Stability of a Heterogeneous Sandy Coastal Cliff

Stabilité d'une Falaise Costale Hétérogène Sablonneuse

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SYNOPSIS The Mediterranean coastline of Israel is bordered by a linear escarpment 10 to 50 m heigh, composed mainly of calcareous cemented quartz sand deposits. Recurring slides are common along a 13 km length of the cliff. This paper describes an investigation of cliff stability, which combined geologic, soil engineering and geodetic studies. It was found that by combining geological mapping of the profile, observation of field slope stability conditions, laboratory testing and analysis, it was possible to estimate reasonable, conservative strength parameters for the units making up the cliff, and so to develop design guidelines for stability analysis.

INTRODUCTION

Part of the Mediterranean coastline of Israel is bordered by a linear escarpment 10 to 50 m. heigh. This coastal cliff is composed of quaternary formations, mainly of calcareous cemented quartz sand deposits, and is heighest in the central part of the coast between Tel Aviv and Netanya. A 13 km section of the cliff in the vicinity of Netanya has been subject to recurring shallow slides (eg. Fig. 1) and rockfalls, creating a serious problem both with respect to the utilization of the heavily populated beaches and to the stability of valuable property on the escarpment.

With a view to developing stability criteria for the troublesome 13 km length of cliff, an extensive investigation was carried out including geological mapping of the cliff profile, detailed surveying of a large number of sections along the cliff and their identification as stable or unstable slopes, sampling and testing of the various soils in the profile, and performance of stability analyses. This paper presents the results of the investigation.

GEOLOGICAL SETTING

The quaternary formations in the Israeli coastal region were formed during a series of interchanging ingressions and regressions of the Mediterranean Sea, accompanied by sedimentation cycles of marine and continental deposits. The major portion of the formations, comprising the lower part of the profile, are pleistocene layers of sands with varying degrees of calcareous cementation (locally called kurkar) and clayey sands. The upper portion of the profile includes holocene layers of hard sandstone overlain by friable sandstone; these are often covered by recent sand dunes. The profile is shown schematically in Fig. 2 in which 5 geological units are defined. Although the figure describes the complete columnar section, one or more of the units may not be present at various locations along the coast. In all cases where both units 2 and 3 are present, they are separated by a clearly visible layer which has been labelled the roof of unit 2. This layer served as an excellent marker during geological mapping, helping to differentiate between units 2 and 3 even in those cases when they were visually similar.

Fig.1 A Typical Slide

The beach is narrow, varying from about 5 to about 30 meters in width. The sea reaches the cliff almost everywhere along the coastal length studied, this being evidenced by tar lines and drift material observed along the cliff base. Tallus from landslides is rapidly washed away.

GEOTECHNICAL CHARACTERISTICS OF THE PROFILE

Unit 2

The exposed thickness of this unit above sea level reaches up to 24 m. It consists of cross-bedded, interchanging laminations of thin sandstone plates and loose monosized sand. The sandstone plates range in thickness from 0.2 - 2.0 cm, and the spacing between them ranges from 0.5 - 5.0 cm. The sand portion of the unit makes up about 70% of the thickness, and consists mostly of particles of about 0.3 mm diameter. The internal structure of the unit is of the type seen in Fig. 3 which shows an X-ray photograph of a sample of the same soil type located at a different site along the coast. The dry density of blocks of material was found to be generally between 1550-1700 Kg/m^3, with some occasional values outside this range. Carbonate content is of the order of 20% in the - 40# fraction and 40% - 60# in the +40# fraction. Specimens for laboratory strength testing were obtained by taking block samples from the cliff, partially saturating them in the laboratory, freezing, and then coring cylindrical, frozen specimens. These specimens were placed in the triaxial cell, subjected to all round pressure and then allowed to thaw; they were then saturated and sheared under fully drained conditions. Frydman et al (1979) demonstrated that this procedure of specimen preparation had no significant detrimental effect on the friction angle, ϕ', of sand. Fig. 4 shows stress-strain curves and Fig. 5 shows corresponding Mohr failure stress circles and strength envelope resulting from one series of tests; the strength parameters obtained in the investigation were $c' \sim$ kPa, $\phi' \sim 37^{o}$.

