, dolomitic limestone and marls of the lower Ajlun Group (A1-2, A3) of Middle and Upper Cretaceous Age. These rocks overlie the Kurnub Sandstone Group (K) of Lower Cretaceous Age. Where rocks are not exposed, a surface cover of weathering products is present (Fig. 5). The geological sequence of the area of Na'ur landslide No. 4 is shown on Table 1.

Subsurface Geology

The area of landslide No. 4 has local variation in soil and rock quality and characteristics. From the different boring made in the area of the landslide (Fig. 6), the subsurface materials encountered can be divided into three generalized strata. These strata vary in thickness from one borehole to

TABLE 1
Geological Sequence of the Studied Area

Time Unit			Time Rock Unit	Rock Unit		Rock Type Lithology
Era	Period	Epoch		Group	Formation	
Cenozoic	Quaternary	Recent	Pleistocene to Holocene	Jordan Valley and Plateau Gravel	Superficial Deposits	Clay Silt Sand Gravel Boulders
Mesozoic	Cretaceous	Cenomanian Turonian	Turonian	Ajlun	A7	Limestone
			Cenomanian		A5-6	Marly Limestone
		Lower Cretaceous	Albian		A4	Limestone and Dolomite
				Kurnub	A3	Limestone Interbeded with Marl
					A2	Limestone
					A1	Marl
					K	Sandstone

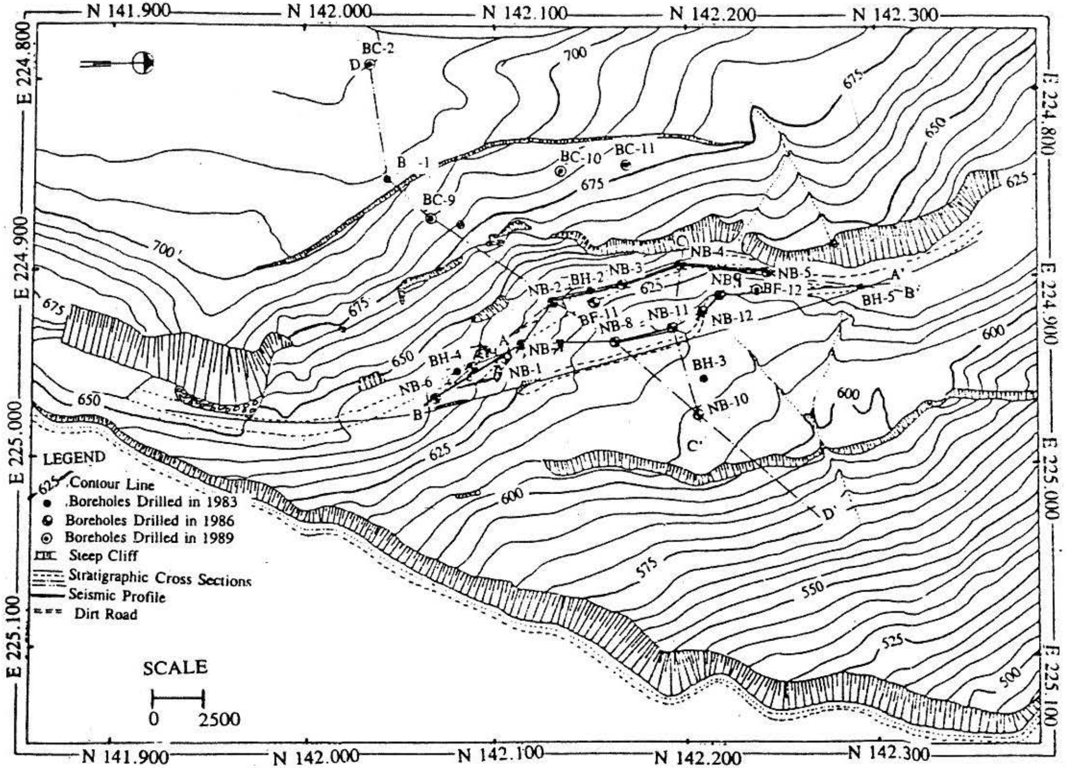

FIGURE 6 Detailed Location Map of Borholes Drilled at Site of Na'ur Landslide No.4

the other depending on its location in the site. These variations are shown on Figs. 7 through 10.

Structural Geology

Landslide No. 4 is bounded to the east by the wadi which follows the approximate course of a north-south trending fault, to the west by the rear limestone scrap of the A3 Formation and to the south by an ENE–WSW trending fault in the area of Grid E 224°.950′ N 141°.930′. The landslide area extends north to approximately N 142°.930′.

The area within the described boundaries is a small open anticline gently plunging to the west. The fold is clearly seen in the limestone outcrop of the rear scarp located at the western boundary. This outcrop appears to be in situ and is used as a geological reference for the remainder of the outcrop below. A traverse eastward from the rear scarp, down across the slope, indicates a series of laterally impersistent limestone scarp with interbedded marl which are poorly exposed. The scarps are often boulder-strewn, or are covered with marl and limestone scree.

The regional dip of the strata of the rear scarp is 8 to 10 degrees into the slope and between WSW to WNW in direction. The direction of the dip and strike of the limestone outcrops above the existing road are anomalous when compared with the in situ rear scarp. It appears, therefore, that the limestone outcrops lying downstream the rear scarp are not in situ.

Changes in joint direction within the blocks is referred to their independent sinking and slipping on softer marl horizons. This makes it appears as if local faulting has occurred.

Below the limestone and marls, both to the north and south of the landslide area, outcrops of Kurnub Sandstone are present. The sandstone is unaffected by the movement activity described above. It defines the lower geological boundary of the area of interest, but is obscured by old landslide debris and fill. Evidence from 1953 aerial photographs, suggest that the sandstone outcrops may be embayed in the vicinity of Grid E 224°.980′ N 142°.230′. The embayment may be caused by the intersection of the north–south wadi fault with an east-west trending fault at the northern end of the anticline. Along much of the wadi and the toe of the old landslide, however, the debris, which embays 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 environments may also have been responsible for reactivating landslide movement.

