

CHECKING SQUEEZE-OUT ZONE UNDER AN EMBANKMENT CONSTRUCTED AT SEABED AND ITS SLOPE-STABILITY ANALYSIS

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ABSTRACT

Embankments are laterally unsupported structural fills, usually constructed to support buildings, highways, dams, etc. When constructed under carefully controlled conditions, these may have the strength and supporting-capability as good as, or better than, many of the natural foundations. Construction of an embankment imposes shear-stress in the foundation. Determining the stability of the foundation, with respect to failure in shear, is an important factor for an embankment-design. The foundation-stability is estimated by comparing the magnitude of the induced stresses with that of available shear-resistance. Embankments may be built on the soft seabed for constructing building-sites or to support a highway formation. On such sites, the soft seabed material of negligible shear-strength cannot support the embankment load and, hence, with the increase in embankment height, the soft sub-soil starts to squeeze out through the toe.

This paper describes the checking of squeeze-out zone under an embankment that was constructed on seabed and raised to about 1.5 meters above the water-level in the sea to support roadway formation. At different locations, the width of the embankment varied from 45 to 100 meters and water-depth in the sea varied from 2.5 to 8.5 meters. By comparing the active forces (that tend to actuate squeezing) and passive forces (that tend to resist squeezing), a squeeze-out zone, about 3 meters deep, was found. Also, the slope-stability of the embankment was checked by using a software named 'STABL', which performed effective stress-analysis and displayed a factor-of-safety of 1.76 for the embankment.

Key Words: Soil, slope-stability, squeeze, embankment.

INTRODUCTION

Embankments are laterally unsupported fills, usually constructed to support buildings, highways, dams, etc. These may be constructed of various categories of natural soils, such as select fills, compacted clay, silt fills and rock fills (Monahan, 1994).

Starting from a leveled platform, the construction of an embankment imposes stresses in the foundation. There are direct stresses, but the stresses that

concern us directly are the shear-stresses. As each lift of fill (embankment) is placed, it also increases the shear-stresses in earlier lifts adjacent to the sloping face of the embankment, as well as in the soil forming the foundation. These applied shear-stresses represent the tendency of the soil-slope to slide under the influence of gravity. For low fill-heights, the shear-stresses are much less than the shear-strength of the fill-material and of the foundation-soil, and shear-deformations are small. As the fill is raised, the ratio of the shear-stress to strength rises and, due to the non-linear relationship between stress and strain, deformation increases faster than the rate of mobilization of the available shear-strength. At some point in the construction-sequence, increasing fill-height will cause the application of stress equal to shear-strength in a part or parts of the soil. Extra load at the crest of the slope will then merely cause movement until the rising of the toe, and settlement of the head of the slide brings the soil-mass back into equilibrium, with the shear strength acting along the sliding surface. It may prove impossible to bring an increase in the height of the embankment and the result of placing more fill at the top is simply to cause a lateral spreading of the embankment-toes (Bromhead, 1986).

Determining the stability of foundation with respect to failure in shear and prediction of settlement to be expected, as a result of compression of the foundation-material, are the important factors in embankment-design (Hough, 1969). The foundation stability is estimated by comparing the magnitude of the induced stresses with that of available shear-resistance. The shearing stress, to which a foundation can be subjected, depends upon the unit-weight of the overlying materials and the geometry of the slope, while the shearing strength that can be mobilized to resist the shearing stress depends upon the character of the foundation-soil, its density and drainage conditions.

Movement of sloped soil masses can be classified into broad categories, depending on the type of motion relative to the adjacent or underlying earth. These may be in the form of slides, block- or wedge-failure and flows or spreads. Slides refer to the occurrence where the moving-mass is rather well-defined and spread from the underlying and adjacent earth by a plane or zone, comprising a number of adjacent planes, where slippage occurs. The slippage-plane or zone

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represents the continuous surface where the maximum shear-strength of the earth material has been reached that results in large displacement. The failure-mass remains essentially intact, although it may fracture into sub-units. Block- or wedge-failure refers to the displacement of an intact mass of soil because of the action of an adjacent zone of earth. Distinct blocks and wedges of soil may become separated from the adjoining earth because of the presence of cracks, fissures or joints, or even because materials having different properties are involved. Flow or spread are more complex types of soil-mass movements. A flow involves lateral movement of soil having the characteristics of a viscous fluid, although the actual consistency of the moving-mass may vary from very wet to dry. Spread refers to the occurrence of multi-directional lateral movements by fractured soil-mass (McCarthy, 1988).