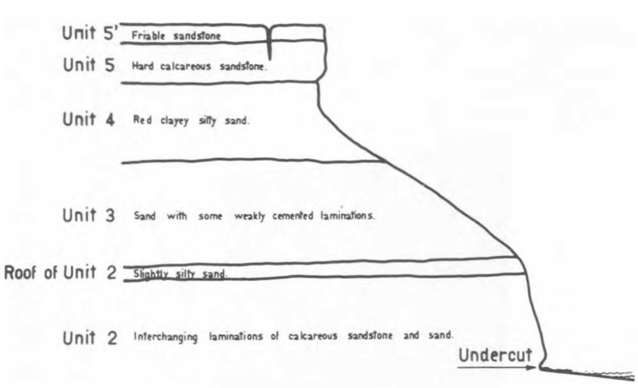

Fig.2 Schematic Representation of Profile

Roof of Unit 2

A layer of silty sand (10%-20% fines), of thickness up to 1.5 m covers unit 2. The carbonate content of the layer varies between 10%-30%, and it has considerable apparent cohesion.

Unit 3

The exposed thickness of this unit above sea level is up to 38 m. Two types of internal structure interchange within the unit. The first is similar to that observed in unit 2, consisting of more friable cemented sand laminations of thickness 0.2 - 1.5 cm sandwiching loose sand and spaced at 0.5 - 12 cm. The loose sand makes up about 90% of the total thickness, and has a carborate content of about 10% (compared to about 20% in unit 2). The other type of internal structure consist of lenses of sand varying in thickness from 0.5 - 3 m; these lenses are located between the interchanging laminations described previously. The sand particles throughout the unit are similar to those in unit 2. Dry density values were obtained from sand replacement tests, and found to vary between 1500 - 1650 Kg/m^3. Attempts to take blocks of soil for laboratory strength testing were unsuccessful due to lack of cohesion of the material. In view of this fact, together with the large scale presence of clean sand lenses, zero cohesion may be reasonably assumed for this material. Based on laboratory test results from similar sands (eg. Zolkov and Wiseman (1965)), the friction angle, ϕ', may be expected to be about 35^{o} - 37^{o}, similar to that of the unit 2 material.