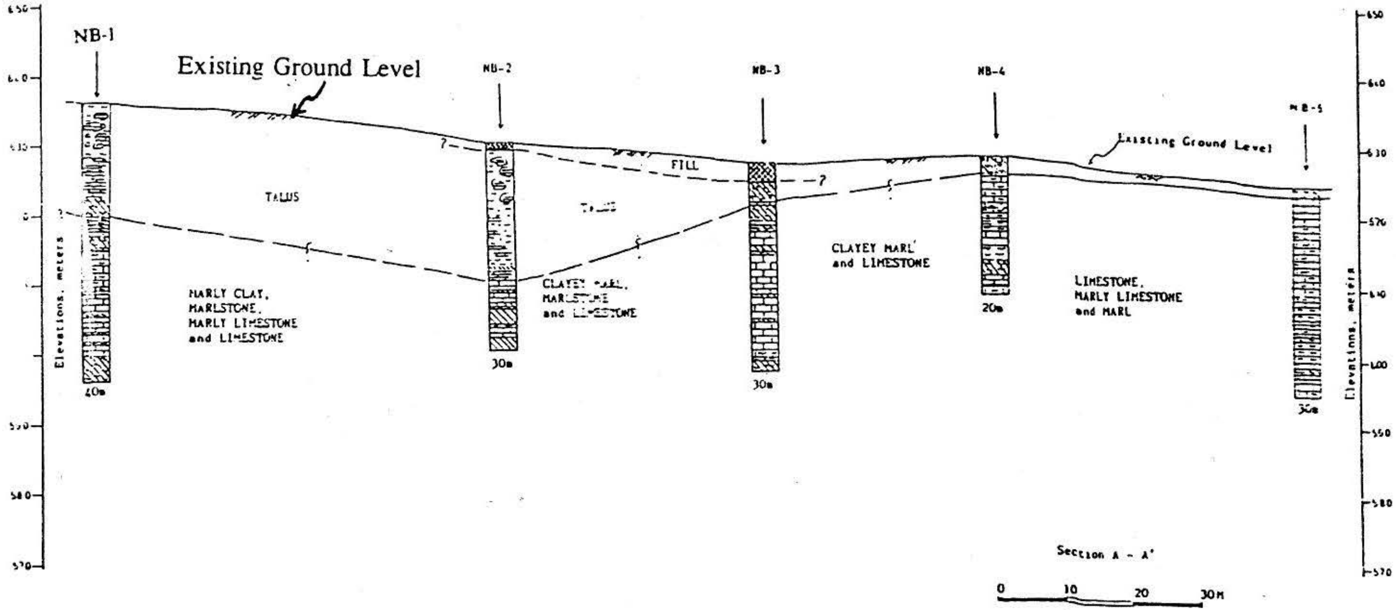

FIGURE 7 Subsurface Cross Section between Boreholes Drilled in Studied Area (Boreholes NB-1, NB-2, NB-3, NB-4, and NB-5,

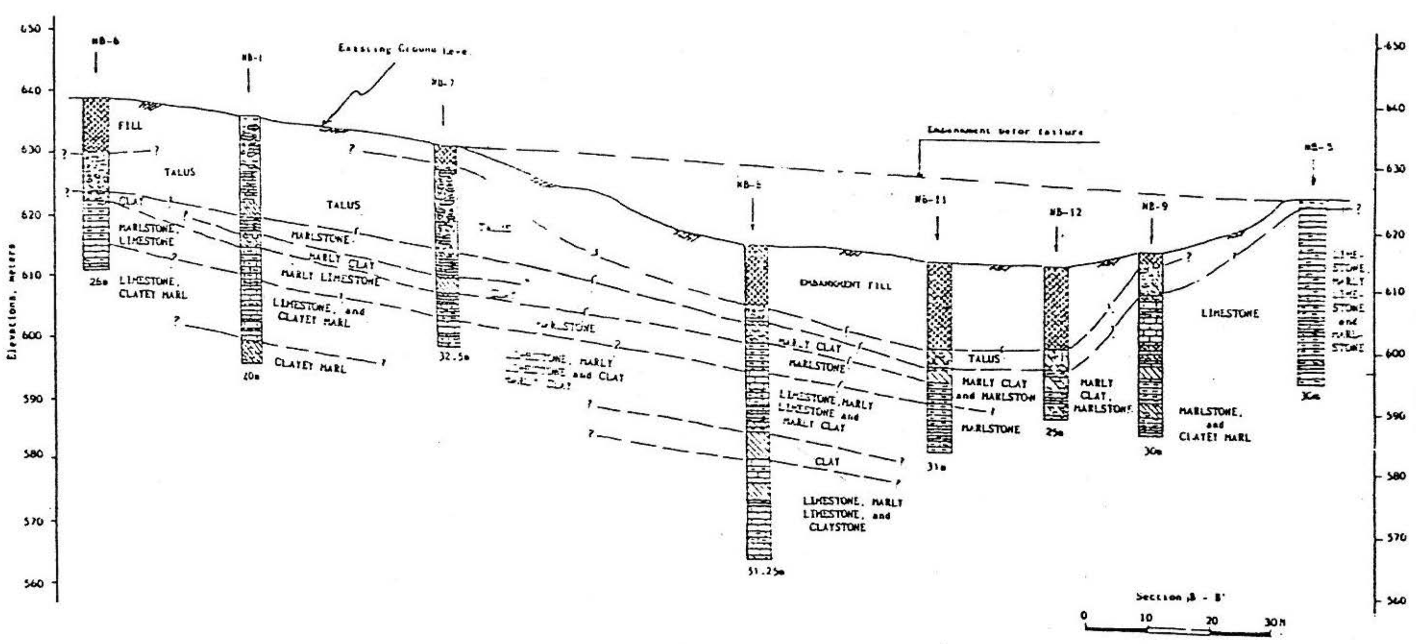

FIGURE 8 Subsurface Cross Section between Boreholes Drilled in Studied Area (Horeholes NB-6, NB-1, NB-7, NH-8, NB-11, NB-12, NB- 9, and NB-5).