Embankments may be constructed in water for different purposes, such as for breakwaters, for constructing building sites for housing on the very soft seabed, etc. (Teraet, 1996). When an embankment is constructed on saturated soft ground, pore-pressure is created due to the embankment load. If the

embankment is constructed in a short period or undrained conditions, the factor-of-safety against failure is reduced due to the pore-pressure, and the most critical conditions arise at the end of construction. Figure-1 diagrammatically shows the variation of pore-pressure and factor-of-safety with time (Walker and Robifell, 1987).

With passage of time, the pore-pressure induced by construction dissipates and the effective stress increases. Consequently, the shear-strength of sub-soil (foundation soil) and factor-of-safety against slip-failure increases with time. Hence, the critical time for stability of embankments constructed in water is at the end of construction.

Stability problems of natural slopes and fill-slopes (embankments, earth dams and levees, etc.) or cut-slopes are commonly encountered in civil engineering projects. Because of its practical importance, the analysis of slope-stability has received wide attention in literature. Fall, et. al., (2006) presented a multi-method approach to study the stability of natural slopes and hazard assessment of land-slides. They found that the slides were influenced by the geo-

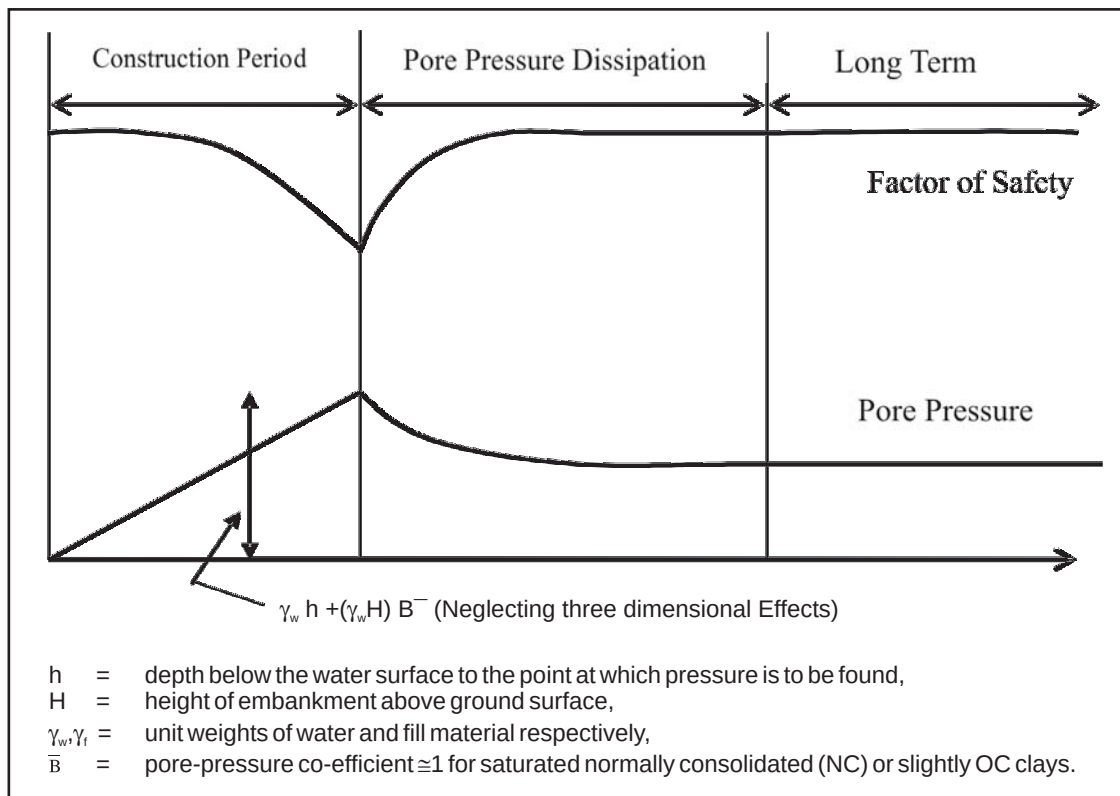


Figure - 1: Time-dependent Behaviour of Embankment

Figure - 2: Estimation of Over-consolidation Ratio from Liquidity Index (Wroth Concept)