Fig.3 X-ray Photograph of Unit 2 Material

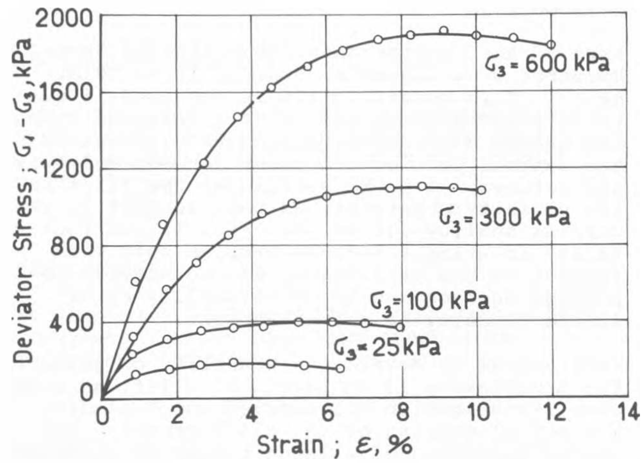

Fig.4 Triaxial Stress-Strain Curves - Unit 2

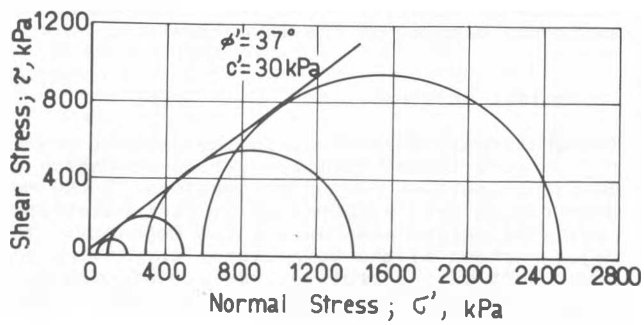

Fig.5 Mohr Failure Circles - Unit 2

Unit 4

The thickness of this unit is up to 6 m, and it consists of reddish-brown clayey sand with insignificant carbonate content, having a liquid limit of about 23% and a plasticity index of about 10%. The dry density is between 1820 - 1860 Kg/m^3. Strength tests were carried out on specimens prepared from blocks taken from the unit. Drained direct shear tests and consolidated undrained triaxial tests, both on saturated specimens, yielded strength parameters $\phi' = 35^{o}$, c' up to 10 kPa.

Units 5, 5'

Unit 5 consists of hard, calcareous sandstone, and varies in thickness up to 8 m. This unit is the only rock like layers in the profile. It is often overlain by unit 5' consisting of friable sandstone of thickness up to 1.5 m. Due to its smaller thickness and less regular occurrence, unit 5' is considered of secondary importance, and is not discussed further here, Unit 5 is formed from carbonate-cemented sand particles similar to those in units 2 and 3, together with some shells. The carbonate content is of the order of 55%, compared to 30% in unit 2. Dry density of unit 5 material is in the range 1550 - 1700 Kg/m^3, and shear strength tests on saturated, cored specimens yielded c' = 170 - 600 kPa, $\phi' \sim 38^{o}$ and unconfined compressive strength 2500-3000 kPa.

ENVIRONMENTAL FEATURES

A number of environmental features have an important effect on the stability and appearance of the coastal cliff. Surface flow of rainwaters leads to erosion, development of gullies and localized instability. The pervious nature of the upper units of the profile results in wetting of the soil to considerable depths, although there is no evidence of the development of a sub-surface saturated flow regime, or of the type of slip phenomena which would be expected to be associated with such a regime.

Wind action on the cliff face results in removal of sand grains from between the sandstone plates of unit 2. Fig. 6 shows a block of soil from this unit, before and after it has been subjected to artificial wind action in the laboratory. This phenomenon is useful in helping to identify fresh slides in the field; immediately after a slide, the slip surface appears as a smooth sand surface, but within one season, sand particles are removed leaving an irregular surface of clean, sandstone plates. This is illustrated in Fig. 1 which shows the sandy surface of a fresh landslide, adjacent to an older face where the sandstone plates are clearly seen.

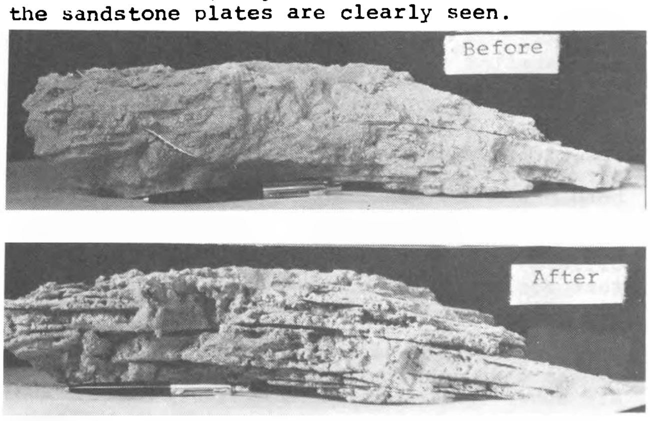

Fig.6 Effect of Wind Action on Unit 2 Material

Wave action at the base of the cliff results in undercutting and associated instability. In cases where unit 3 is at the base of the cliff, and subject to wave action, no stable, unsupported undercut can stand, and the cliff slope adapts itself to the erosive wave action. In the case of unit 2, however, the cohesion of the material enables formation of an unsupported undercut which increases in dimensions with time until collapse occurs, sometimes leaving an overhanging upper section which itself is metastable and may eventually collapse. The stable size of the undercut increases with increasing cohesive strength of the soil at the base of the cliff. Once collapse has occurred, the process begins anew, except at those locations where erosive wave effects are prevented by the presence of man made or natural protective features; it is not uncommon for large rock masses of unit 5, which have collapsed from the cliff face, to protect the adjacent cliff base against further wave action.