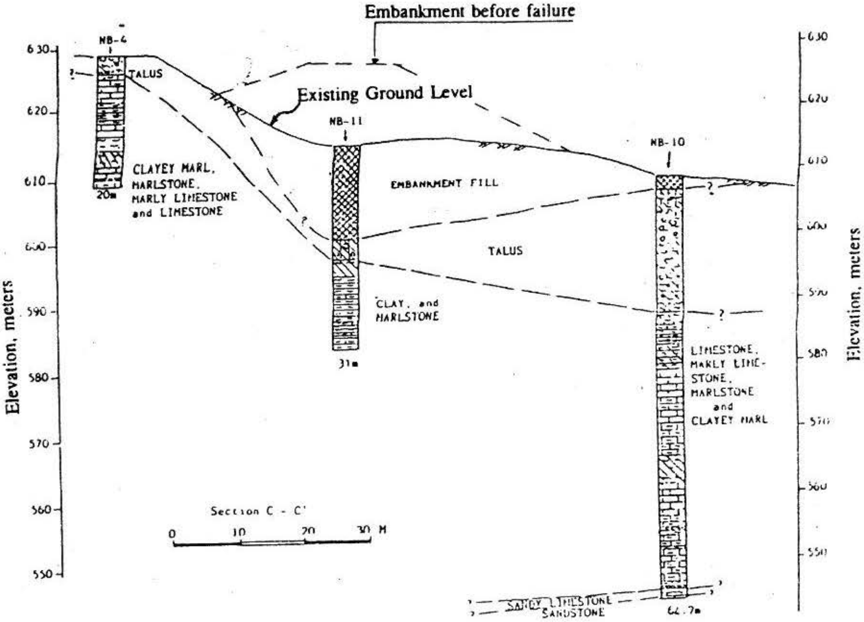

FIGURE 9 Subsurface Cross Section between Boreholes Drilled in Studied Area (Boreholes NH-4, NB-11, and NB-10).

Engineering Geology

It was apparent from the outset of the geotechnical mapping that the area where natural outcrops exist was a site of extensive earlier major instability and wide variety of gravitational mass- movement terrain. Human activities on the site, such as the construction of the improper road embankment, was an effective triggering mechanism of the failed embankment.

Many types of gravitational mass movements were recognized as failure mechanisms, which are essentially superficial, as a results of the lithological and structural characteristics of the rocks and marls, and of the steepness of the valley side. Following is a listing of movements observed in the area:-

A. Natural Ground Movements

These movements occurred in the natural materials of the landslide area and they can be divided into different types and modes of failures.

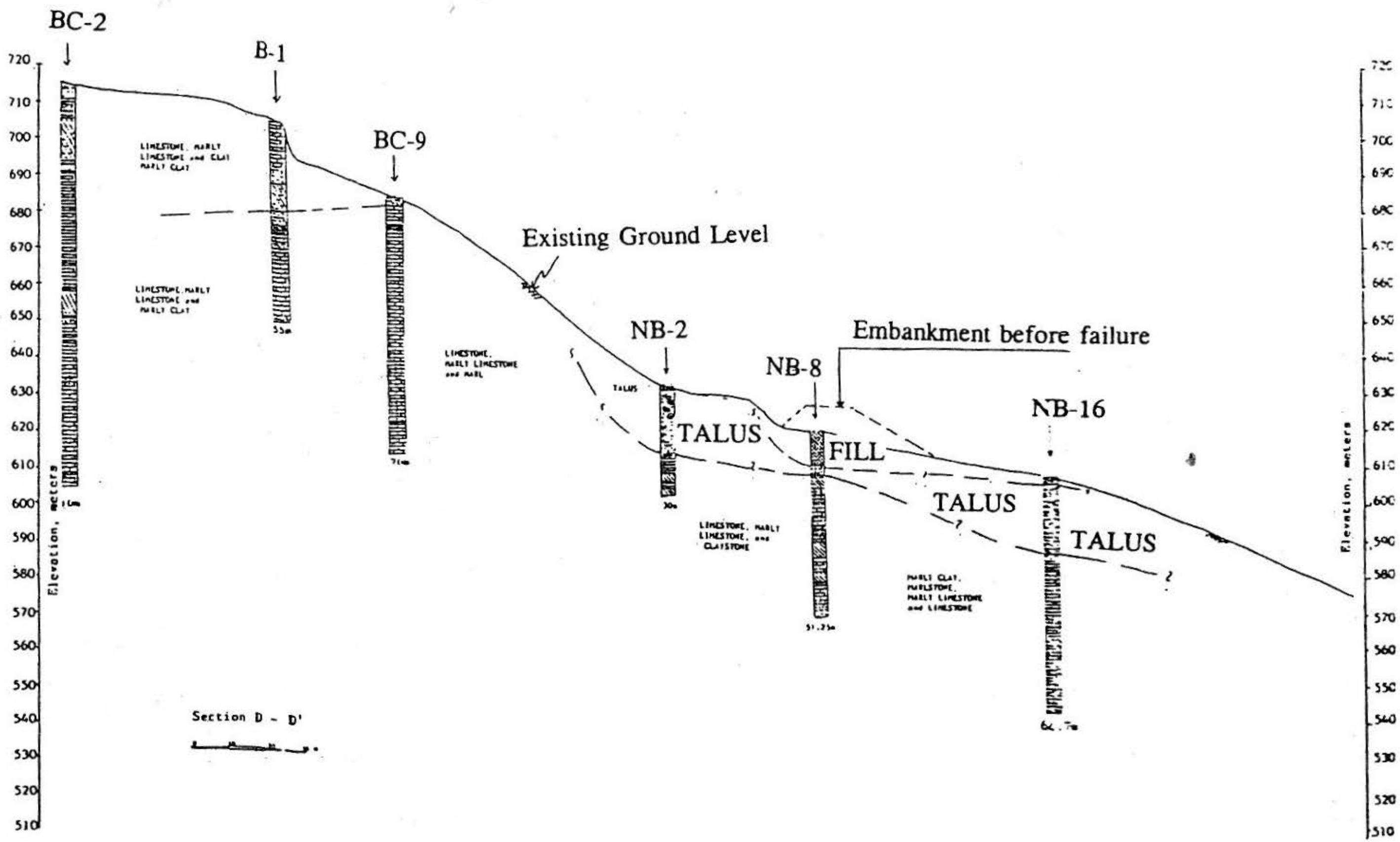

FIGURE 10 Subsurface Cross Section between Boreholes Drilled in Studied Area (Boreholes BC-2, B-1, and BC-9, NB-8, and NB-16)

These are as follows.