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For computing undrained shear strengths, the following correlation formulas are used:

$$I_L = (w_n - w_p) / I_p \dots\dots\dots (1)$$

$$Su/\sigma'_0 = 0.11 + 0.0037I_p \text{ for normally consolidated clay} \dots\dots\dots (2)$$

$$Su/\sigma'_0 = 0.11 + 0.0037I_p(OCR)^{0.8} \text{ for over consolidated clay [20]} \dots\dots\dots (3)$$

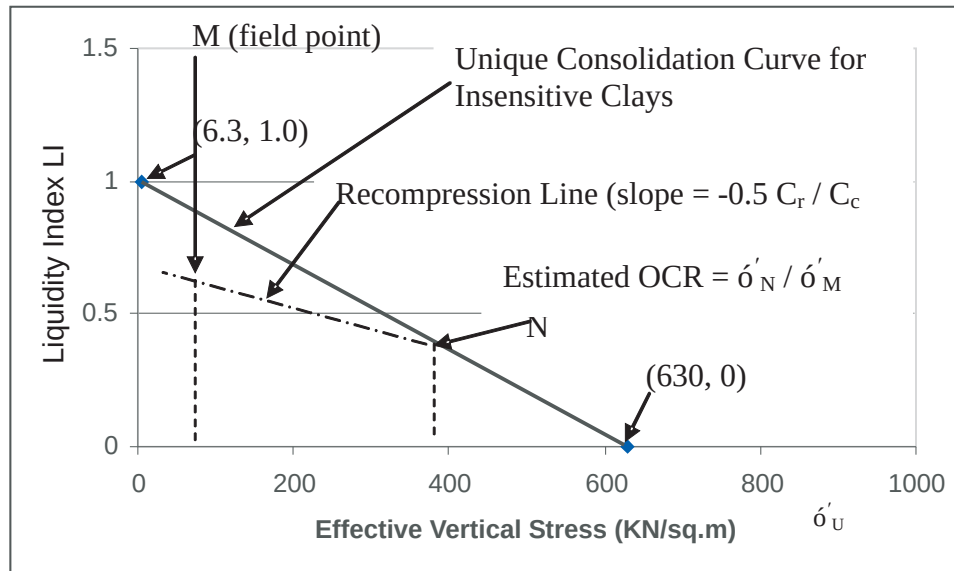


Figure - 2: Estimation of Over-consolidation Ratio from Liquidity Index (Wroth Concept)

technical properties of the soil along with other factors. Kvalstad, et. al., (2005) gave an overview of soil investigation, evaluation of potential trigger-mechanisms and stability-analysis in the Ormen Lange Area. Day (1992) presented a study on the effect of cohesion on stability-analysis for natural clay slopes and embankments. Park, et. al., (2005) developed a probabilistic analysis-procedure and related algorithms. They used this approach to analyze rock-slope stability for Inter-state Highway 40, North Carolina, USA. Durand, et. al., (2006) presented formulation, implementation and validation of numerical limit-analysis procedures for the study of stability-problems in soil and rock-masses. Kim, et. al., (1999) presented a finite-element formulation in terms of effective stress for limit-analysis of soil-slopes subjected to pore-water pressure. Radoslaw and Lei shi (1993) presented lower and upper-bound solutions for bearing-capacity of cohesive layers over rigid rough bases and proposed a method for calculations of embankments' failure-heights. Yu, et. al., (1998) compared the conventional limit equilibrium results with rigorous upper and lower-bound solutions for the stability of simple earth slopes. Kim, et. al., (2002) used limit-analysis method to compute lower and upper bound on the factors of safety for slopes with inhomogenous soil-profiles and irregular slope

geometry, subjected to the effects of pore pressure. Huang, et. al., (2002) proposed a sophisticated and computer-oriented three-dimensional slope-stability analysis. Christian, et. al., (1994) derived a probabilistic description of soil-parameters from field and laboratory-data and applied it in stability-analysis. Griffiths and Fenton (2004) investigated the probability of failure of cohesive slope, using both simple and more advanced probabilistic analysis tool. Hsu and Nelson (2006) used stochastic field models and Monto Carlo simulation to understand the impact of the spatial distribution on excavation and slope-stability. When an embankment is constructed on a soft seabed, the upper zone of the soft seabed material has negligible shear-strength and cannot support the embankment load. Thus, with the increase in the embankment height, the soft sub-soil starts to squeeze out through the toe. This continues till the active and passive forces become equal.

