Stability of Embankments on Soft Bangkok Clay

Two types of analysis

1. Total stress analysis – $\phi_u = 0$ method
2. Effective stress analysis --- use effective cohesion & effective friction angle

SHEAR STRENGTH OF SOILS

Safe Conditions

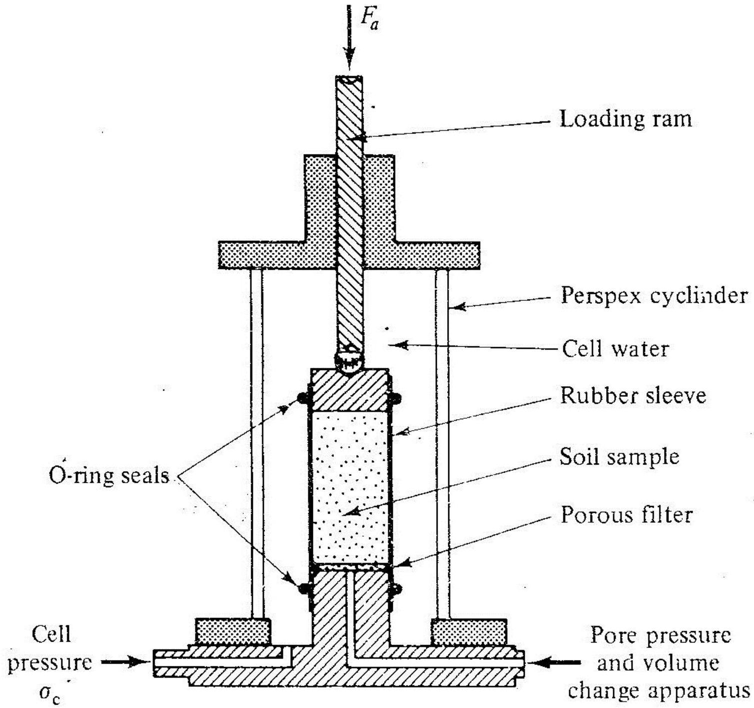
Unsafe conditions-- Failure

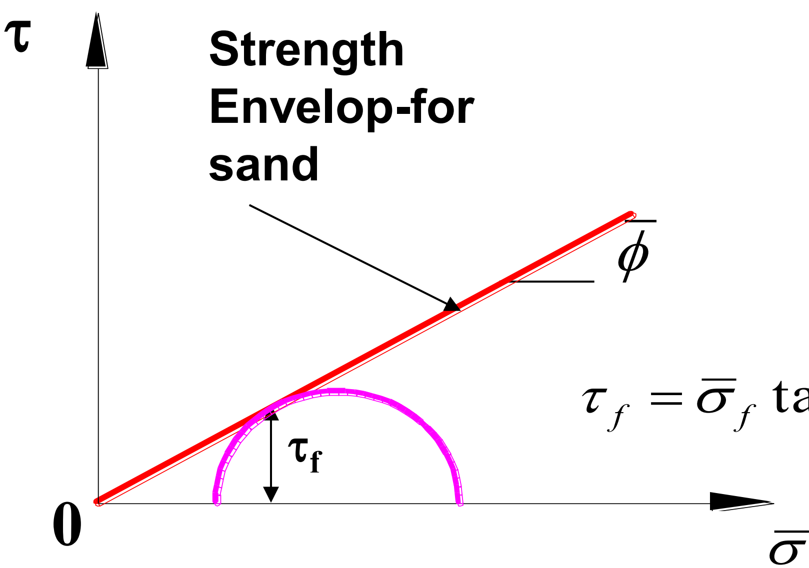
Section -9 : Part 1(a)

Mohr-Coulomb Strength Envelops in Effective stress

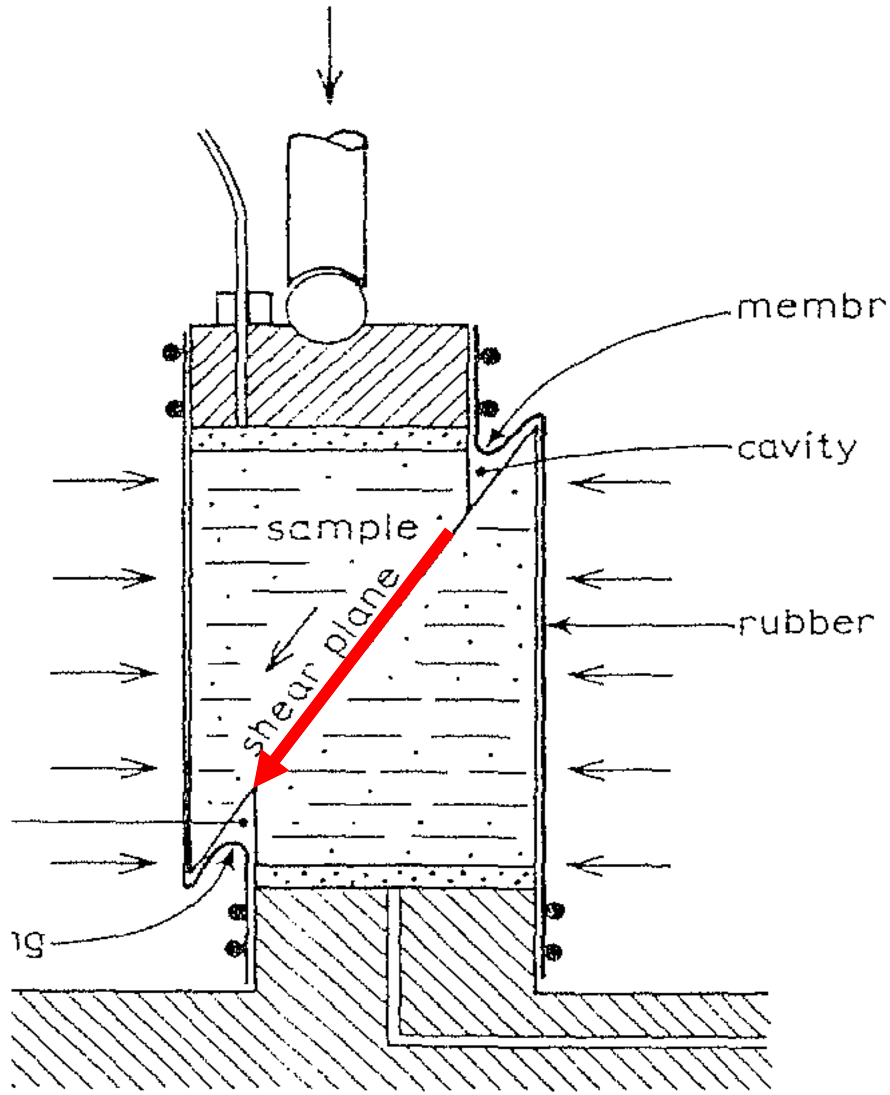
(i) for sand (chesionless soil—only $\overline{\varphi}$)
(ii) clays (Cohesive soils- $\overline{c}, \overline{\varphi}$)

Mohr-Coulomb Strength Envelops in Total stress for only clays: c_u , φ_u





$$\tau_f = \bar{\sigma}_f \tan \bar{\phi}$$



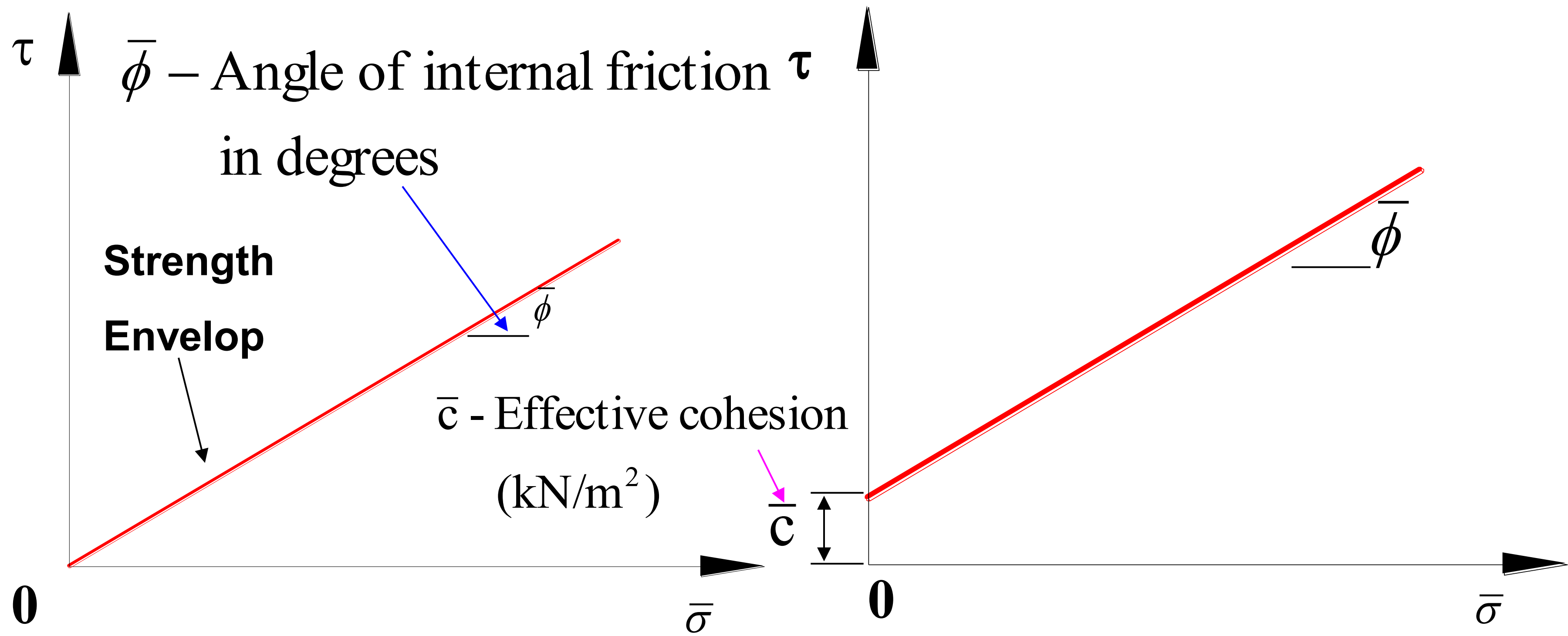
Sand sample in triaxial apparatus

Coulomb was the first to express shear strength. Subsequently with the invent of effective stresses the equations below are applicable for sand and clays respectively; sand is cohesionless while clay possesses cohesion and friction

$$\text{Sand}: \tau_f = \bar{\sigma}_f \tan\bar{\phi}$$

$$\text{Clays}: \tau_f = \bar{\text{c}} + \bar{\sigma}_f \tan\bar{\phi}$$

Strength in Effective stresses



Sand : ($\bar{\phi}$)

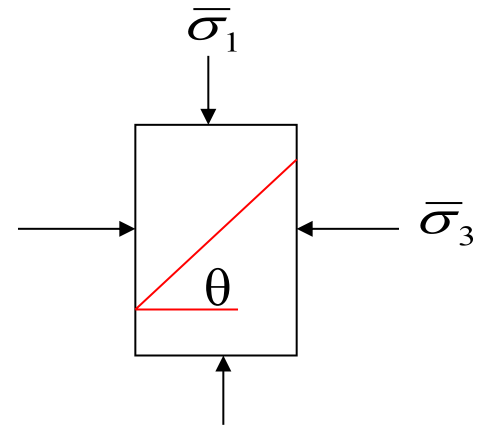
$$\tau_f = \bar{\sigma}_f \tan \bar{\varphi}$$