Gravitational Mass-Movements

1. Movements by slippage along plans of weakness

Rocks Falls : Subarea 2 (Fig. 4) include rock fall features and cliff scarps of the limestone and the other competent beds of the Middle Cretaceous Ajlun Group A1-2 and A3 Formation. Movement of rocks in this category resulted when materials at the base of the escarpment are removed during the slide, thereby leaving the overlying rock layer unsupported. The main features characterizing the rock falls are the tensional cracks and open joints behind the cliff scarps.

Debris Falls : As in the rock falls, debris falls are present in the studied area forming escarpment of loose materials and incompetent rocks of the same mentioned formation, and in a similar mechanism. This type of movement is obviously seen in subarea 11 (Fig. 4)

Block Slides : Block slides are well established in subarea 3 (Fig. 4) of the landslide. Due to the sub-aerial weathering processes, which lead to lithification, jointing and fracturing, the rocks tend to separate into blocks and slide as units across the landslide area.

2. Movement by Internal Deformation

Earth and Debris Flows : Regionally the whole landslide can be classified as an earth and debris flow, however, locally the flows of this type are well recognized in subarea 11 (Fig. 4) of the landslide area. The main reasons behind this movement is the variability in water content and the wide range of particle size, from clay through large boulders of the unconsolidated materials, also due to existence of the weathered limestone and weak marly layers of the bedrock.

The main characteristic features of this type in the area, are the hummocky topography, tensional cracks, piping, transverse depressions, building effect, concave scarps and cracking.

Mode of Failures

1. Large Mass Failures

Translational Failure : This type of failure extends from about N 142 000 northwards. Large blocks of limestone foundering and slipping on the marly layers is the main mechanism of this failure, the main processes which led to this failure are as follows :

- ❖ Surface water seepage to the interbedded marly layers which increased the weight of the layers, increased the pore pressure and caused softening and a change in the physical properties of these layers resulting in a decrease in their shear strength.

- ❖ Mechanical erosion of the surface which caused removal of the lateral constraint from the clayey marl layers.

- ❖ Man-made cuts at the toe of the landslide which produces the same effect as the previous process.

All these processes decrease the stability and possibly lead to reactivation of this mode of failure. Driving mechanism of this type of failure is the heavy weight of the limestone blocks.

Topping Failure : Toppling is a localized, smaller version of the multiple regressive failure. The main cause of this failure are as follows :

- ❖ The joints bounding limestone columns within a foundered block, which are sinking and rotating on the underlying marls and clays in the direction of the slope face.

- ❖ Ingress of water softens the underlying marls and clays which accelerates the failure.

- ❖ Trapped water in the open joints increases the pore pressure and therefore horizontal forces which are likely to initiate the rock fall.

Toppling failure is responsible for much of the near surface joints. It increases with increasing height of the limestone face and with decreasing joint spacing. The area of zone 6 through zone 9, zone 2 and the rear limestone scarp are susceptible to the topping failure.

2. Single Block Failures

Plane Failure : Plane failure possibility exists in the area of zone 7 and zone 9, the main causes of this failure are as follows:

- ❖ Where the slope face is inclined at an angle steeper than the joint plane allowing the joint to expose in the face and slip to occur when the shearing resistance on the joint plane is exceeded.

- Ingress of water into the slope reduces the stability by increasing the water pressure action on the Joint surfaces behind the face.

Wedge Failure : This mode of failure is found to characterize all the outcrops of the strong limestone rocks in the landslide area. The most controlling factor of this failure is the intersection of two joint sets conductive to the development of sliding, however, only small block volumes are involved in this failure. This failure mode is considered to be superficial in nature since only small block volumes appear to be involved.

B. Man-Placed Embankment Movements

This man-made embankment has failed several times after construction. Many remarks can be made on the failure of this embankment, these are as follows :

1. During construction the embankment was sited on sloping ground probably with no benches or steps keyed-in to the slope.

2. The embankment was sited on ground containing landslide debris.

3. From the available evidence, it is known that the failure is not deep-seated, and it does not extend much below the materials boundary that existed prior to first time construction.

4. During construction of the embankment, driving stress increased and the pore pressure increased in the soils at some saturated zones within the embankment materials itself due to the high stress from the weight of the embankment fill therefore reducing the shear strength leading to failure process within the embankment body.

5. Heavy rainfall has direct effect on activating the landslide, therefore, a certain relationship with periods of rainfall can be made, due to the intermittency of the embankment movements.

6. Surface and subsurface drainage of water within and around the embankment materials, both during and after construction, have not been taken into consideration, this has given the opportunity for water to enter the ground and increases the weight of the soil and to have an adverse effect on the shear strength of certain zones of the embankment and the interface between the fill and the natural ground or in the landslide debris.

7. The man-made embankment was never stable, due to the poor foundation materials and lack of understanding of the failure mechanism within the materials.

Field Investigations

Drilling

The area of Na'ur landslide No. 4 has been studied and investigated several times. The first borehole drilled in the area was by Harris Western in 1972. This hole was drilled to 40 m depth. At the road level, just north of the main landslide, a piezometer was installed in the boring, but no follow-up in subsequent seasons was made. The material encountered during boring consists of limestone, claystone and some shale, high fracturing and permeability of the subsurface materials was indicated by loss of circulation in the drilling water.

Geotechnical Engineering and Materials Testing Co. (G.E.M.T.) has studied this landslide in the years 1982, 1987 and 1989. Several boreholes were drilled during these periods. The locations of these holes are shown in Fig. 6.