EXPERIMENTAL PROGRAMME

An embankment called 'Kardon embankment' was constructed on soft seabed in the city of Izmir – Turkey. The construction work was carried out by a Turkish construction company named Bayinder Holding. The embankment was raised to about 1.5m above the

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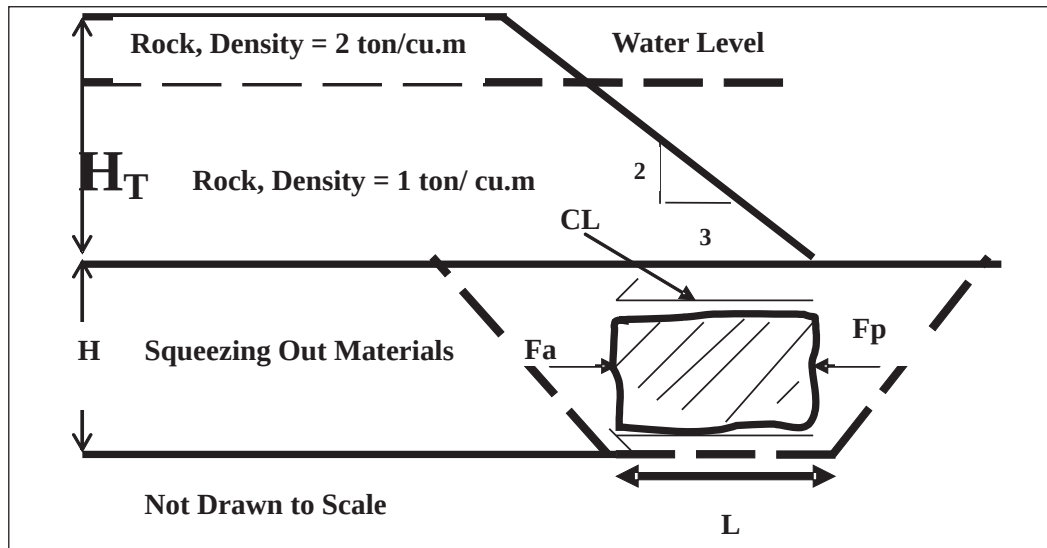


Figure - 3: Foundation Stability (Squeeze-out Zone)

water-level in the sea, to support roadway formation. The width of the embankment was varying from 45 to 100m and the depth of water in the sea was varying from 2.8 to 8.5m, at different locations along its length. Boring system was adopted for getting information about the soil below the seabed. Soil samples were taken at different depths and laboratory tests, including, Sieve analysis, liquid limit(w_l), plastic limit(w_p), undrained shear-strength(S_u) and compression index (C_c), etc., were performed. Some in-situ tests, such as standard-penetration tests and vane-shear tests, were also performed. As a whole, the soil was found to be soft to very soft clay, normally consolidated up to about 15m below the water. Somewhere at depths greater than 15m, thin sand-gravel layers were present, below which stiff over-consolidated clay was found down to greater depths/TBC.

OVER-CONSOLIDATION RATIO AND SHEAR-STRENGTH

Stability analysis requires information about the strength properties of the underlying soil-layers. The strength of soil-materials is dependent on the effective stress and the past consolidation. For computing over-consolidation ratio, Wroth Concept was adopted. Wroth suggests that the ratio of the recompression index C_r to the compression index C_c ranges from about 0.17 (for $I_p = 15\%$) to about 0.34 (for $I_p = 100\%$). As shown in Figure-2, it is found, that remoulded soils have a more or less unique one-dimensional normal consolidation liner, which passes through an effective

vertical stress of about 6.3 kN/m^2 at a liquidity index of 1.0, and 630 kN/m^2 at a liquidity index of 0.0. In the Figure-2, the point M represents the in-situ condition of the soil and, by drawing the recompression line (of slope $-0.5 C_r / C_c$ on this plot) through M to intersect the unique consolidation line, the point N may be found. The over-consolidation ratio (OCR) is then simply σ'_N / σ'_M .