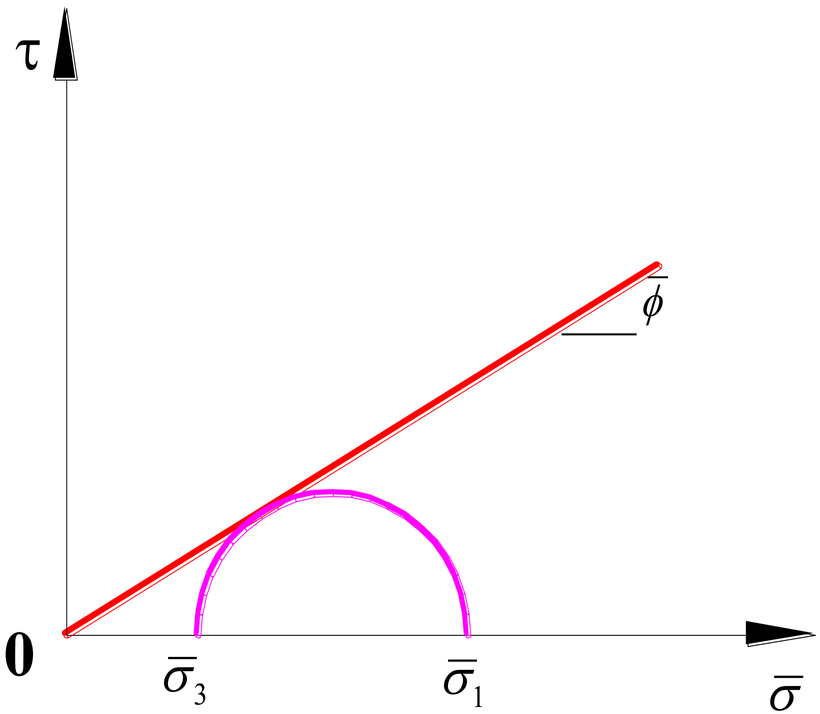
Clay : ($\bar{c}$, $\bar{\phi}$)

$$\tau_f = \bar{c} + \bar{\sigma}_f \tan \bar{\varphi}$$

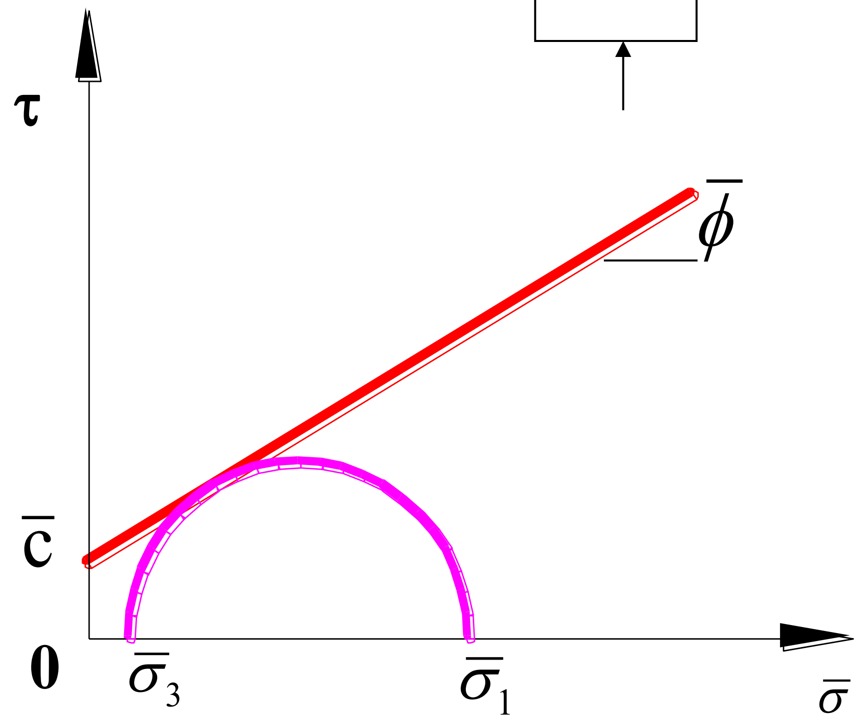
Strength in Effective stresses

(Compression)



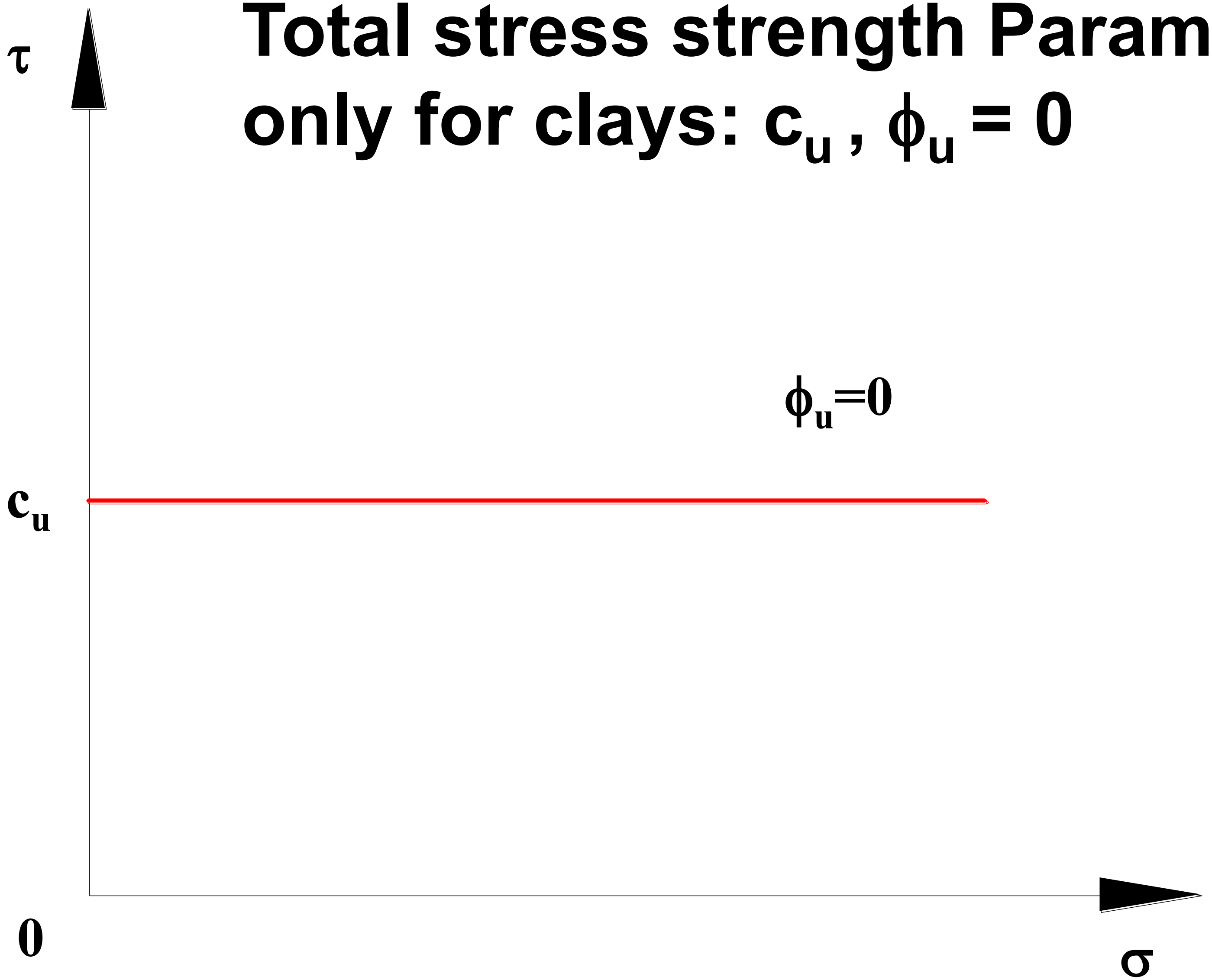


Sand : ($\bar{\phi}$)

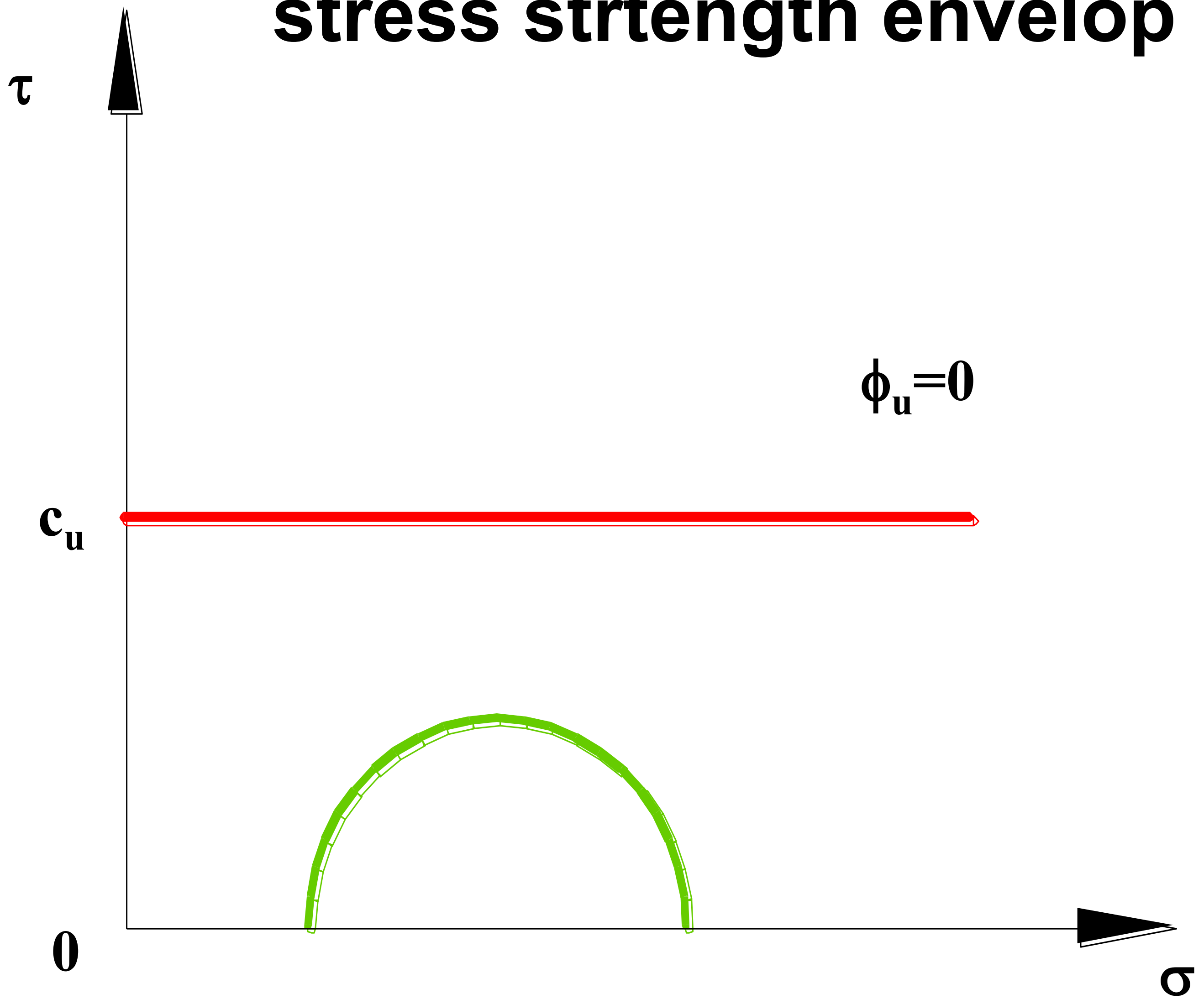


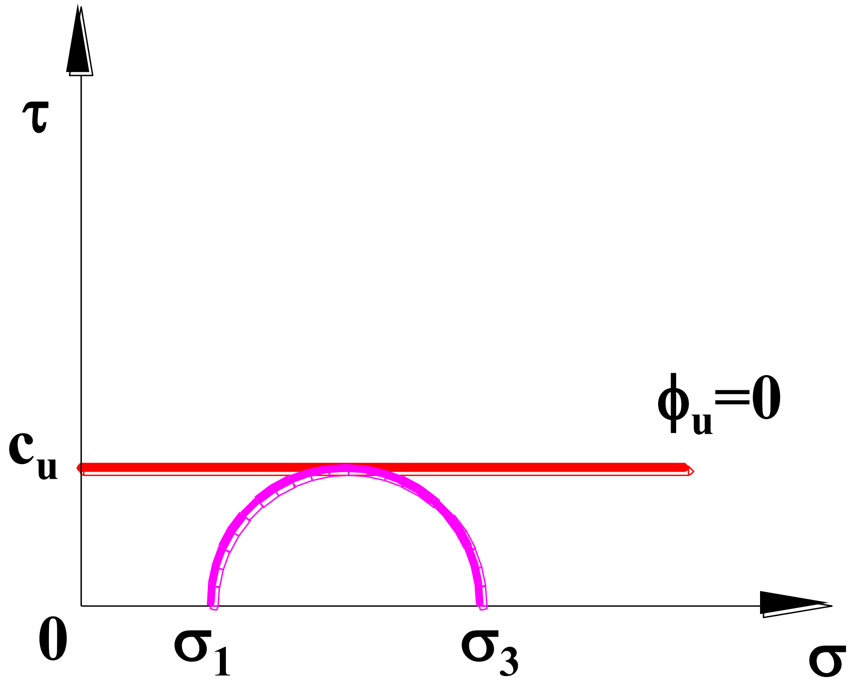
Clay : ($\bar{c}$, $\bar{\phi}$)