In 1982, five boreholes were drilled (indicated in Fig. 4 as B.H. 1 through B.H. 5). The maximum depth reached in these holes in 55 m.

In 1987, twelve boreholes were drilled in the area (indicated in Fig. 4 as NB 1 to NB. 12). The deepest boreholes was 64.7 m deep, where the Kurnub sandstone was encountered.

In 1989, the landslide area was investigated for the last time to give a decision on reconstructing the road passing through the area. Seven boreholes were drilled within the area indicated in Fig. 6 as B.C. 2, B.C. 9, B.C. 10, B.C. 11, B.F. 10, B.F. 11 and B.F. 12. The maximum depth reached was 110 m (i.e. B.C. 2). This borehole was drilled at the highest level of the landslide area.

The boring were drilled with rotary drilling rigs generally using the air flush rotary method of drilling. In order to obtain bulk samples of fill and talus deposits. Due to the high fracturing of bedrock 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 boring were recovered using sampling tools.

Geophysical Studies

A number of seismic profiles were made across the landslide area, using the 1570C Signal Enhancement Seismograph.

Seismic profiling was performed at different locations (Fig. 6), one at the upper dirt road, and the second was below (to the east) the failure scrap to investigate the different strata. For all the profiles, travel-times of the first P-waves arrivals were measured from every shot, P-wave velocities and layer thicknesses were calculated from the T–X graphs (the travel-times versus their respective horizontal distance). The velocity was taken as 1/slope of the defined linear relationship. Three different subsurface layers (strata) were encountered (Fill, Talus and Bedrock).

For the fill and talus, the velocity range was between 0.385 km/s to 0.635 km/s. These low velocities are typical for loose and uncompact materials. Thickness of these layers at the location of the different profiles were from ground level to 6 m.

With increasing depth below 6m the seismic velocity increased and the range was 0.986 km/s to 1.583 km/s. This velocity represents generally weathered bedrock or highly compacted materials as well as the existence of the boulder content (talus).

Due to the highly fractured, jointed and weathered character of the upper bedrock, especially near the bedrock talks interface, the velocities were relatively low: 0.183 km/s for marlstone and limestone. An anomalous velocity of 3.929 km/s was registered at 11 m and 13.5 m depth which indicates the presence of a large compact bedrock.

A summary of the seismic results is given in Table 2.

Permeability

In situ permeability testing was performed to estimate the permeability of the fill, talus and the natural bedrock. Two types of tests were carried out in borings, these are pressure type test and Gravity type test.

pressure permeability (Packer) tests were carried out in boreholes NB 1, NB 3, NB 6, NB 7, NB 8 and NB 9 (Fig. 6). This test was conducted mainly in the natural bedrock. The range of permeability results was between 6.1×10^{-4} cm/s and 1.2×10^{-4} cm/s (Lugeon values from 62.8 to 787.7), these values are considered high and are caused by fracturing of bedrock. This fracturing is recognized by the low Rock Quality Designation (RQD) in the tested sections which range between 0% to 20%. A summary of the results of pressure type permeability tests are shown in Table 3.

Gravity permeability tests were conducted mainly within the fill and talus materials, and were made in boreholes NB 8 and NB 11 (Fig. 6). The range of permeability results was between 1.3×10^{-3} cm/s and 7.3×10^{-5} cm/s. These values are considered high which indicates that the surface water was always penetrating through these layers and reducing the shear strength along the materials of the surface where sliding occurred. A summary of the results of the permeability gravity type tests are shown in Table 4.

Laboratory Investigations

Most laboratory works were conducted at G.E.M.T. laboratories in order to determine the physical and engineering properties of materials encountered in boreholes. The results of the physical and engineering tests are summarized in Tables 5 and 6.

TABLE 2
Summary of Seismic Profiling Results

Depth Range (m)		Average Seismic Velocity (m/sec)	Borehole Numbers	Comments
From	To			
0	2	385	NB-2, 3, 4, 6, 7	Loose Debris (Talus or Fill), Silty Clay with Gravels
0	4.5	530	NB-8, 9, 11, 12	Talus-moist as compared to 0 to 2m
2	4	635	NB-6, 7	Talus-moist as compared to 0 to 2m
2	4	986	NB-2, 3	Talus, moist more compacted compared to 0 to 2m range
4	8	1583	NB-2, 3, 4, 5, 9,12	Highly weathered and fractured Marlstone, Limestone. Includes Talus deposits
8	10	1290	NB-6, 7, 8, 11	Boulders and Gravels of Rock and Marly Clay. Includes Talus

TABLE 3

Summary of Permeability Pressure Test Results

Borehole Number	Permeability Test Section Depth (m)		Description	Rec %	RQD %	Yield Pressure kg/cm^2	Lugeon** Value	Coefficient of Permeability cm/s
	From	To						
NB-1	11.5	15.0	Talus : Blocks of Limestone with Chalky Marl			1.4*	201	2.2 E-3
	18.5	22.0	Marlstone Cleyey Marl and Marly Limestone	95	0	1.7	80.7	8.9 E-4
	25.0	30.0	Fractured Limestone and Marly Limestone	85	0	2.9*	76.5	9.2 E-4
NB-3	17.2	20.0	Highly Fractured Limestone with Cleyey Marl	100	0	2.0*	242.4	2.5 E-3
	27.2	30.0	Marly Limestone intercalated with Marly Clay	100	10	5	62.8	6.2 E-4
NB-6	23.1	26.0	Fractured Limestone	100	11	4.5*	122.6	1.2 E-3
NB-7	24.0	25.0	Fractured Claystone	100	20	4.6	142.2	1.1 E-4
NB-8	14.5	15.5	Marly Clay	100	20	2.6	251	4.4 E-4
	28.5	30.0	Marlstone and Clay	95	20	3.0*	280	2.4 E-3
NB-9	13.5	14.5	Highly Fractured Limestone	94	0	1.5*	787.7	6.1 E-4
	29.0	30.0	Highly Fractured Marlstone	77	0	4.0*	262.5	2.0 E-3

* Indicates there was no build up of pressure during the test.