SQUEEZE-OUT ZONE

When an embankment is constructed on a soft seabed, the upper zone of the soft seabed material has negligible shear-strength and cannot support the embankment load. Thus, with the increase in the embankment height, the soft subsoil starts to squeeze out through the toe. This continues till the active and passive forces become equal. Referring to Figure-3, the depth of squeeze-out zone under an embankment can be found by comparing the sum of active forces represented as F_a (the forces that actuate squeezing) to the sum of passive forces represented as F_p (the forces that resist squeezing). This concept gives the following equation:

$$qH - 2CH = 2CL + 2CH \quad \text{OR} \quad qH - 4CH = 2CL \dots \dots \dots (4)$$

- q = static overburden pressure (due to embankment load)
- C = S_u = strength of the subsoil
- L = length of the squeeze-out zone
- H = depth/thickness of the squeeze-out zone

For small thickness of the squeeze-out zone, F_a is less than F_p and, therefore, the squeezing continues. With increasing thickness of the squeeze-out zone, there is a gradual decrease in F_a and an increase in F_p . At a certain thickness, both F_a and F_p become equal and beyond this thickness F_a becomes less than F_p and, hence, whereon no further soil will squeeze out. The thickness, at which F_a and F_p becomes equal, is taken as the depth/thickness of squeeze-out zone.

RESULTS AND DISCUSSIONS

The results of various tests performed by Bayinder Holding on soil samples obtained from bore holes No. 3,6,7 and 9 are presented in Table-1. Some missing

values have been calculated by making interpolation between the nearest upper and lower values.

This table indicates that the subsurface-soil mainly consists of clay having low plasticity, while sandy clay is also present at some depths. Traces of clay having high plasticity and silty clay are also found.

GEO-TECHNICAL SOIL PROFILE

A geotechnical soil profile is shown in Figure-4. The profile indicates the description of soil strata, soil-group symbols and graphs for w_l , I_p , w_n , S_u and N , plotted from the readings are given in the Table-1.

Table - 1: Laboratory Results

Serial No.	Depth	Bore Hole No.	Sieve Analysis		Atterberg's Limits			Natural Water Content	Group Symbol	Standard Penetration Resistance	Saturated Density
			-No.4 sieve	-200 sieve	w_l	w_p	I_p	w_n			
-	-	-	-	-	%	%	%	-	-	N	γ_{sat}
-	M	-	-	-	%	%	%	-	-	N	t/m^3
1	6	3	83.8	9.3	28	22	(12)	46.4	SC	1	1.74
2	7	6	84.8	22.3	41	28	13	(42)	SC	0	(1.45)
3	9	7	90.5	57.3	48	21	27	39.1	CL	3	1.80
4	10	6	93.1	74.8	48	24	24	63.9	CL	0	1.59
5	11	7	100	90.1	42	23	19	46.7	CL	4	1.74
6	12	9	99.9	92.7	50	28	22	47.8	CL,CH	4	1.73
7	13	6	91.8	63.4	50	29	21	62.6	CL,CH	4	1.60
8	14	9	100	94.0	40	23	17	46.8	CL	6	1.74
9	15	3	95.4	86.7	34	22	12	29.3	CL	18	1.93
10	16	7	97.4	68.4	33	21	12	20.5	CL	40	2.00
11	17	9	94.4	66.3	37	20	17	22.5	CL	24	2.03
12	18	9	100	56.9	36	20	16	28.5	CL	31	1.94
13	20	3	72	18.0	34	17	17	11.6	SC	87	2.26
14	22	9	100	74.2	37	20	17	28.1	CL	22	1.95
15	23	7	94.7	56.6	35	19	16	18.4	CL	34	2.01
16	24	3	99.4	67.2	26	19	7	21.7	CL,ML	21	2.08
17	26	3	97.8	88.1	33	17	16	25.3	CL	26	1.99
18	27	7	97.9	63.0	35	19	16	15.4	CL	38	2.16
19	30	6	99.4	74.5	38	20	18	26.8	CL	28	1.86
20	31	9	100	74.4	54.3	16.6	37	(24.3)	CH	40	1.92
21	33	3	99.4	82.6	46	20	26	22.6	CL	(50)	2.17
22	34	3	98.5	76.0	37	21	16	27.1	CL	55	1.97
23	35	3	90.4	50.9	26	17	9	13.4	SC,CL	40	2.25
24	36	9	85.8	49.8	32	18	14	17.3	SC	46	2.13
25	37	33	79.8	37.4	33	23	10	12.5	SC	34	2.24
26	38	7	95.0	66.6	34	15	19	16.8	CL	34	2.14
27	39	9	99.2	68.6	32	20	12	23.1	CL	52	2.04
28	40	9	53.5	32.9	32.1	15	16	15.0	SC	78	2.10
29	43	3	100	73.9	35	10	25	22.1	CL	36	2.22
30	45	6	71.7	47.0	26.2	13.5	14	13.6	SC	60	1.89
31	47	7	100	81.9	39	23	16	26.1	CL	48	1.98
32	49	7	100	92.6	40	24	16	28.0	CL	46	1.95