The problem of undercutting is being effectively solved by the construction of protective structures along the coast. However shallow slips have continued to occur even at locations where such preventative action has been taken, indicating the inherent instability of some sections of the cliff due to their geometry and soil properties. It is this classical slope stability problem which has received the brunt of the effort of the present investigation. Other instability phenomena such as rockfalls from unit 5, resulting from collapse of sections of this unit left cantilevered following slip or erosion of the lower portion of the profile, are not considered in this paper.

CHOICE OF FIELD OPERATIVE SOIL STRENGTH PARAMETERS

With a view to developing a rational basis for analyzing the stability conditions of any particular section of the cliff, and for designing safe engineering solutions for unstable sections, further effort was made to verify the applicability of the strength parameters measured in the laboratory to actual field conditions. The first step in this process consisted of detailed geological mapping and geodetic surveying of a large number of sections along the cliff face. Table I present a summary of cliff characteristics at 87 sections. It is seen that units 2 and 3 constitute the major portion of the cliff profile, and may be expected to have a predominant effect on the cliff slope stability over most of its length. Unit 2 appears to present the most uncertainty with regards choice of relevant strength properties. Unit 3, which clearly lacks significant cohesion, can confidently be characterized by c' = o and $\phi' = 35^{o} - 37^{o}$. These strength parameters are consistent with the data listed in Table I indicating that almost all slopes of unit 3 with an inclination of greater than 35^{o} were unstable. Unit 4 is generally of limitted extent and thickness, and independent tests performed on material sampled from different locations have shown the cohesion value to be of the order of 10 kPa, and ϕ' of the order of 35^{o}. Unit 5 is generally of high strength, and landslides (as opposed to rockfalls) through this unit are unlikely.