Total stress strength Parameters only for clays: c_u , $\phi_u = 0$



Safe conditions under total stress strtength envelop

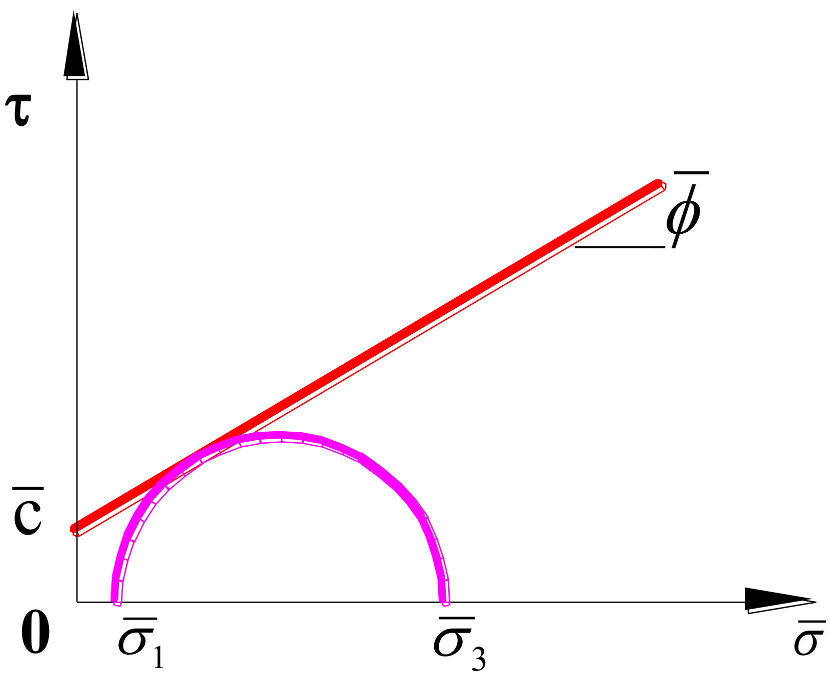




Clay

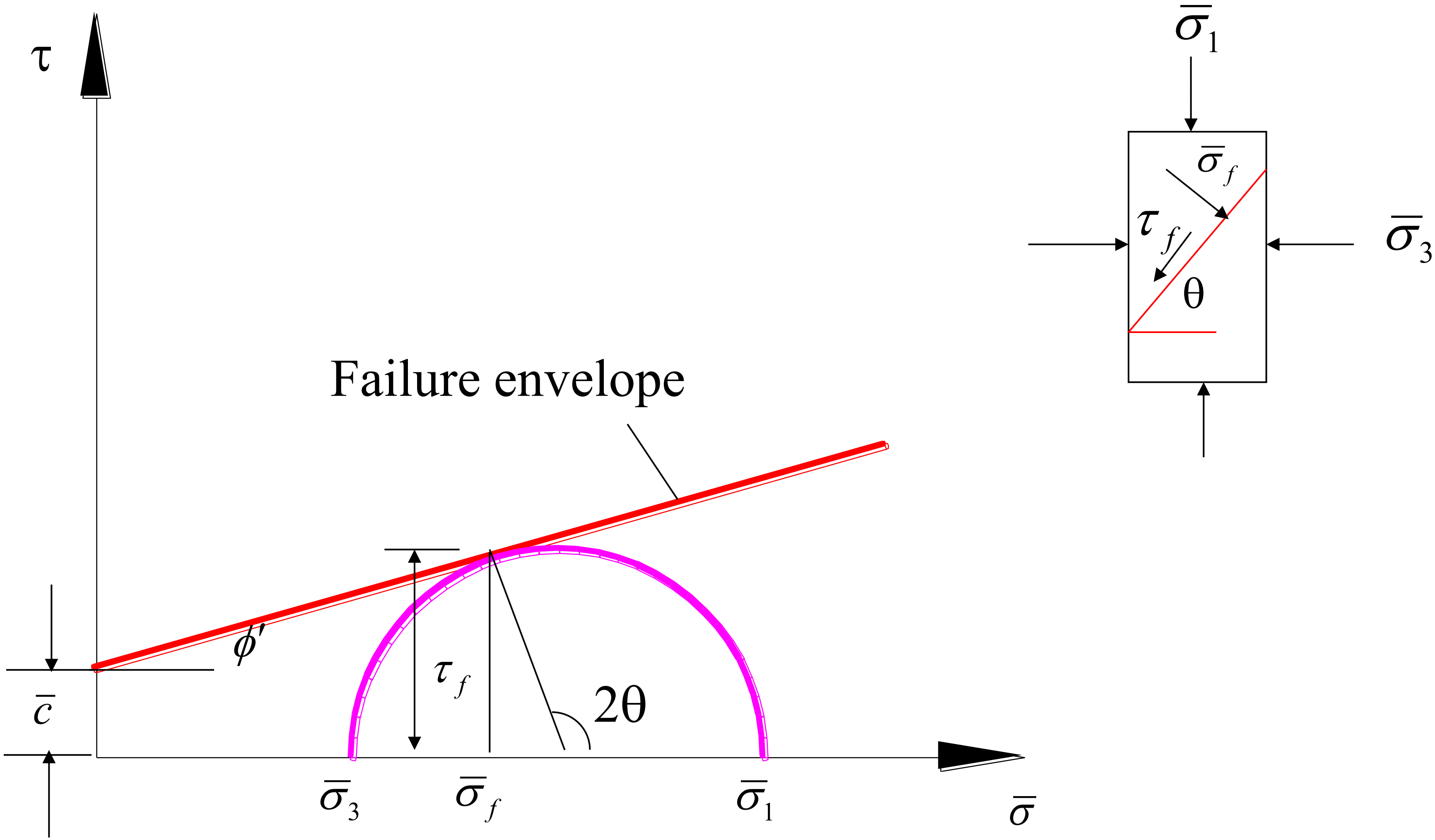
Total stress analysis:

At failure

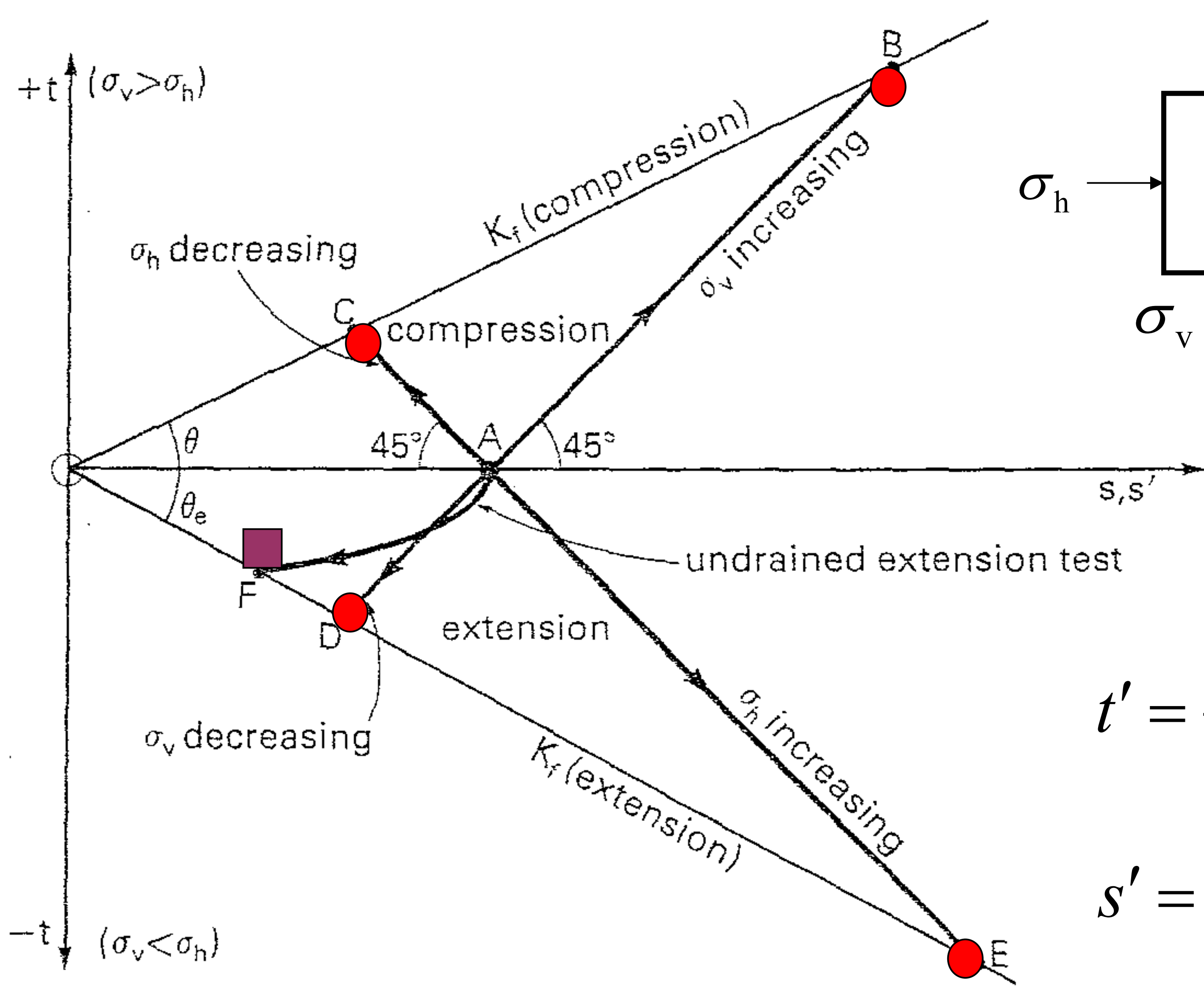


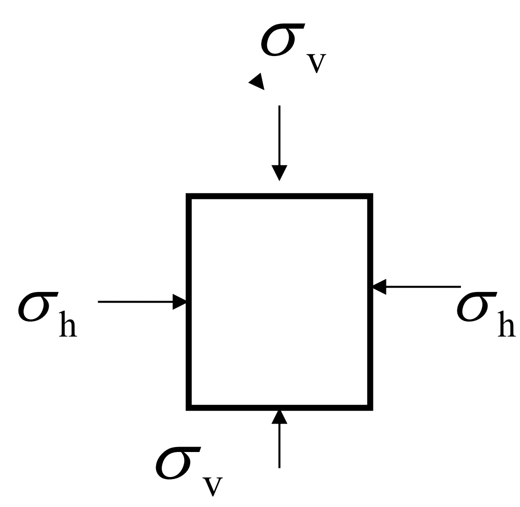
Effective stress analysis:

At failure



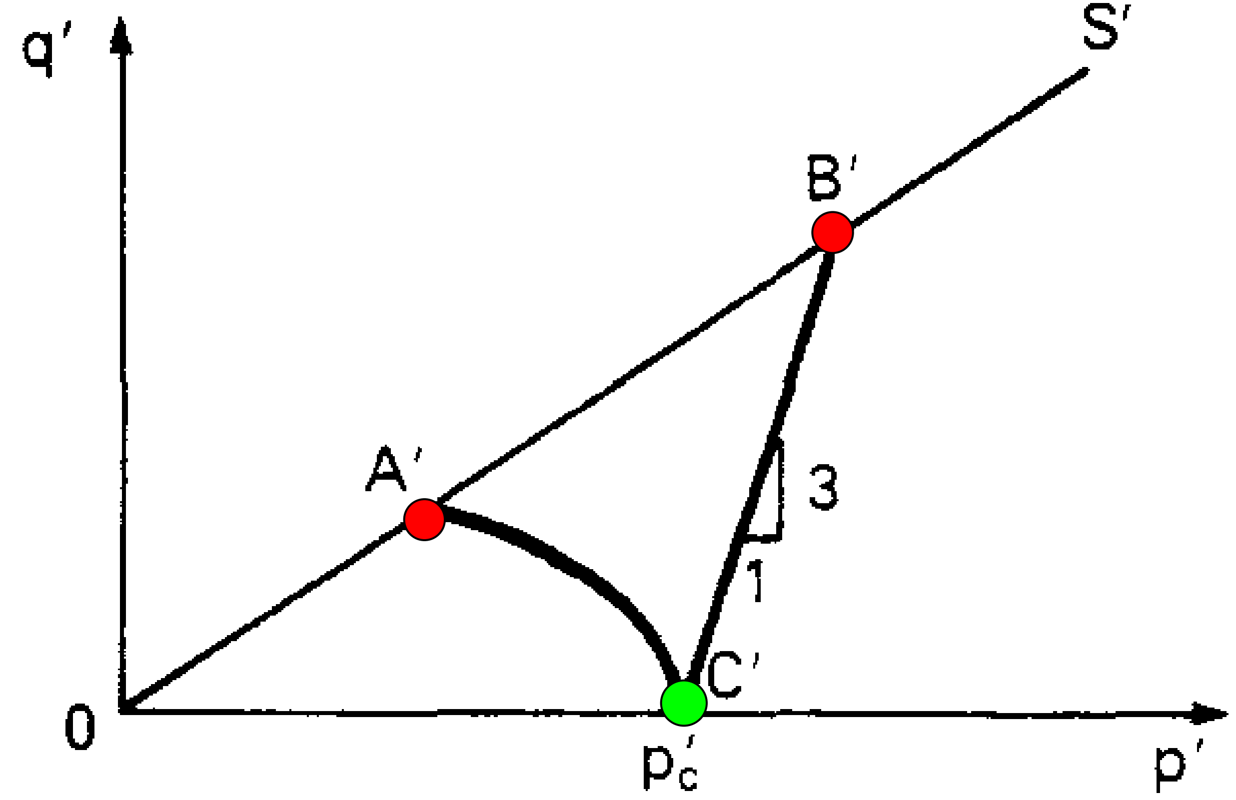
τ
Failure envelope
ϕ'
$\bar{c}$
τ_f
2θ
$\bar{\sigma}_3$
$\bar{\sigma}_f$
$\bar{\sigma}_1$
$\bar{\sigma}$
$\bar{\sigma}_1$
$\bar{\sigma}_f$
τ_f
θ
$\bar{\sigma}_3$