Note: Values in () are calculated by interpolation

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Depth (m)	Soil Description	Group Symbol
0.00 to 3.00	Sea water	
3.00 to 8.50	Very soft, black silty organic clay of low to intermediate plasticity	COI-COL
8.50 to 15.00	Very soft to soft. Greenish grey to dark, silty clay of low to intermediate plasticity	CI-CL
15.00 to 17.00	Stiff greenish, grey to grayish brown, fine gravelly, silty clay of low plasticity	CLG-CLS
17.00 to 23.50	Very stiff to hard, silty clay of intermediate plasticity to slightly to high clayey silt of low plasticity	ML-CL
23.50 to 29.50	Brown highly silty, fine gravelly sandy clay of low plasticity to fine to medium grained clayey silty, sandy gravel	CLS-CLG - GWC-GWS
29.50 to 33.00	Very stiff hard yellowish brown highly silty clay of low plasticity. Occ.cons. some fine gravel	CL
33.00 to 34.50	Very stiff yellowish brown silty clay of intermediate plasticity	CI
34.50 to 47.00	Very stiff hard, brown silty sandy, fine, medium gravelly clay of low to intermediate plasticity	CLS-CIS-CLG-CIG (GWC)
47.00 to 51.00	Fine to medium grained high silty, sandy clayey gravel clay of intermediate plasticity	GC-CG

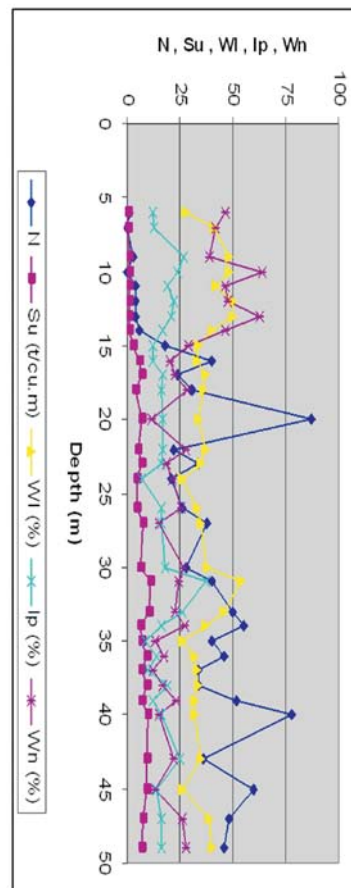


Figure - 4: Geo-technical Soil Profile

The over-consolidation ratio and undrained shear-strengths for soils at various depths are shown in Table-3. The results of Table-1 are used to calculate liquidity index (I_L) and over burden pressure (σ'_0). The over-consolidation ratios and undrained strengths are calculated on the basis of Wroth concept and correlation equations.

The over-consolidation ratio in Table-3 reveals that the

soil is normally consolidated up to 14 m depth, and below that it is over-consolidated. Also, the soil near the surface is very soft clay, having low shear-strength.

THICKNESS OF SQUEEZE-OUT ZONE

To check the thickness of squeeze-out zone under the embankment, four different sections were chosen and the thickness of the squeeze-out zone was estimated

Table - 2(a): Section I: $H_t = 8.5 + 1.5 = 10\text{m}$, $L = 15\text{m}$, $q = 2 \times 1.5 + 1 \times 8.5 = 11.5\text{t}$

H (m)	$C = S_u \text{ (t/m}^2\text{)}$	qh -4CH	2CL
0.5	0.08	6.60	2.40
1	0.2	10.70	6.00
2	0.45	19.40	13.50
2.5	0.62	22.55	17.60
3	0.80	25.00	24.00
3.5	0.90	26.60	27.00
4.5	1.10	32.00	33.00

Note: Required thickness of squeeze out zone found by interpolation = $H \approx 3.4\text{m}$