TABLE I

Cliff Characteristics at 87 Sections

Slide in Unit indicated	UNIT 5' Unit Slope	UNIT 5' Unit thickness (m)	UNIT 5' Total ht. of the unit and overlying layers (m)	UNIT 5 Unit Slope	UNIT 5 Unit thickness (m)	UNIT 5 Total ht. of the unit and overlying layers (m)	UNIT 4 Unit Slope	UNIT 4 Unit thickness (m)	UNIT 4 Total ht. of the unit and overlying layers (m)	UNIT 3 Unit Slope	UNIT 3 Unit thickness (m)	UNIT 3 Total ht. of the unit and overlying layers (m)	UNIT 2 Unit Slope	UNIT 2 Unit thickness (m)	UNIT 2 Total ht. of the unit and overlying layers (m)	Section Height (m)	Section Number
					0.7	0.7		4.2	4.9							4.9	1'
					1.4	1.4				38°	8.3	9.7				9.7	2'
3										42°	12.0	13.0				13.0	3'
3										40°	13.2	13.2				13.2	4
					1.6	1.6		1.0	2.6	33°	9.1	11.7				11.7	8
3					0.4	1.0				31°	18.0	20.0				20.0	9
3										36°	12.3	12.3				12.3	10
					1.1	1.1				38°	11.6	12.7				12.7	11
					1.3	1.5		1.2	2.7	36°	8.1	10.8				10.8	12
3					0.8	0.8		0.6	1.4	38°	11.0	12.4				12.4	13
					3.8	3.8	20°	4.9	8.7							8.7	14
3					0.7	0.7				80°	13.6	14.3				14.3	15
2		3.8			0.4	4.2					3.3	7.3	60°	11.3	18.8	18.8	16
2								0.7	0.7				62°	20.9	21.6	21.6	17
2													82°	17.8	17.8	17.8	18
2								0.4	0.4				71°	14.8	15.2	15.2	19
											7.1	7.1		8.2	15.3	15.3	20
		1.3	1.3		0.3	1.6	40°	3.9	5.5				90°	5.9	11.2	11.2	21
										42°	6.2	6.2	86°	9.4	15.6	15.6	22
					1.6	1.6		3.6	5.6	38°	9.6	15.2	90°	2.0	17.2	17.2	23
2					1.1	1.1		0.6	1.7	80°	3.5	5.2	65°	9.6	14.8	14.8	25
2								0.7	0.7	44°	5.4	6.1	61°	8.2	14.3	14.3	26
3										66°-44°	11.4	11.4	48°	5.0	16.4	16.4	27
3										62°	11.8	11.8		4.0	15.8	15.8	28
2											2.7	2.7	>90°	13.3	16.0	16.0	29
										60°	17.0	17.0	90°	5.0	22.0	22.0	30
					1.5	1.5	35°	11.2	12.7				90°	3.9	16.6	16.6	31
2													>90°	10.2	10.2	10.2	32
2					2.0	2.0		2.6	4.6				90°-45°	4.5	9.1	9.1	40
					1.8	1.8	31°	5.2	7.0				78°	6.0	13.0	13.0	41
2													>90°	20.0	20.0	20.0	42
2													>90°	14.3	14.3	14.3	43
													>90°	16.5	16.5	16.5	44
2											2.4	2.4	90°	12.0	14.4	14.4	45
2													58°	13.0	13.0	13.0	46
													54°	13.2	13.2	13.2	47
								1.1	1.1	42°	6.2	7.3	87°	8.7	16.0	16.0	48
3								1.1	1.1	40°	10.4	11.5		6.0	17.5	17.5	49
2													90°-45°	22.8	22.8	22.8	50
													90°	23.3	23.3	23.3	51
														20.3	20.3	20.3	52
													>90°	18.1	18.1	18.1	53
					2.5	2.5		1.0	3.5	42°	10.0	13.5	90°	4.5	18.0	18.0	54
					1.2	1.2		2.0	3.2	46°	7.5	10.7	82	10.2	20.9	20.9	55
3					2.3	2.3		1.3	3.6	35°	14.6	18.2				18.2	56
3					2.1	2.1		1.4	3.5	80°-45°	20.7	24.2				24.2	57
3					2.6	2.6		1.2	3.8	61°	18.4	22.2				22.2	58
2					2.0	2.0				38°	18.9	20.9	87°	8.5	29.0	29.0	59
		1.1	1.1		3.9	5.0		2.5	7.6	36°	14.0	21.5	80°	6.5	28.0	28.0	61
					0.7	0.7		1.3	2.0	45°	16.0	18.0	43°	17.5	35.5	35.5	62
		0.5	0.5								3.5	4.0	80°	23.0	27.0	27.0	101
				30°	5.5	5.5		1.2	6.7	39°	22.0	28.7	90°	2.8	31.5	31.5	102
				90°	2.0	2.0	90°	1.9	3.9	40°	13.5	13.5	86°	17.5	37.0	37.0	103
				90°	2.4	2.4		2.1	4.5	41°	26.5	31.1	88°	9.0	40.1	40.1	106
3				90°	2.7	2.7	90°	4.7	7.5	75°-45°	37.5	45.0				45.0	107
3				90°	2.1	2.1		0.6	2.7	75°-45°	35.3	38.0	64°	3.2	41.2	41.2	109
3										36°	33.0	33.0				33.0	110
					2.0	2.0		1.5	3.5	40°	13.0	16.5	77°	8.0	24.5	24.5	111
				75°	3.1	3.1	75°	2.0	5.1	36°	24.5	29.6				29.6	112
				90°	3.5	3.5	36°	1.5	5.0	36°	19.0	24.0	83°	11.0	35.0	35.0	115
2					3.2	3.2	36°	1.7	4.9	30°	7.5	12.4	90°	16.2	28.5	28.5	118
3				87°	4.4	4.4	24°	2.0	6.4	67°-42°	23.8	30.2				30.2	121
				82°	2.0	2.0	82°	1.4	3.4	36°	14.5	18.0	90°	8.0	26.0	26.0	124
		0.5	0.5	30°		2.5		1.0	3.5	35°	10.5	14.0				14.0	127
									3.0	40°	9.5	12.2				12.2	128
				90°	2.0	2.0	90°	2.0	4.0	34°	20.0	24.0				24.0	130
										36°	16.0	15.0	86°	14.0	30.0	30.0	140
					4.0	4.0	90°	2.4	6.4	37°	28.1	34.5	90°	5.5	40.0	40.0	145
				90°	4.0	4.0	90°	1.8	5.8	38°	32.7	38.5				38.5	146
2													90°	24.5	24.5	24.5	149
3										41°	17.5	17.5	90°	14.0	31.5	31.5	152
3				90°	2.5	2.5	90°	0.9	3.3	42°	18.2	21.0	90°	8.0	29.1	29.4	155
3				90°	3.0	3.0	36°	1.6	4.6	36°	13.4	18.0	90°	14.0	32.0	32.0	158
3				90°	2.5	2.5	27°	2.3	4.8	45°	18.3	23.1	90°	5.9	29.0	29.0	161
3				90°	3.4	3.4	20°	1.9	5.3	41°	14.9	20.2	86°	3.5	23.7	23.7	164
3				90°	3.0	3.0	90°	0.6	3.6	34°	9.8	13.3	90°	10.7	24.0	24.0	167
3				90°	3.0	3.0	30°	0.5	3.5	34°	10.9	14.0	75°	13.5	27.5	27.5	170
2		1.0	1.0										71°	29.3	30.3	30.3	173
2					1.0	1.0							75°-35°	19.0	20.0	20.0	176
					3.5	3.5	45°	1.7	5.2	45°	3.5	8.7	82°	3.0	11.7	11.7	182
		0.8	0.8		2.2	3.0		0.5	3.5	20°	3.2	6.7	74°	6.8	13.5	13.5	185
										35°	6.4	6.4	81°	8.4	14.8	14.8	188
		3.0	3.0					2.8	5.8	30°	1.0	6.8	70°	7.2	14.0	14.0	191
2										45°	1.2	1.2	85°	13.8	15.0	15.0	194
2													90°	12.0	12.0	12.0	197
2													90°	8.0	8.0	8.0	200