$$t' = \frac{1}{2}(\sigma_1' - \sigma_3')$$

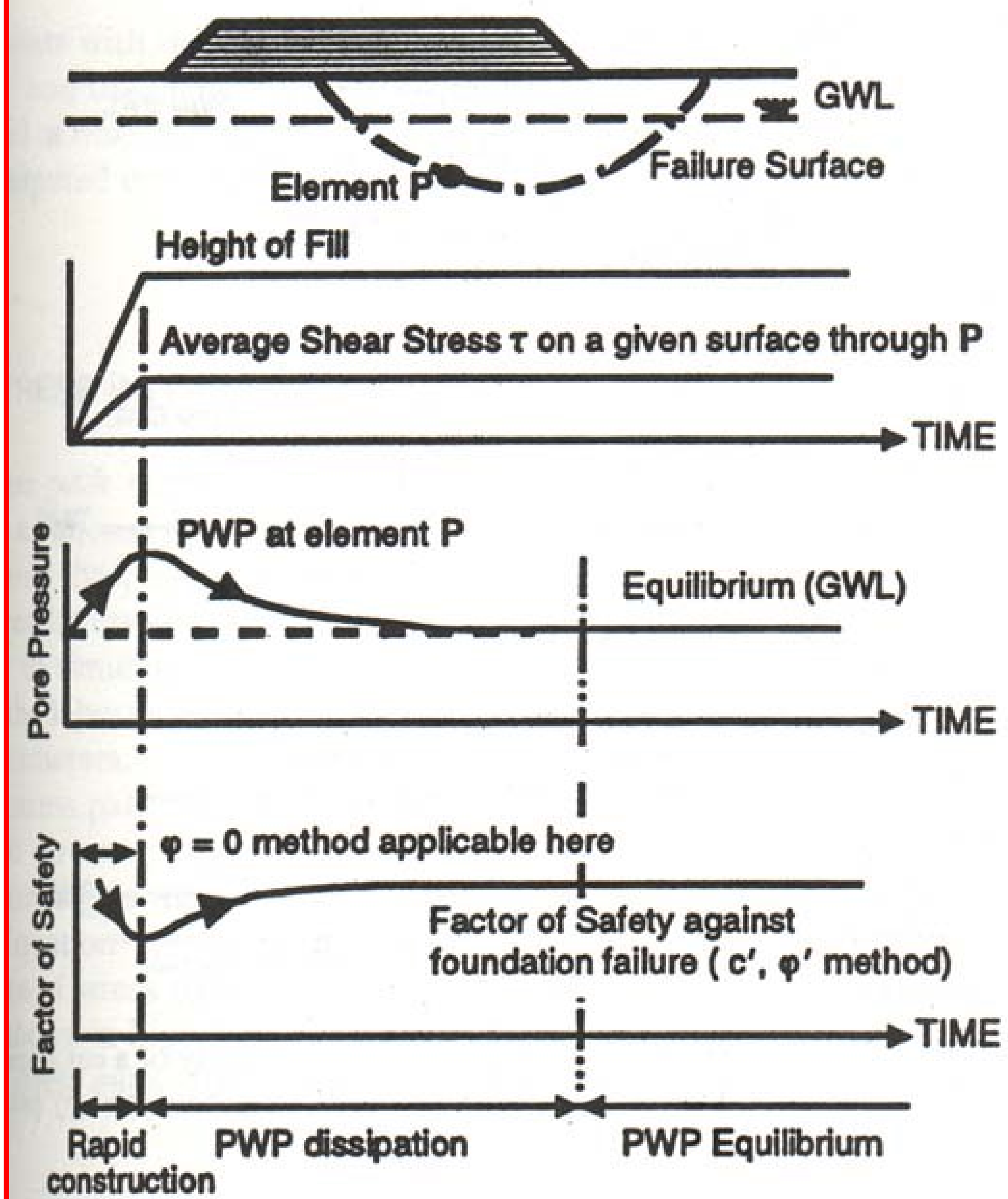
$$s' = \frac{1}{2}(\sigma_1' + \sigma_3')$$



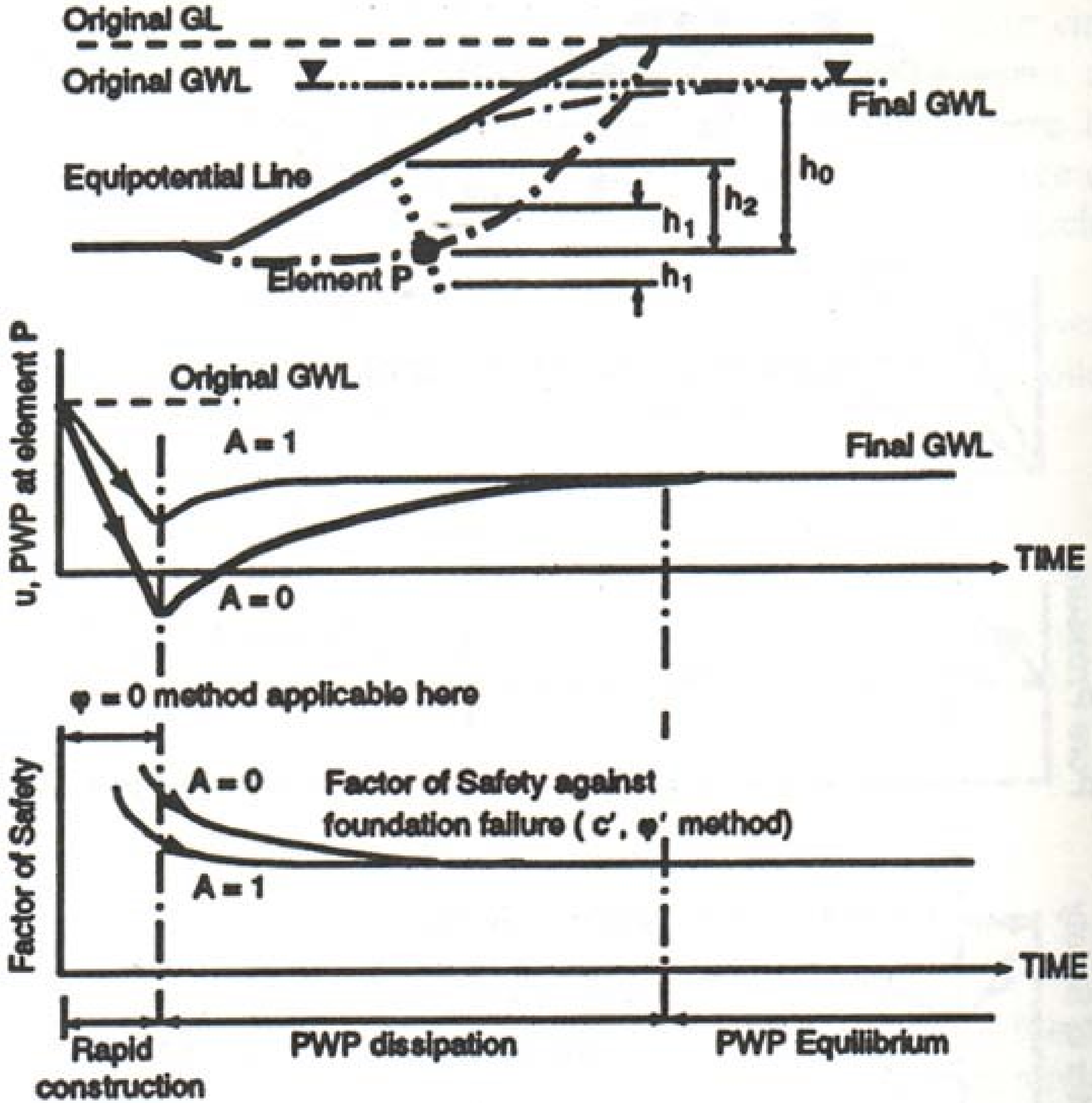
(a)

$$q' = \left(\sigma_1' - \sigma_3'\right)$$

$$p' = \frac{1}{3}\left(\sigma_1' + 2\sigma_3'\right)$$



GWL
Failure Surface
Element P
Height of Fill
Average Shear Stress τ on a given surface through P
TIME
Pore Pressure
PWP at element P
Equilibrium (GWL)
TIME
Factor of Safety
φ = 0 method applicable here
Factor of Safety against
foundation failure (c′, φ′ method)
TIME
Rapid
construction
PWP dissipation
PWP Equilibrium


Original GL
Original GWL
Final GWL
Equipotential Line
h_0
h_2
h_1
Element P
h_1
u, PWP at element P
Original GWL
A = 1
Final GWL
TIME
A = 0
φ = 0 method applicable here
Factor of Safety
A = 0
Factor of Safety against
foundation failure (c′, φ′ method)
A = 1
TIME
Rapid
construction
PWP dissipation
PWP Equilibrium

Total stress analysis:
Limit equilibrium analysis

This analysis, in terms of total stress, covers the case of a fully saturated clay under undrained conditions. That is for the condition immediately after construction.

For this analysis, the undrained strength of the sub-soil is taken as the in-situ strength prior to the construction activities and since the time is short there is no increase or decrease in strength due to the pore pressures developed during the construction stage.

Only the moment equilibrium is considered in the analysis.
In a cross-section where the failure is shown, the potential failure curve is assumed to be a circular arc. A trial failure surface (with center O, radius r and length L) is shown in Fig. 1.

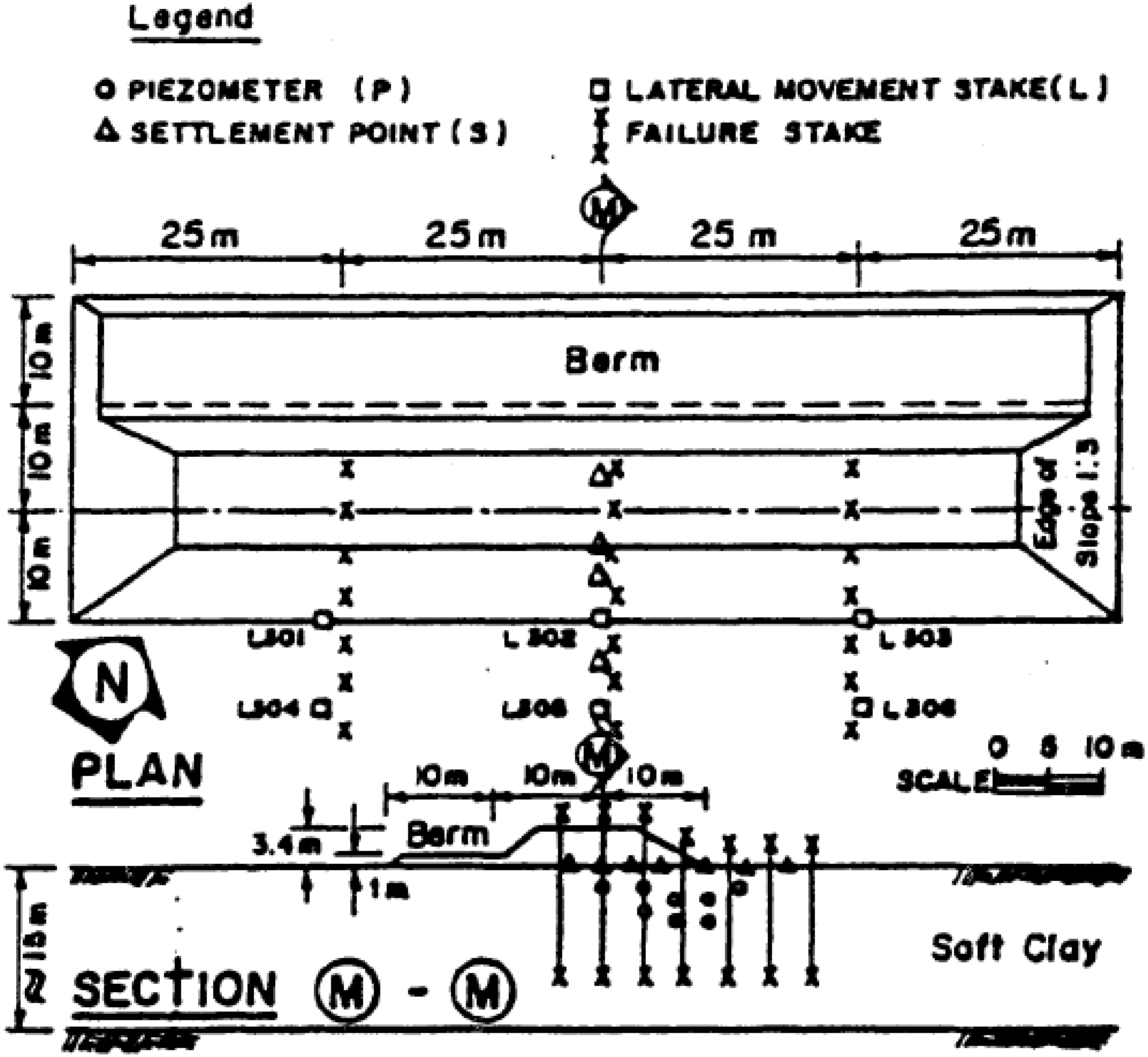


Fig. 1. Embankment I Taken to Failure.