Table - 2(b): Section II: $H_t = 6.5 + 1.5 = 8\text{m}$, $L = 12\text{m}$, $q = 2*1.5 + 1*6.5 = 9.5\text{t}$

H (m)	C = S_u (t/m ²)	qH -4CH	2CL
0.5	0.08	4.60	1.92
1	0.2	7.70	4.80
2	0.45	14.00	10.80
2.5	0.62	16.55	15.00
3	0.80	17.90	19.20
3.5	0.90	20.65	21.60
4.5	1.10	22.95	26.40

Note: Required thickness of squeeze-out zone found by interpolation = $H \cong 2.8\text{m}$

Table - 2(c): Section III: $H_t = 4.5 + 1.5 = 6\text{m}$, $L = 9\text{m}$, $q = 2*1.5 + 1*4.5 = 6.5\text{t}$

H (m)	C = S_u (t/m ²)	qH -4CH	2CL
0.5	0.08	3.59	1.44
1	0.2	6.70	3.60
2	0.45	11.40	7.10
2.5	0.62	12.60	11.16
3	0.80	12.90	14.40
3.5	0.90	13.65	16.20
4.5	1.10	14.00	20.00

Note: Required thickness of squeeze-out zone found by interpolation = $H \cong 2.7\text{m}$

Table - 2(d): Section IV: $H_t = 2.5 + 1.5 = 4\text{m}$, $L = 6\text{m}$, $q = 2*1.5 + 1*2.5 = 5.5\text{t}$

H (m)	C = S_u (t/m ²)	qH -4CH	2CL
0.5	0.08	2.59	0.96
1	0.2	4.70	2.40
2	0.45	6.40	5.40
2.5	0.62	6.55	6.44
3	0.80	6.90	9.60
3.5	0.90	6.65	10.80
4.5	1.10	4.95	13.20

Note: Required thickness of squeeze-out zone found by interpolation = $H \cong 2.50\text{m}$

by using equation-4, while using average soil-strengths for each section (see the tabulated values as given in Table-2(a) to 2(d)).

The above analysis for four typical sections of the embankment indicates that the soft soil near the surface (up to about 2.5 to 3 m depth) will fail in shear, due to load of the embankment and will squeeze-out.

SLOPE- STABILITY ANALYSIS

The slope-stability analysis (effective stress) was performed by using the well known slope-stability computer code 'STABL', which displayed factor-of-safety alongwith critical failure circle. The approximate shape of the critical failure circle, along with factor of safety is shown in Figure-5.

The stability analysis displayed a factor-of-safety as

1.76 and, hence, declares it safe for the given embankment load.

CONCLUSIONS

1. The soil which has natural moisture-content significantly less than the liquid limit is pre-consolidated.
2. Squeeze-out zone can be estimated before construction, by comparing active and passive forces.
3. Factor-of-safety against slope-failure can be calculated by using a software "STABL" for effective stress conditions.
4. Squeeze-out zone of 2.50 to 3 m depth was found by analysis and, nearly, the same value was observed practically when surcharge load was applied to the soil.
5. A factor-of-safety of 1.76 is displayed by the slope-