In the case of unit 2, the large variability observed in the degree of cementation within the material raises doubts as to the generality of the parameter c' = 30 kPa measured in the laboratory. As the laboratory tests were only carried out on blocks of material strong enough to allow sampling, transportation and handling, it is quite feasible that this value does not conservatively reflect the overall operative strength of the unit in the field. On the basis of the data presented in Table I, Fig. 7 was prepared, the points indicating the height of unit 2 as a function of the face slope of unit 2; a distinction is shown between cases in which unit 2 was stable and unstable. In most cases, unit 2 is overlain by additional units, in particular unit 3. However since unit 3 is generally inclined at a low slope (the order of 35^{o} in the stable condition), and slips, if they develop, are generally shallow due to the granular nature of the profile, the material overlying unit 2 generally has little effect on the stability of a potential slip surface through unit 2. In specific sections, where the inclination of unit 3 is similar to that of unit 2, the height shown in Fig. 7 is that of the both units combined, since it is clear that unit 2 would be at least equivalent in stability to this combination. Superimposed on Fig. 7 are curves obtained from Taylor's stability charts showing the stable height of a homogeneous soil slope with a horizontal upper surface as a function of slope angle for a soil with a saturated density, γ, of 2000 Kg/m^3, $\phi' = 35^{o}$ and c' varying from 10 to 30 kPa. It is noted that using Taylor's charts as a first approximation for analysis of the field conditions, a cohesion value, c', of between 10 to 20 kPa would provide a reasonable lower bound to the points representing instability within unit 2.