** A Lugeon value of 1 is defined as water leakage of 1/min per one meter of test section.

Stability Analyses

Na'ur Landslide No. 4 has been studied by many geotechnical firms for the last 20 years. In these studies, stability analyses were carried out to explain the reasons behind the instability and evaluate remedial measures. Following is a summary of main studies.

Previous Stability Analysis Studies

Saket (1975) carried out stability analysis for the area using two methods, Fellenius (1936) and Janbu (1954). He assumed a non circular slip surface passing through the marly clayey material and tangential to the limestone bedrock. Two case were analyzed :

Case 1 : Stability prior to 1964 failure

Case 2 : Stability of configuration after 1964 failure.

Geotechnical Engineering and material Testing Company (G.E.M.T.) (1983) analyzed the stability of the slope using the Morgenstern and Price (1965) method. The stability analyses were made for the present configuration of the slope only, assuming different slip surfaces and various soil parameters. Three cases were analyzed :

Case 1 : Sliding on fill or talus/ in-situ bedrock interface with a failure plane reaching to 40 meters depth below existing ground level.

TABLE 4
Summary of Permeability Gravity Test Result

Borehole Number	Depth	Description	K cm/sec
NB-8	6m	Fill : Silty Clay and Chalky Marl with Gravels	4.5E-4
	10m	Fill : Silty, Marly Clay with Gravels	3.7E-4
NB-11	5m	Fill : Gravels and Boulders with Silty Clay and Chalky Marl	7.3E-5
	10m	Fill : Silty Clay with Gravels	5.5E-3
	15m	Talus : Silty, Marly Clay with Gravels	1.3E-3

TABLE 5

Summary of Classification Test

Test		Type of Material					
		Fill	Talus	Clayey Marl	Clay	Marl Clay	Marl
(1)		(2)	(3)	(4)	(5)	(6)	(7)
Unit Weight (g/cm3)		2.179	2.289	2.269	2.339	2.186	2.182
Water Content (%)	min	2.90	0.70	3.6	8.2	7.5	2.6
	max	20.0	17.0	353.7	34.6	25.4	15.9
	Avg	8.3	5.98	14.0	16.4	16.3	9.12
Dry Unit Weight (g/cm3)	min	1.88	1.94	1.40	1.74	1.52	1.79
	max	2.28	2.43	2.58	2.68	2.16	2.27
	Avg	2.02	2.16	1.99	2.01	1.88	2.00
Specific Gravity	min	2.69	2.68	2.54	2.56	2.55	2.57
	max	2.85	2.75	2.84	2.8	2.81	2.59
	Avg	2.7	2.7	2.7	2.7	2.7	2.6

TABLE 5 Contd.

(1)		(2)	(3)	(4)	(5)	(6)	(7)
Passing Sieve No. 200	min	32	44	63.4	51.1	69	
	max	61	91	100	100	100	
	Avg	50	60	92	90	93	
Liquid Limit LL (%)	min	27	20	33	45	48	43
	max	60	50	91	106	88	59
	Avg	34.6	32.5	55	72.9	70.5	52
Plastic Limit PL (%)	min	15	14	17	22	22	25
	max	22	22	40	47	38	27
	Avg	18.5	17.6	24.9	31.3	29.7	26
Plasticity Index PI (%)	min	9	2	12	23	26	17
	max	40	27	52	70	60	34
	Avg	16.2	14.9	28.9	41.1	41.3	26

TABLE 6

Summary of Average Shear Strength Test Results

Material	Undrained Strength			Effective Strength Parameters from CD Tests				Effective Strength Parameters from CU Tests			
	UC kg/cm^2	Undisturbed UU kg/cm^2	Remolded UU kg/cm^2	Undisturbed ϕ (degrees)		Remolded ϕ (degrees)		Undisturbed		Remolded	
				Peak	Residual	Peak	Residual	c (kg/cm^2)	ϕ (degrees)	c (kg/cm^2)	ϕ (degrees)
Talus			24.12			33	30			0.0	32.8
Clay, Clayey, Marl and Marl Clay	5.888	6.98	3.64	37	25	28	18				
Claystone and Marlstone		104.95	30.52								
Marly Limestone	235.09										
Limestone	264.5										
Clay								4.2	22.8	0.0	22.8
Clayey Marl								28.0	35.7	0.0	35.7

Notes : UC – Unconfined Compressive Strength
UU – Unconsolidated Undrained Triaxial Compression Strength
CU – Consolidated Undrained Triaxial Compression Strength
CD – Consolidated Drained Direct Shear Test
ϕ – Effective Angle of Friction
c – Effective Cohesion

Case 2 : Sliding on fill or talus/ in-situ bedrock interface with a failure plane reaching to 50 meters depth below existing ground level.

Case 3 : Sliding on fill or talus/ in-situ bedrock interface with a failure plane reaching to 70 meters depth below existing ground level.

Parsons Brinckerhoff International (1987) conducted a series of stability analyses of a cross section through the failed embankment area. Slope stability analyses were performed on a representative cross section using the two-dimensional limiting equilibrium method, for both sliding wedges ("blocks") and circular arcs. The computer program ST ABL5M (Siegel, 1975) was used in the analyses.

Two sliding surfaces were assumed, sliding on fill/talus interface and sliding on talus/insitu clay layer interface. Saturated unit weights were used for the fill and talus. No water table was assumed in the analyses. Five cases were analyzed as follows :

Case 1 : Prior to 1957 failure, sliding on fill/talus interface.

Case 2 : Prior to 1957 failure, sliding on in-situ clay layer interface.

Case 3 : Partially rebuilt embankment, 1961 status (which failed in 1964), sliding on fill/talus interface.

Case 4 : Present configuration of embankment, which has remained essentially unchanged since 1964 failure, sliding on fill/talus interface.

Case 5 : Present configuration of embankment, sliding on talus/insitu clay layer interface.

The authors believes that analyses conducted by different investigators are reasonable. Table 7 shows a comparison between the three stability analyses studies.