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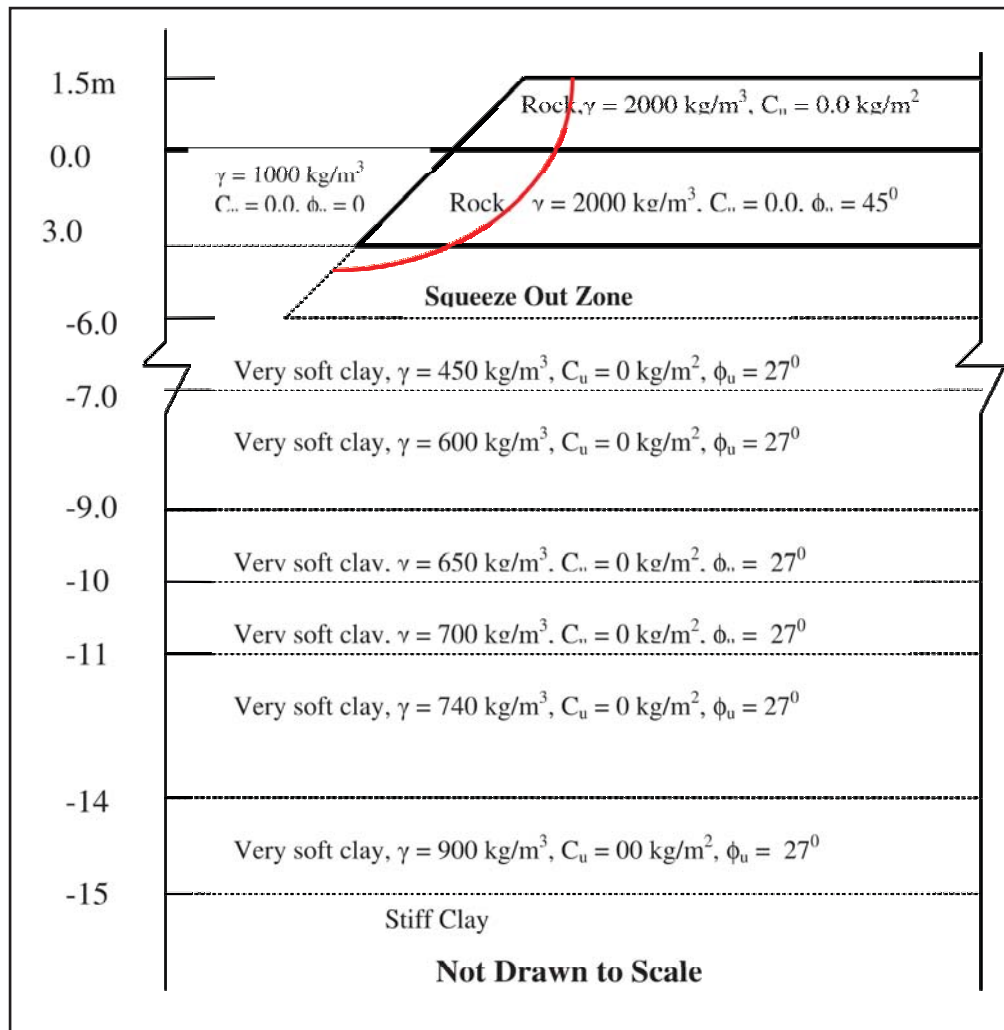
Table - 3: Over-consolidation Ratio and Undrained Shear-Strength

Depth	I_p	Over Burden	Pre-consolidation pressure	Over consolidation Ratio	Undrained shear strength
m	-	σ'_0	σ_c	OCR	S_u
-	-	t/m ³	t/m ³	-	t/m ³
6	-	2.52	NC	1	0.33
7	-	3.00	NC	1	0.48
9	0.67	4.20	NC	1	0.88
10	1.70	4.80	NC	1	0.95
11	1.25	5.40	NC	1	0.98
12	0.945	6.13	NC	1	1.17
13	1.60	6.86	NC	1	1.29
14	1.40	6.59	NC	1	1.31
15	0.61	7.50	25.50	3	3.16
16	0.00	9.50	63.00	6.63	5.96
17	0.15	10.50	56.00	5.33	6.92
18	0.52	11.54	31.50	2.73	4.36
20	0.00	13.62	63.00	4.63	7.02
22	0.48	15.70	34.50	2.20	5.10
23	0.00	16.74	63.00	3.76	7.17
24	0.39	16.85	39.50	2.20	4.56
26	0.53	20.21	30.50	1.51	4.75
27	0.00	21.40	63.00	2.94	7.60
30	0.38	25.00	40.00	1.60	6.43
31	0.20	26.10	51.80	1.98	11.25
33	0.10	27.30	57.60	2.10	10.56
34	0.38	29.40	39.50	1.34	6.30
35	0.00	30.50	63.00	2.07	6.82
36	0.00	31.60	63.00	2.07	9.15
37	0.00	32.70	63.00	1.93	7.13
38	0.10	33.80	57.00	1.72	9.40
39	0.26	34.90	46.50	1.36	6.90
40	0.00	36.00	63.00	1.75	9.76
43	0.48	39.30	-	-	-
45	0.00	41.50	63.00	1.52	9.32
47	0.19	43.70	51.50	1.18	7.44
49	0.25	46.00	47.00	1.04	7.03

stability analysis that shows it to be safe and stable under the load of the given embankment. Practically, no problem was observed, except for the occurrence of minor cracks in the existing road- surface adjacent to the embankment.

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Radius of critical circle = 2.2m and factor of safety for critical circle = 1.76

Figure - 5: Slope-Stability Analysis (Effective Stress)

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