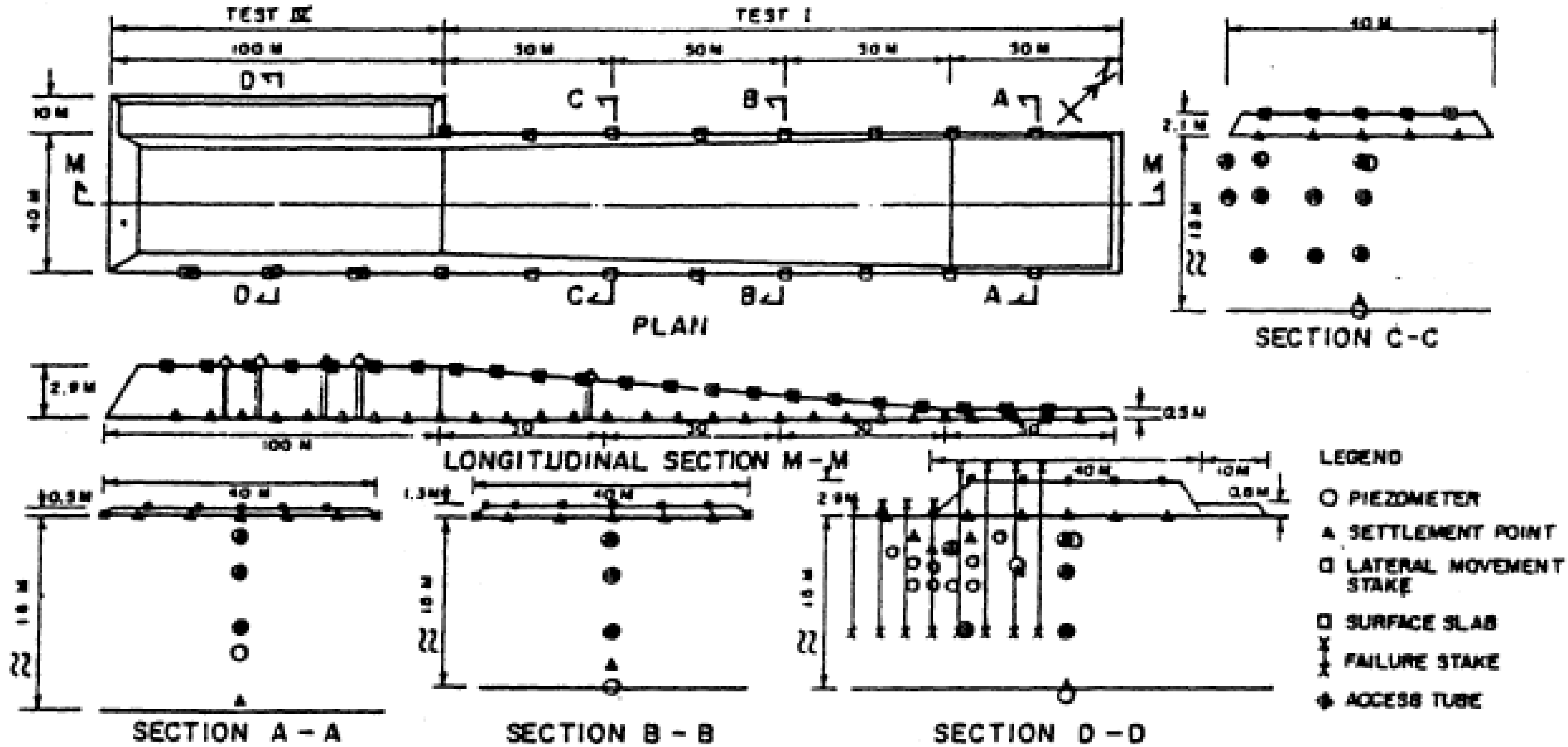


Fig. 2. Embankment II for Long term Settlements.

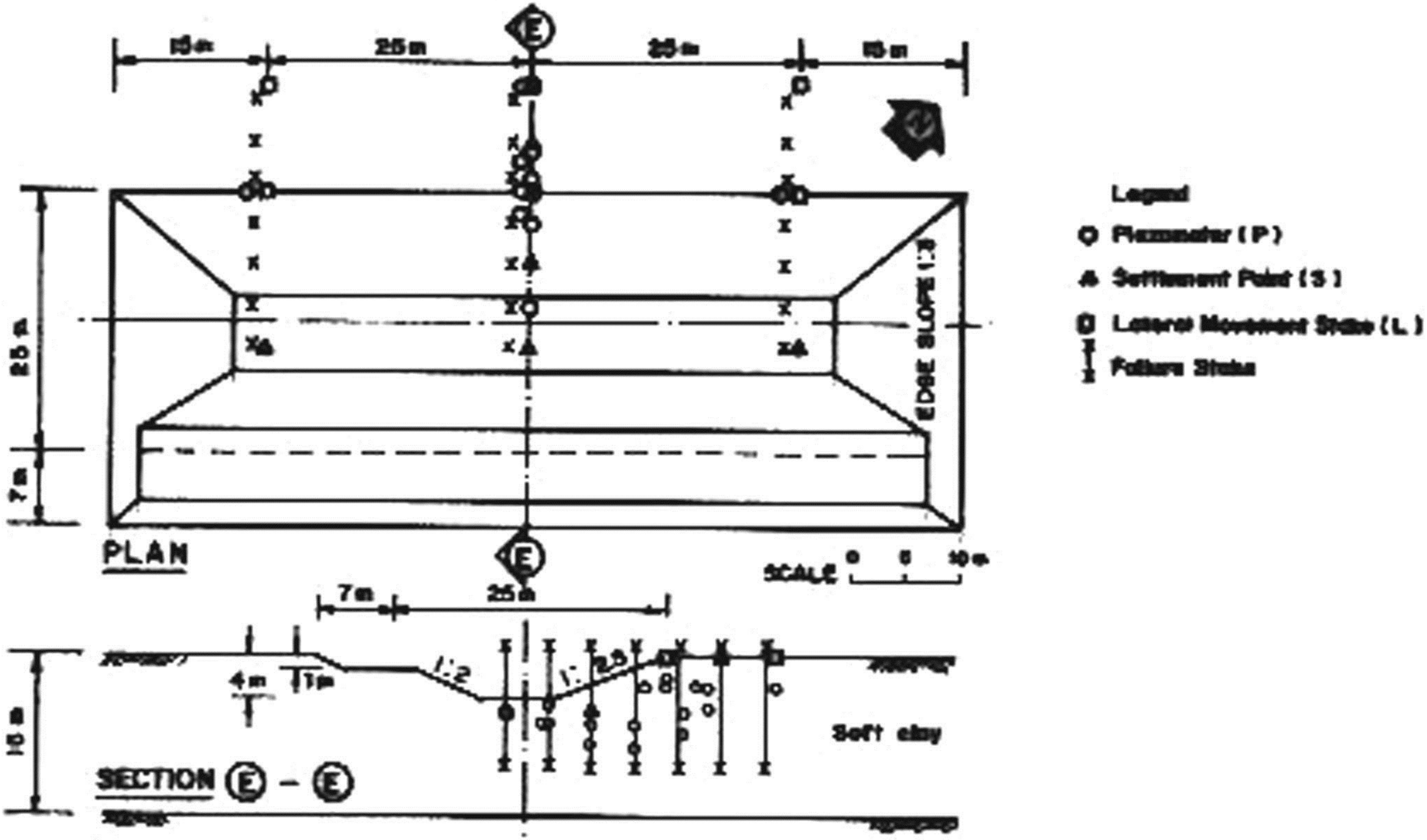
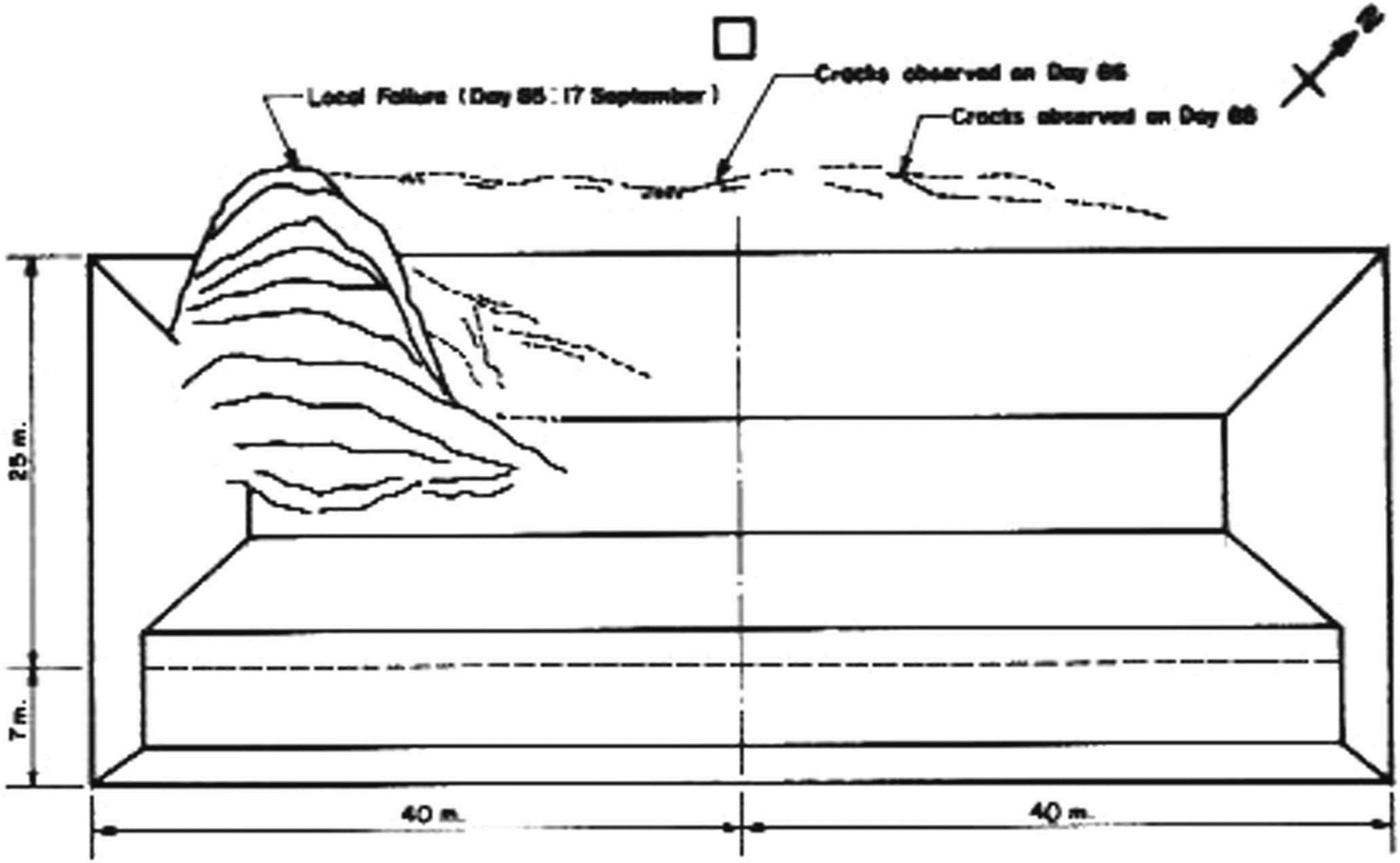


Fig. 14 Full scaled test excavation at Nong Ngoo Hao



Test : Plan of Visible Cracking Prior to Failure

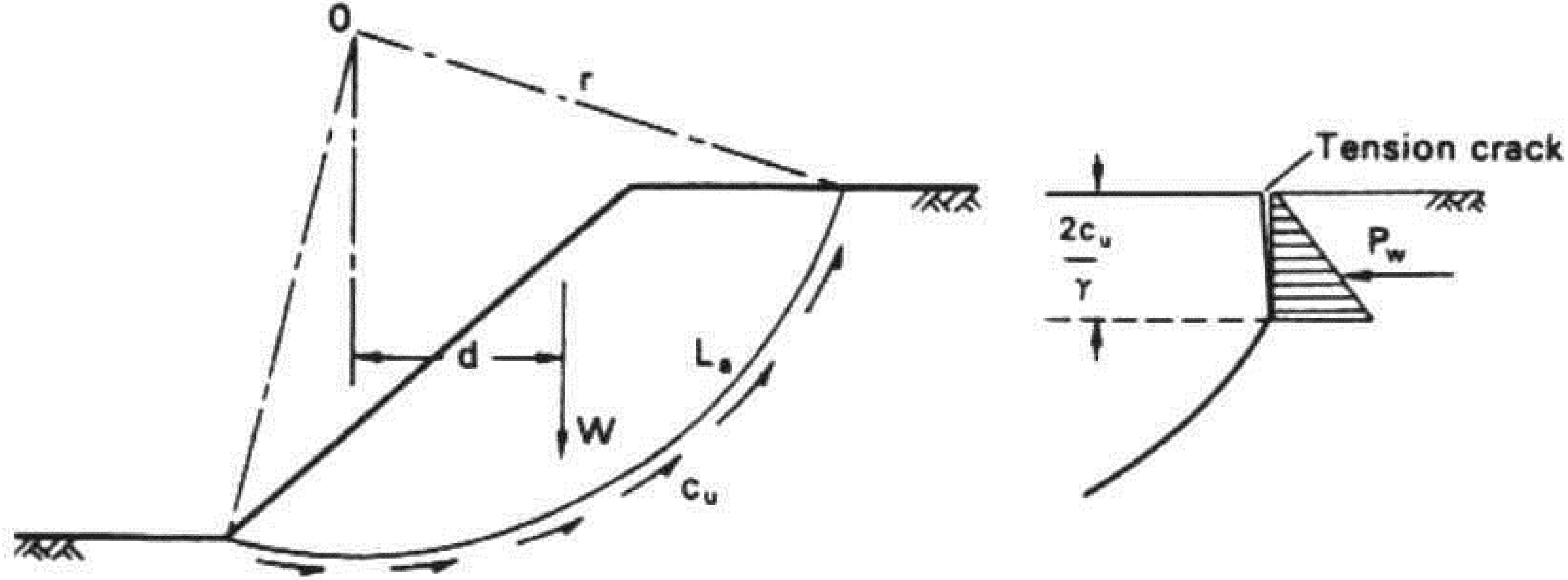


Fig.6.2 The ϕ_u = 0 analysis

Potential instability is due to the total weight of the soil mass (W per unit length) above the failure surface.