Considering the geometry of the sliding mass, all previous studies suggested that the movements occurred along a non- circular failure surface, translational type or with scalloping mechanism. Moreover, all the modes of failure which were considered in previous stability analyses studies assumed that the movements occurred at or above the bedrock surface.

Water table was assumed in the analyses by Saket (1975) and G.E.M.T. (1983) but no water table was assumed in the analyses by Parsons and Brinckerhoff International (1987). However, saturated unit weights for the materials were used in all the previous studies, because it was agreed that the movements and failures occurred after periods of heavy rain.

TABLE 7

Comparison Between the Previous Stability Analysis Studies for Na'ur Landslide No. 4

	Previous Study		
	Saket 1975	G.E.M.T. 1983	Siegel, Parsons Brinckerhoff 1987
(1)	(2)	(3)	(4)
Stability Analysis Method Used	Fellenius and Janbu	Morgenstern and Price	Sliding Wedge and Circular Arc
Geometry of Failure	Non Circular	Non Circular	Non Circular and Circular
Range of Soil Parameters			
c kg/cm3	>0 to 0.2	0 to 13.0	0.1 to 0.420
ϕ degrees	10 to 15.0	12 to 25.0	9 to 23
Wet Density g/cm3	1.89 to 2.03	1.800 to 2.000	2.94 to 3.068
Water Table	With and Without	With and Without	Without
Sliding Surface Depth	20m to 25m	40m to 70m	10m to 30m

TABLE 7 Contd.

(1)	(2)	(3)	(4)
Stability Cases	– Prior to 1961 – Existing Condition	– Existing Condition	– Prior to 1957 – 1961 Rebuilt Embankment – Existing Condition Status
Factor of Safety for Nearly Similar Conditions			
1. Prior to 1964 Movement	1.33 and 1.28		1.99 and 1.74
2. Existing Condition	0.95 and 0.85	1.85	1.59
Main Recommendations and Conclusions	1. The Existing Slope is Stable 2. Drainage is a must 3. The Upper Slope Should be Flattened 4. The Road Construction is feasible	Same as Saket, 1975	Same as Saket, 1975

ϕ : Effective Angle of Friction

c : Effective Cohesion

Effective strength parameters used in all previous stability analyses studies were variable. The ranges of effective residual cohesion (c′) and friction angle (ϕ') values were zero to 0.14 kg/cm^2 and 9 to 12 degrees, respectively, while ranges of peak cohesion (c′) and friction angle (ϕ') values were 0.2 to 0.42 kg/cm^2 and 12 to 25 degrees, respectively.

The residual strength parameters were used mostly in stability cases after the failure occurred assuming that the material usually lost some of its shear strength after failure. The peak strength parameters were used in stability cases prior to failure.

The main conclusions from different studies are:

1. The 1957 embankment failure most probably took place along the fill/talus interface or along the talus/insitu clay layer interface.

2. The shape of the slide mass essentially has not changed since 1964 and therefore has remained stable.

3. There is no evidence of any potential deep-seated failure through the limestones and marlstones of the Lower Ajlun Group down to the top of the Kurnub Sandstone.

4. The present configuration of the slope mass is marginally stable. It is considered stable when the effects of ground water are eliminated. However, the safety factor is reduced due to pore water pressure increase.

The solutions, recommendations, and conclusions of all the previous studies almost match in :

1. The complete reconstruction of the roadway is feasible, using various solutions (e.g. reinforced earth).

2. The fill and talus materials should be removed or reinforced by nailing or grouting.

3. The upper slope should be flattened and excavated prior to any road construction.

4. Construction of a split level carriageway using reinforced concrete wall or a reinforced soil system is possible. This carriage way is to be shifted to the west to reduce the height of structural walls.

5. Surface and subsurface comprehensive drainage system should be provided to divert waters from the slide area.

Remediation and Road Reconstruction

Based on previous studies and assessment of the current condition at the landslide area, it is the authors opinion that the soft materials (fill, talus and the soft bedrock) should be excavated and removed from the landslide area to reach the solid and strong bedrock (limestones and marlstones).

The maximum depth of excavation will not exceed 22 m below existing ground level. The calculated excavation quantities will reach about 400,000 cubic meter along the slided area.

Following the excavation within the slided area a free drained embankment can be constructed on the top of the strong bedrock, this embankment may consist of rockfill of limestone and marly limestones gravels and boulders compacted in layers. A free draining blanket consisting of pure rock is recommended to be placed between the bedrock and the new embankment to guarantee a good subsurface drainage.

The road alignment can pass the slided segment straightly. The maximum height of the embankment will not exceed 35 m. A box culvert should be placed on the bedrock below the reconstructed embankment.

The calculated quantities of rockfill materials needed to reconstruct this embankment along the slided area is about 500,000 cubic meters. The side slope of the embankment is recommended to be 1–vertical to 1.5–horizontal. Fig. 11 shows the details of the solutions to construct the road through Na'ur Slide Area No. 4

Stability analysis is carried out for the above recommended solution. The stability analysis is made using a cross-section taken through the center of the slided area, which is considered the most critical part of the slided area, where the embankment thickness will be greatest. A circular failure surface is considered through the embankment fill and tangential to the strong bedrock. Simplified Bishop method incorporates in the computer code REAME (Huang, 1983) is used in evaluating the factor of safety.

Partially submerged embankment condition is assumed in the analysis, Strength parameters used in the analysis are cohesion $c' = 0\ \text{kg/cm}^2$ and friction angle $\phi' = 35$ degrees for both the fill and the bedrock.

The stability analysis results are presented in Table 8. Figure 11 show the recommended solution for road reconstruction and geometry of critical slip surface.