A further opportunity to check the field operative strength parameters in unit 2 was provided by the occurance, during the investigation, of a well defined slide in this unit at Beit Yanai, about 6 km north of Netanya.

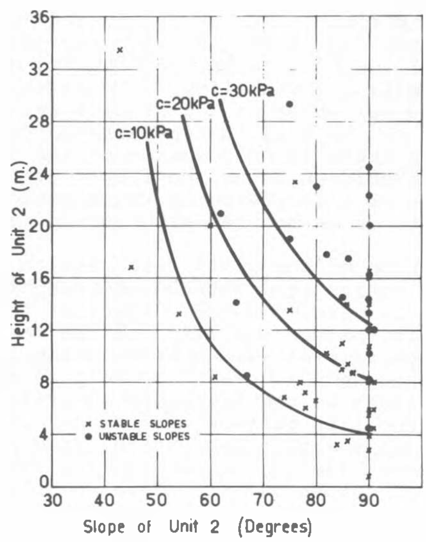

Fig.7 Height versus Slope - Unit 2

The detailed topography of the slip region was established using photo-theodolite measurements, while the slope of the cliff preceding the slide was known to correspond more or less to that of the stable material at its sides. Fig. 8 shows a section through the middle of the slip region both prior to, and after the slide. Stability analyses by the simplified Bishop method were carried out for the actual slip surface, using different strength parameters and it was found that for strength parameters c' = 20 kPa, $\phi' = 30^{o}$, a factor of safety of 0.99 was obtained. Using these strength parameters, a large number of possible slip surfaces were checked, and the resulting factors of safety are shown in Fig. 9. It was found that of all the possible surfaces passing through the base of the slope, the actual slip surface was the shallowest one which provided a factor of safety of less than 1. No surfaces analyzed which cut the slope face above its base, yielded factors of safety below 1. The fact that slip actually occurred on the most shallow surface having a factor of safety below 1 and not on deeper surfaces which yielded lower factors of safety suggested that the cohesion increases with distance into the cliff profile away from the face, possibly as a result of decreased wetting and erosive effects.

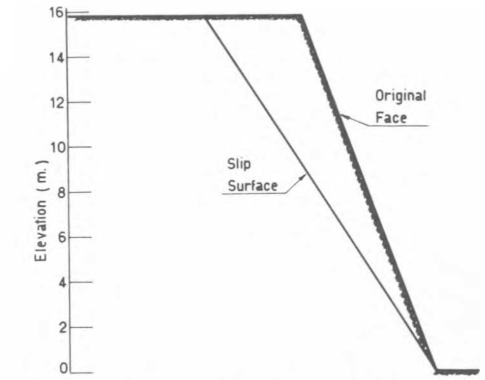

Fig.8 Slide Section at Beit Yanai

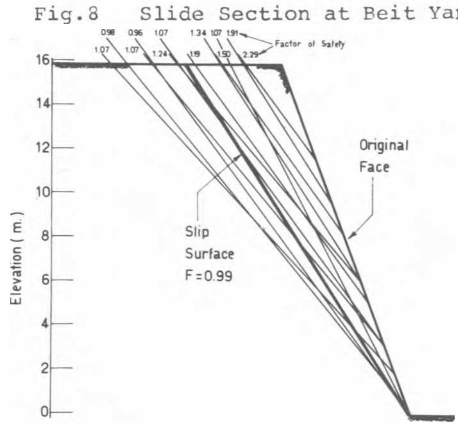

Fig.9 Slip Surfaces Analysed at Beit Yanai

It may be concluded on the basis of laboratory test results, modified by observation and analysis of field behaviour, that unit 2 may be represented by operative strength parameters $c' = 20$ kPa, $\phi' = 35^{\circ}$.