.For equilibrium the shear strength, which must be mobilized along the failure surface, is expressed as

$$\tau_m = \frac{\tau_f}{F} = \frac{c_u}{F}$$

F is the factor of safety with respect to shear strength.
Equating moments about O:

$$Wd = \frac{c_u}{F} Lr$$

Therefore

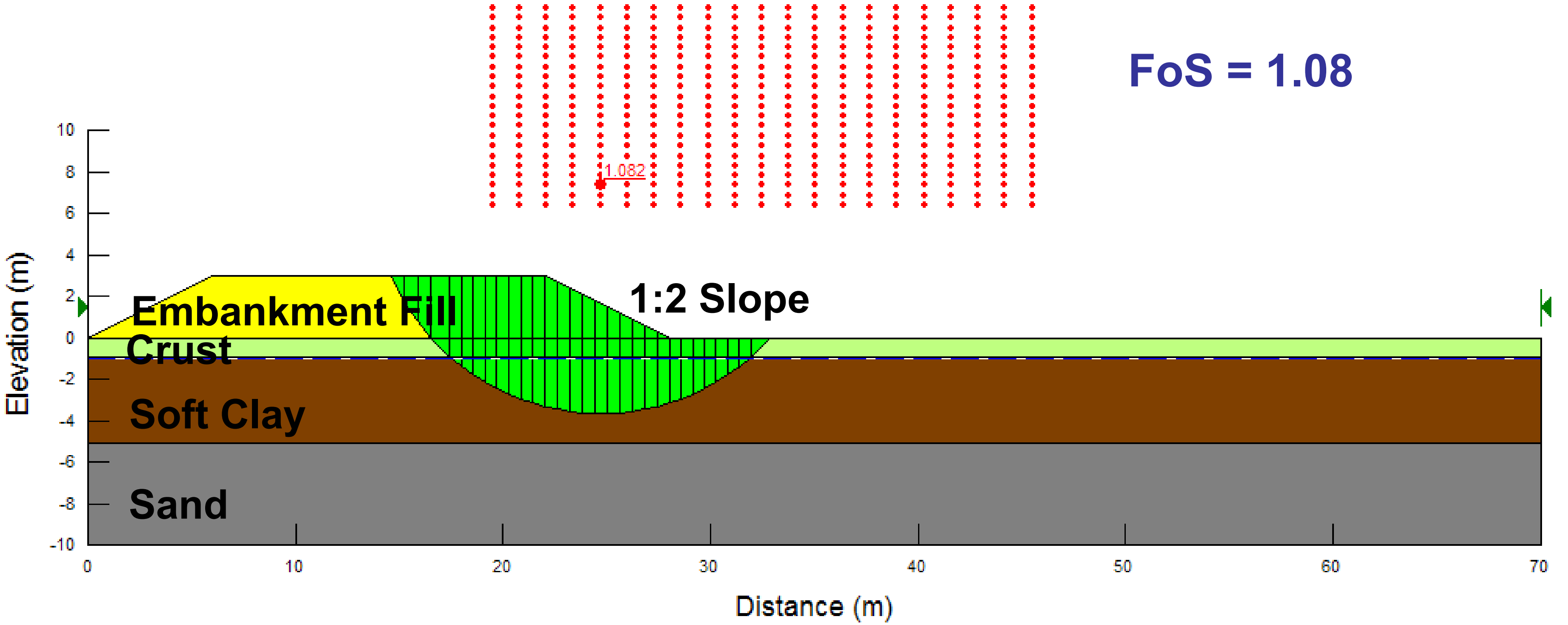
$$F = \frac{c_u Lr}{Wd}$$

Design Criteria for Embankment Stability

- Minimum FoS = 1.3

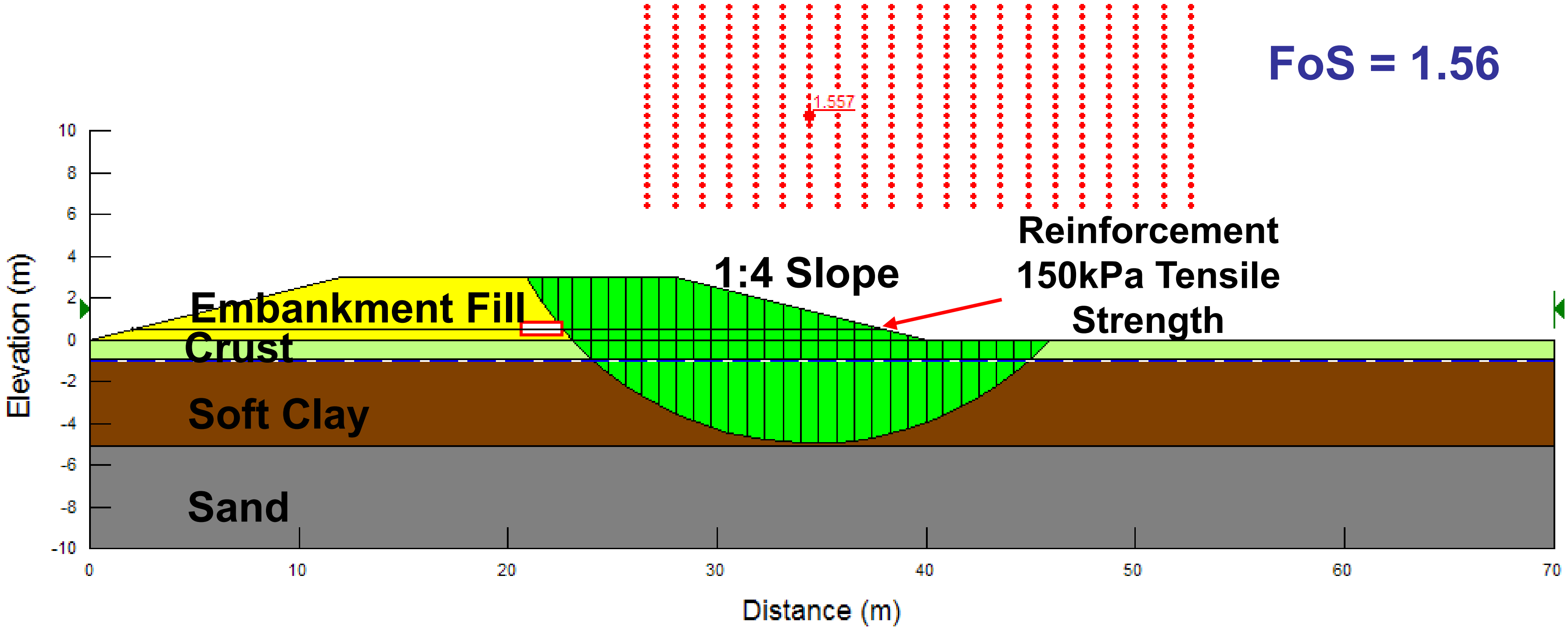
  at construction stage (short term)
- Minimum FoS = 1.5

  at service stage (Long term)
- Minimum FoS = 1.0 when reinforced embankment is analysed without reinforcement

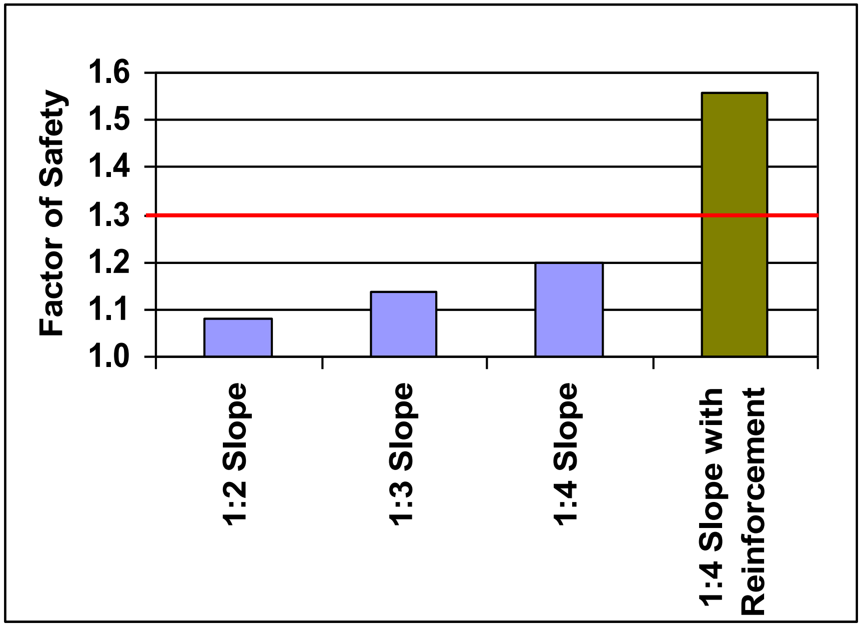


Soil	Model	Unit Weight (kN/m^3)	Phi (degree)	Cohesion (kN/m^2)	Height/Depth (m)
Embankment Fill	Mohr-Columb	21	30	5	3
Weathered Crust	Undrained	18	0	30	1
Soft Clay	Undrained	17	0	5.5-12	4
Sand	Mohr-Columb	17	30	0	5

FoS = 1.56

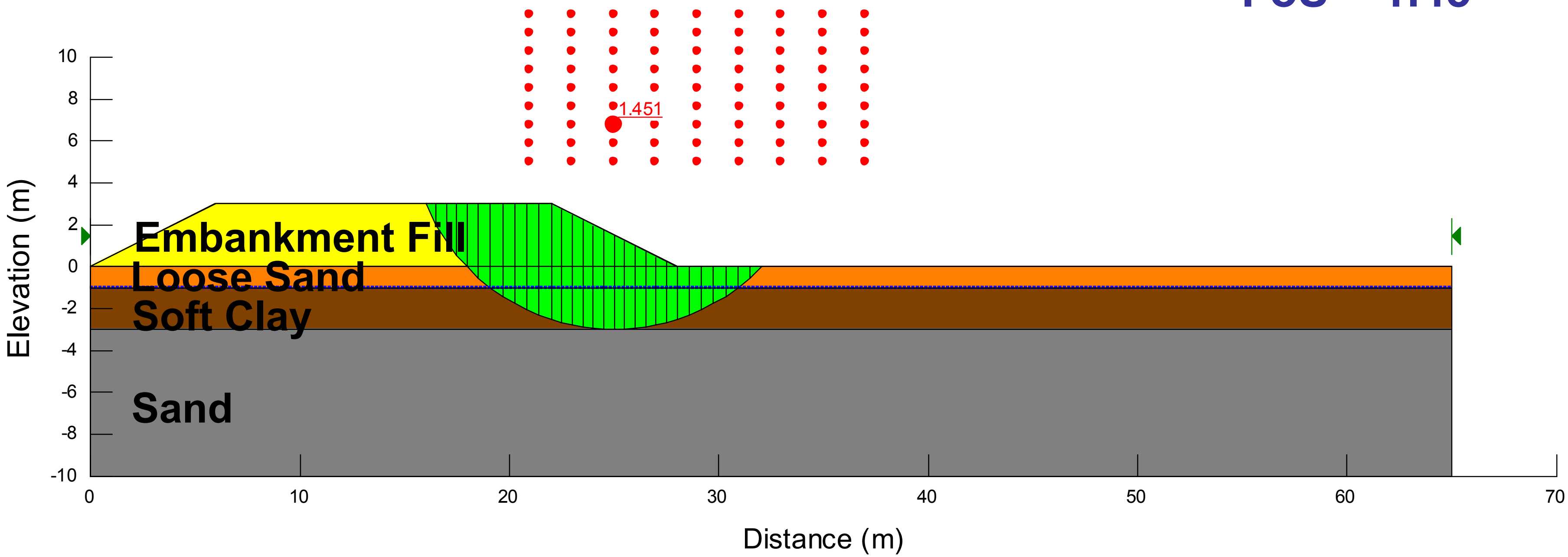


1.557
Reinforcement
150kPa Tensile Strength
1:4 Slope
Embankment Fill
Crust
Soft Clay
Sand
Elevation (m)
Distance (m)
10
8
6
4
2
0
-2
-4
-6
-8
-10
0
10
20
30
40
50
60
70