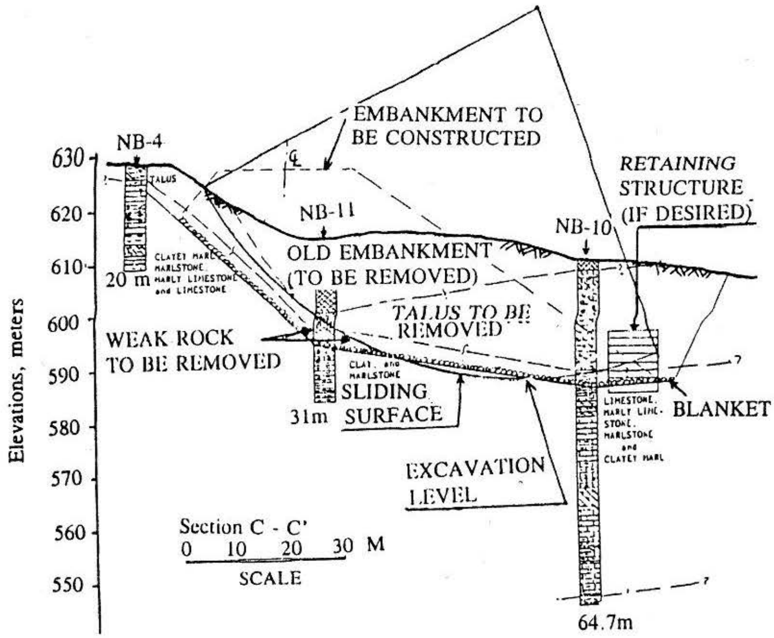

FIGURE 11 Recommended Solution for Road Reconstruction According to this Study (Geometry of Critical Slip Surface is also Shown).

As-Built Road

Although the area at landslide No. 4 was studied several times in the past and many solutions were suggested to pass the road through this area safely, the route was shifted to be west by cutting through the mountain which borders the slided area from the west (Fig. 1). This new alignment was studied once by drilling several boreholes in 1989. The new road is under construction in the present time.

The total length of this road section is about 8 km. The most critical part is the 1 km passing above the area of Na'ur landslide No. 4, because due to the shifting of the road to the west of the slided area, there is cutting into the mountain, causing many problems which started to happen in that 1 km segment. These problems can be summarized as follows:

1. Due to the presence of deep cuts which reached 45 m below existing ground levels, many landslides occurred in this segment. (Station 10 + 400 to Station 10 + 800).

2. As a result of landslides three detours were constructed to keep the road open in case of emergency.

3. The quantities of cut in the segment will reach about 2,200,000 cubic meters. This large amount of excavation need at least two years to be completed. Moreover, very strong limestone beds were encountered in the area, these limestones are being removed in the present time by explosions.

4. The benches of the slide slopes of the deep cuts in this segment are continuously sliding, which requires future solutions.

Table 9 shows an approximate comparison between the As-Built road and the recommended solution in this study (i.e. passing the road through the area of landslide adopting the recommendations mentioned previously).

It is obvious that the road should have passed the slided area causing limited problem and least cost compared to the one being constructed now. It is the authors opinion that the new road will have problems continuously in the future, due to the large interaction and disturbance affecting the nature of that segment of the road.

TABLE 8
Stability Analysis Results of the Recommended Solution

Case No.	Method of Analysis	Soil Parameters		Water Table	Factor of Safety
		c (kg/cm^3)	ϕ (degree)		
1	Fellenius	0.0	35	No	1.820
				With	1.201
2	Fellenius	0.3	35	No	1.996
				With	1.320

ϕ : Effective Angle of Friction

c : Effective Cohesion

Conclusions

Based on the comprehensive review of previous investigations of Na'ur landslide No. 4 and the authors own investigation, it is concluded that :

1. Drilling remarks and seismic results show that the fill and talus deposits in the landslide area re heterogenous uncompacted materials, and the bedrock is homogenous compacted materials.

2. Rock Quality Designation (RQD) values as well as permeability results indicate that the bedrock is highly fractured and the fill and talus are highly permeable materials.

3. The movements which occurred in the upper slop above the embankment in the area of landslide No. 4 were in the lower Ajlun A1-2 Formation of Cenomanian age. This formation is characterized by the presence of clay and marly layer.

4. Quartz and calcite are common non-clay minerals while kaolinite, illite and high expanded mixed layer illite/smectite are common clay minerals, within the clayey marly beds of the natural bedrock.

5. The 1957 and 1964 embankment failures (most probably) took place along the fill/talus interface or along the weak top part of the bedrock. No evidence of any potential deep-seated failure through the bedrock layers was found.

6. The lack of geologic information before constructing the failed embankment led to, placing this embankment on a talus materials which are not stable and are considered geologically as a fossil landslide material.

7. Tectonic activity causing faulting and jointing of the strata in this area played an important role in triggering the sliding.

8. Surface waters (rainfall, runoff water and perched water) during the rainy seasons of more than 600 mm/year were the main reasons behind the failure of the embankment in the landslide area.

9. The erosion of the main wadi (wadi Umariya) which runs at the eastern border of the landslide area have triggered successively the materials in the wadi (talus) initiating the movements.

10. The fill and talus materials are not a good foundation for the new embankment reconstructions.

11. Comparing factors of safety obtained from different methods by different investigators at the same conditions, it can be seen that the Fellenius and Janbu methods gave the lowest values of factors of safety, while the Wedge method gave the highest values.

12. The construction of a road through the area of landslide No. 4 is feasible on the strong bedrock.

13. The new As-Built road which is being constructed at present will have continuous problems in the future.

TABLE 9
Comparison Between the Present As-Built Road and the Recommended Solution in this Study (i.e. Road Passing Through the Sliding Area)

	As-Built Road	Road Passing Through Na'ur Landslide No. 4 Adopting Recommendations in this Study
1. No. of Trees being Cut	4,200	0
2. Quantities of Cut (m3)	2,200,000	400,000
3. Quantities of Fill (m3)	70,000	500,000
4. Grade of Road (%)	8% Upwards and Downwards	4% Downwards
5. Interaction with Nature	High	Low
6. Detours Cost (JD)	450,000	Zero
7. Cost of Cut and Fill (JD)	2,700,000	1,000,000
8. Total Cost (JD)	3,150,000	1,000,000
9. Time of Construction	3 Years	1 Year
10. Stability of Cut and Fill	Low	High
11. Cost of Maintenance after Construction	High	Low

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