DEVELOPMENT OF DESIGN CURVES

Design curves have been developed for cliffs comprising unit 2 overlain by unit 3. It is considered that these curves could be used conservatively also for cases in which layers of unit 4 overly the profile, since this unit has a similar ϕ' value to that of unit 3, but an additional small cohesion value of about 10 kPa. Unit 5, due to its high strength, does not normally take part in landslide phenomena, and often remains, overhanging, when slips occur below it. In some cases, however, unit 5 is cracked, and in these cases it may be conservatively represented by a $c' = o$ material, whose contribution to stability, being on the top of the profile, is almost insignificant.

Curves were developed for the stable height of unit 2 when overlain by unit 3 inclined at 30°. This latter inclination represents a factor of safety of 1.2 against sliding within unit 3. Stability analyses were carried out using the simplified Bishop method, to check the stable height of cliffs with inclination of unit 2 varying from 60° to 90°. A large number of possible slip surfaces were analyzed for each cliff geometry, and that surface with a factor of safety of less than 1.0, cutting the cliff face at the heighest elevation, was taken as the critical surface, leading to an estimate of stable cliff height. Fig. 10 shows curves for overall factors of safety, F, of 1.0 and 1.5. The curve corresponding to a factor of safety of F = 1.0 is shown in Fig. 11 superimposed on the field data presented previously in Fig. 7. The figure also includes the curve obtained using Taylor's charts (i.e. for a horizontal top surface to unit 2) with $c' = 20$ kPa and $\phi' = 35^{\circ}$. It is seen that only one point representing an unstable slope in the field lies significantly below the design curve, and it appears to conservatively represent overall field observations. Obviously, design of safe slopes in the field would make use of design curves incorporating a factor of safety, and the curve corresponding to F = 1.5 in Fig. 10 would be more suitable for this purpose.

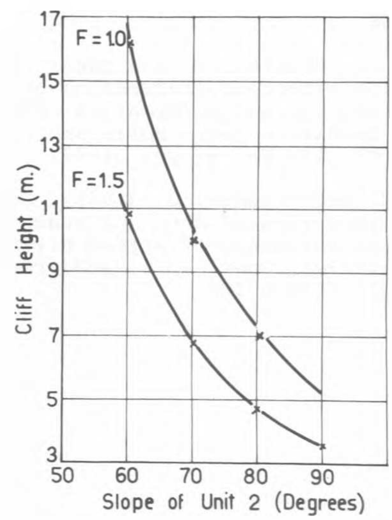

Fig.10 Design Curves for Slope Stability Analysis

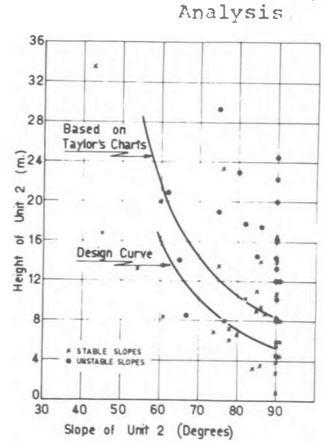

Fig.11 Design Curve Compared to Field Observations

CONCLUSIONS

The success of the investigation described in this paper was made possible only by the cooperation of engineering, geologist and geodetic personel, making use of field observations and measurements, laboratory testing and analysis. Due to the extremely heterogeneous nature of the profiles encountered, estimation of reasonable, field operative strength parameters could not confidently be made on the basis of laboratory tests alone. These tests, together with careful measurement and observation of field conditions, and back analysis of stable and unstable slopes, provided a framework for a rational choice of suitable strength parameters. The methodology used in the investigation could be applied to other sites where similar problems exist.

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