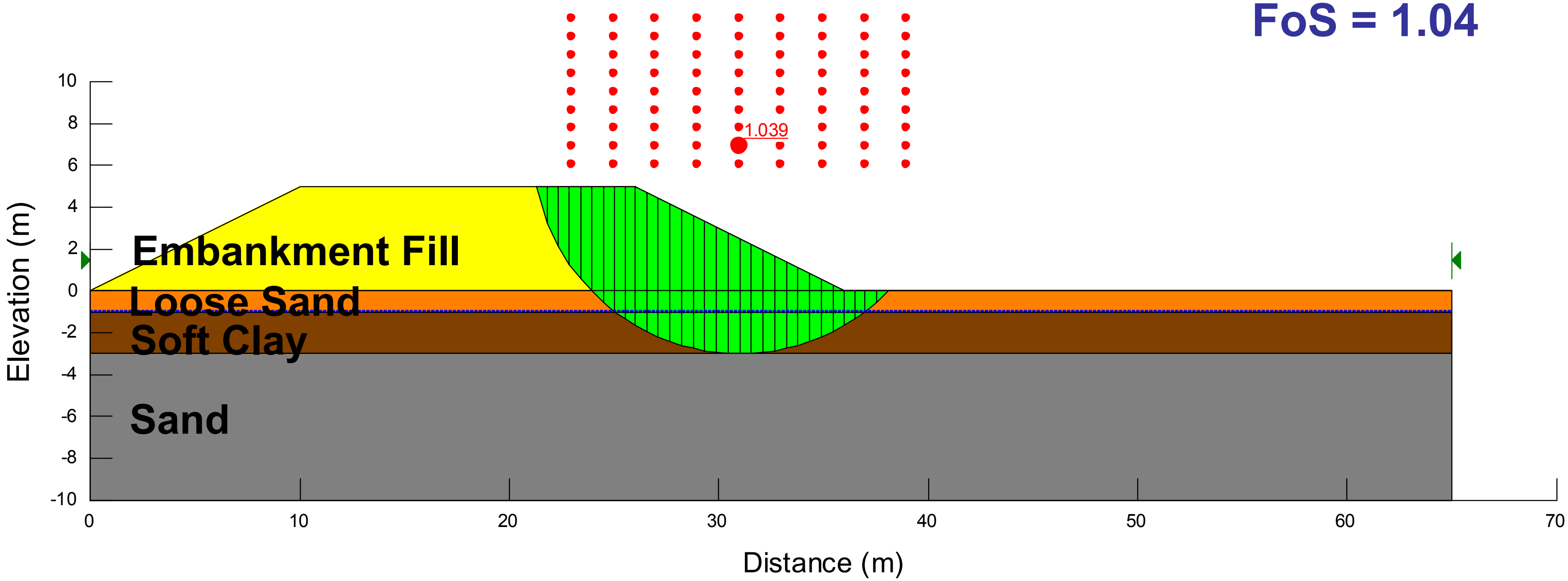
Factor of Safety
1.6
1.5
1.4
1.3
1.2
1.1
1.0
1:2 Slope
1:3 Slope
1:4 Slope
1:4 Slope with Reinforcement

FoS = 1.45

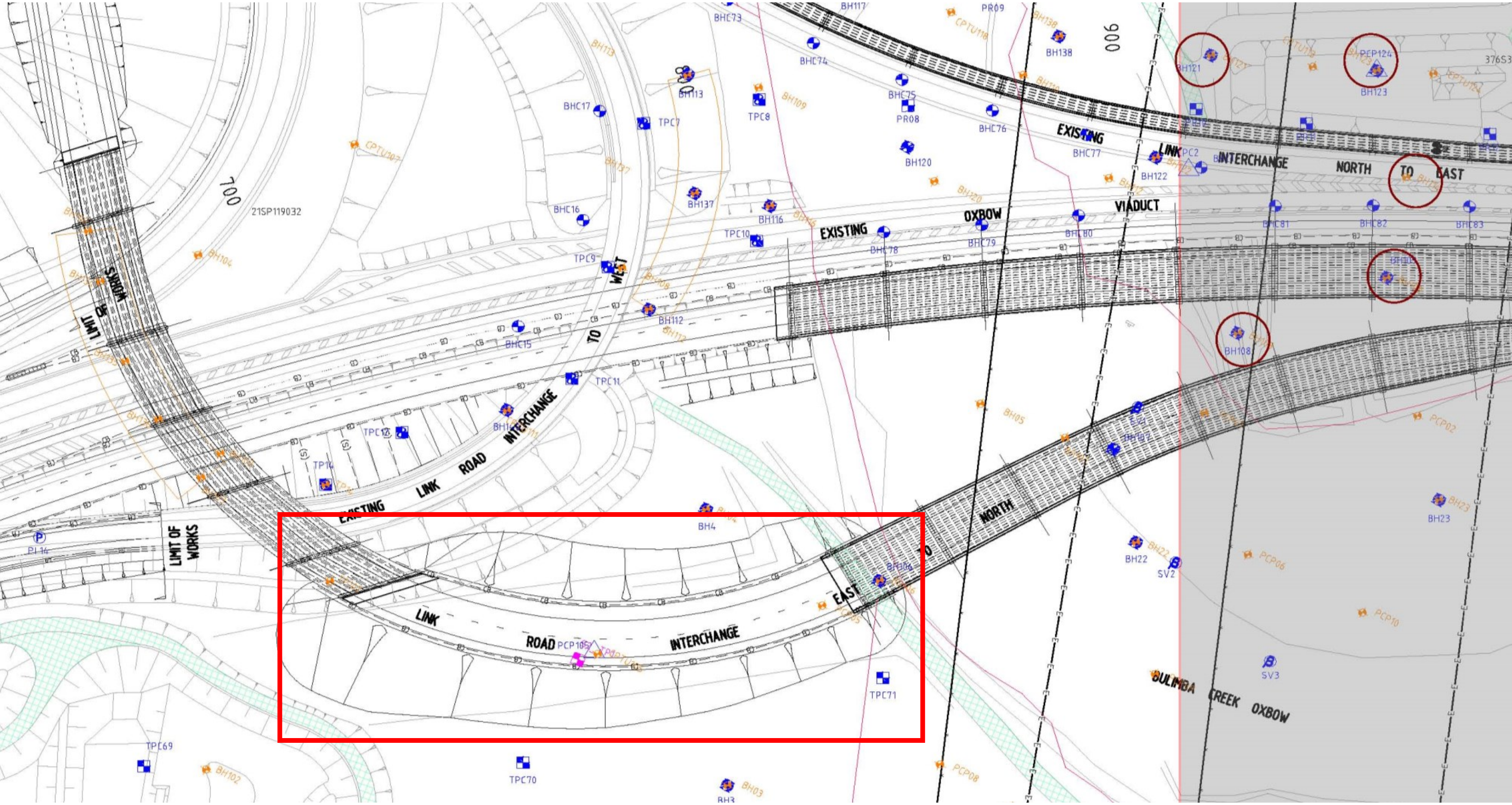


Soil	Model	Unit Weight (kN/m^3)	Phi (degree)	Cohesion (kN/m^2)	Height/Depth (m)
Embankment Fill	Mohr-Columb	21	30	5	3
Loose Sand	Mohr-Column	18	28	0	1
Soft Clay	Undrained	17	0	15	2
Sand	Mohr-Columb	17	30	0	7

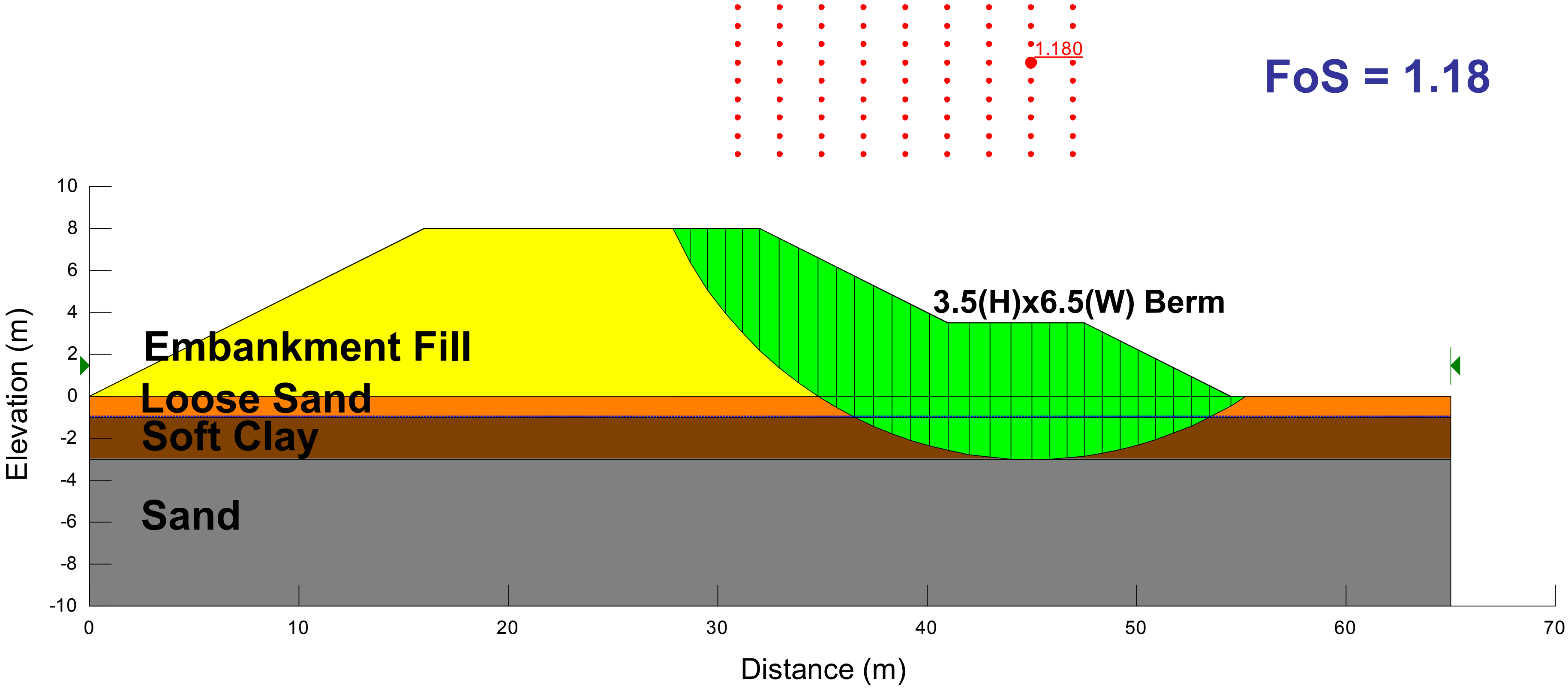
FoS = 1.04



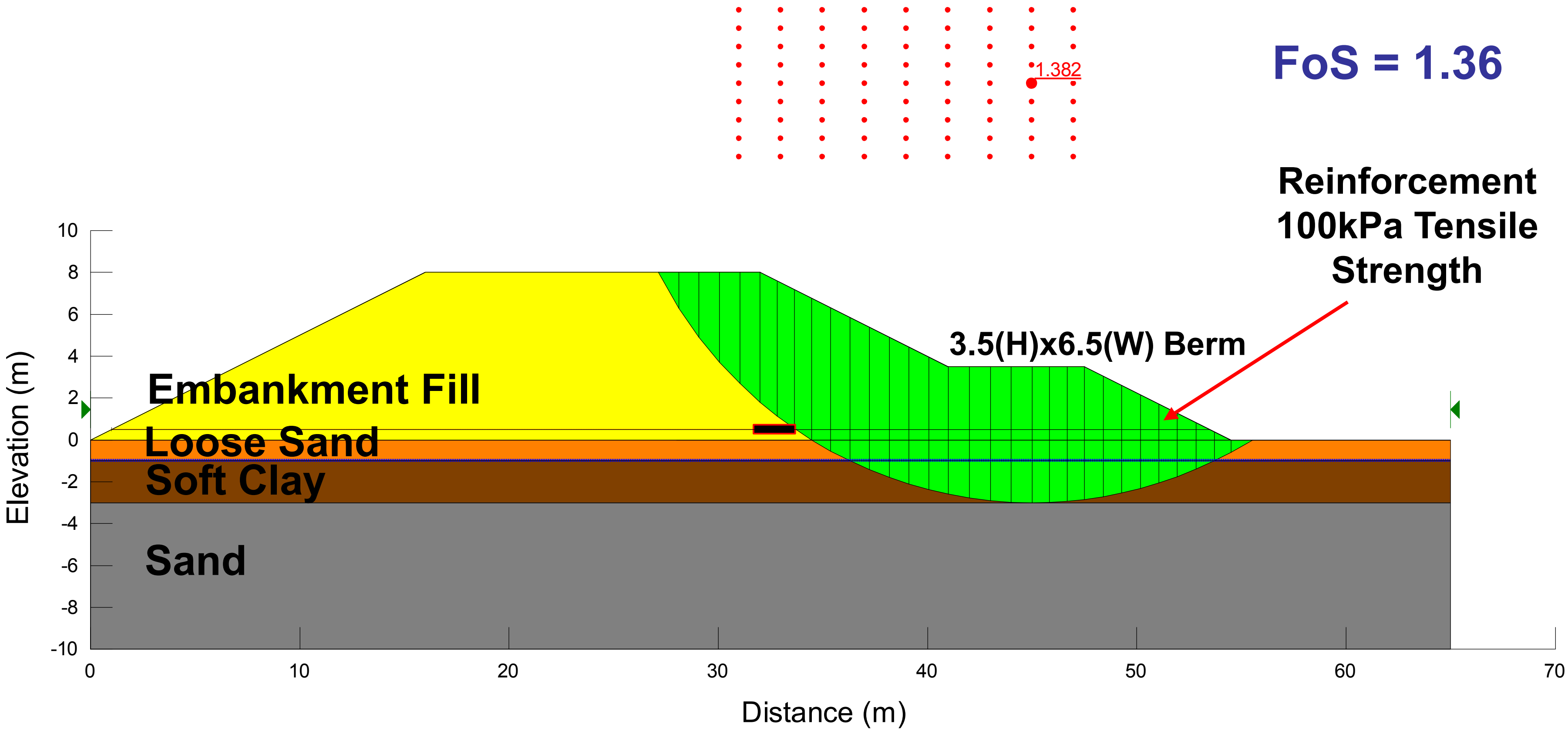
Soil	Model	Unit Weight (kN/m^3)	Phi (degree)	Cohesion (kN/m^2)	Height/Depth (m)
Embankment Fill	Mohr-Columb	21	30	5	5
Loose Sand	Mohr-Column	18	28	0	1
Soft Clay	Undrained	17	0	15	2
Sand	Mohr-Columb	17	30	0	7



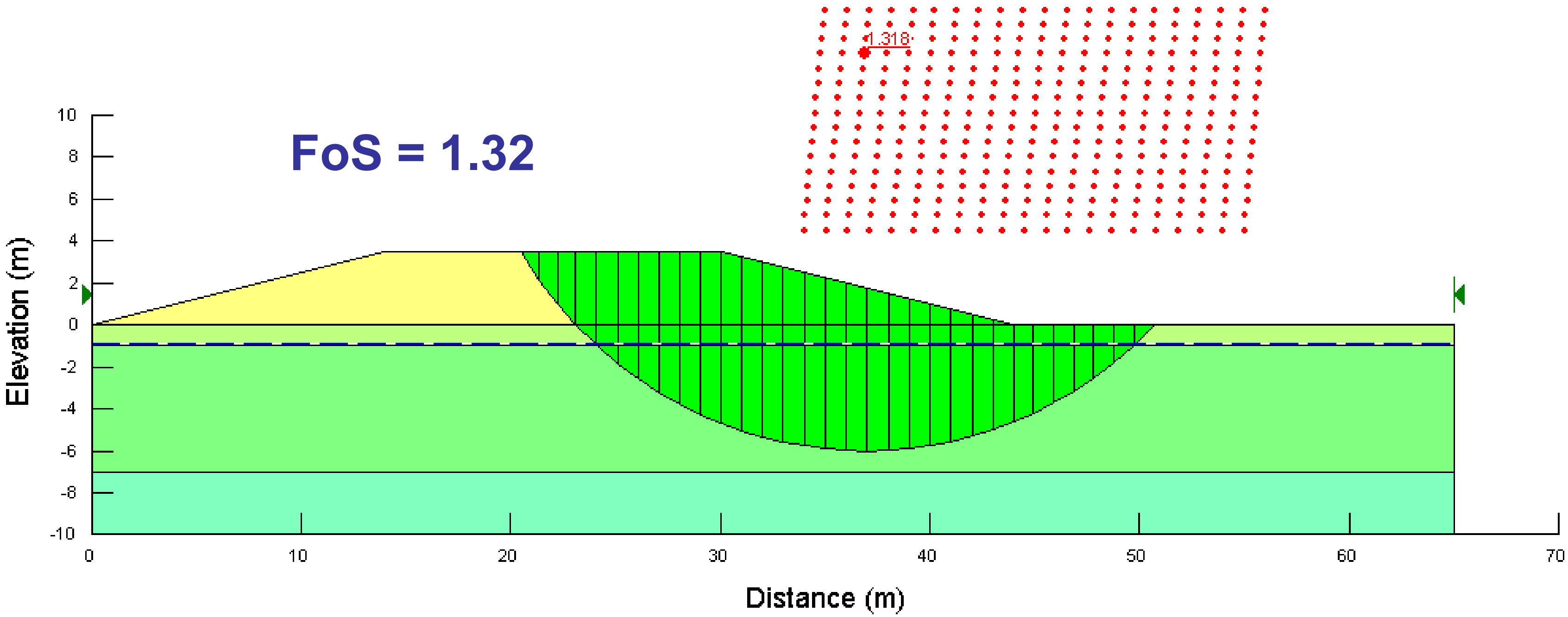
Approach embankment

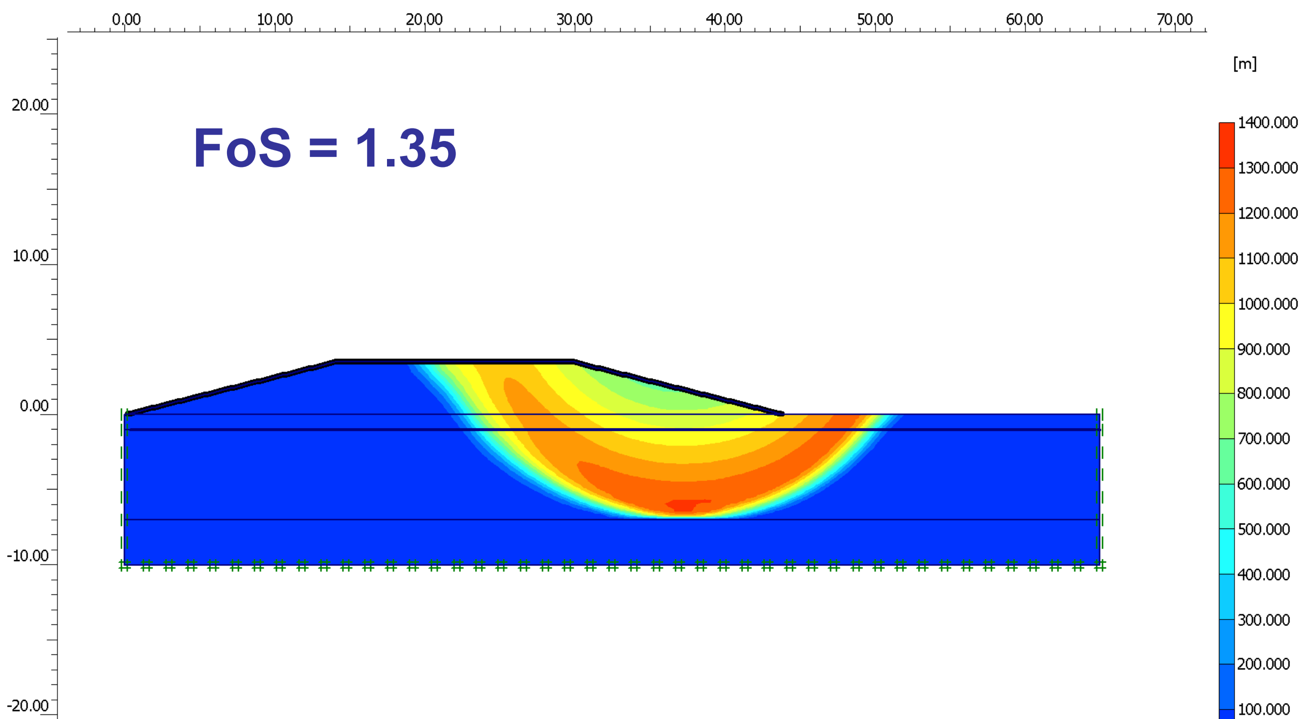


Soil	Model	Unit Weight (kN/m^3)	Phi (degree)	Cohesion (kN/m^2)	Height/Depth (m)
Embankment Fill	Mohr-Columb	21	30	5	8
Loose Sand	Mohr-Column	18	28	0	1
Soft Clay	Undrained	17	0	15	2
Sand	Mohr-Columb	17	30	0	7



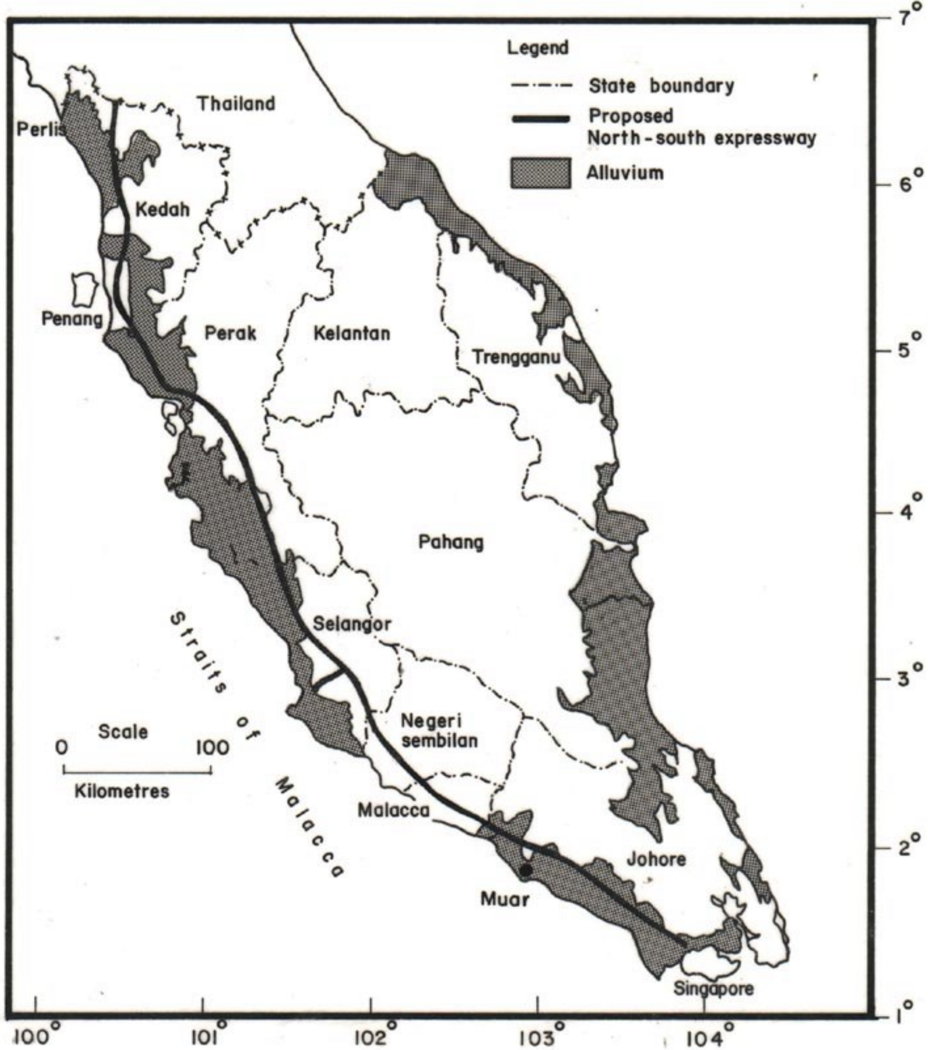
Soil	Model	Unit Weight (kN/m³)	Phi (degree)	Cohesion (kN/m²)	Height/Depth (m)
Embankment Fill	Mohr-Columb	21	30	5	8
Loose Sand	Mohr-Column	18	28	0	1
Soft Clay	Undrained	17	0	15	2
Sand	Mohr-Columb	17	30	0	7


FoS = 1.32
1.318
Elevation (m)
Distance (m)
10
8
6
4
2
0
-2
-4
-6
-8
-10
0
10
20
30
40
50
60
70


FoS = 1.35
0.00
10.00
20.00
30.00
40.00
50.00
60.00
70.00
20.00
10.00
0.00
-10.00
-20.00
[m]
1400.000
1300.000
1200.000
1100.000
1000.000
900.000
800.000
700.000
600.000
500.000
400.000
300.000
200.000
100.000

Embankment Built to Failure on Soft Clay

E.W.Brand



River and Coastal Alluvium of Peninsular Malaysia

Prof Lambe had a prediction symposium of an embankment in Boston Blue clay at the M.I.T. Campus

Prediction Type	When Prediction Made	Results known When Predictions Made
A	Before Event	-
B	During Event	No
B1	During Event	Yes
C	After Event	No
C1	After Event	Yes

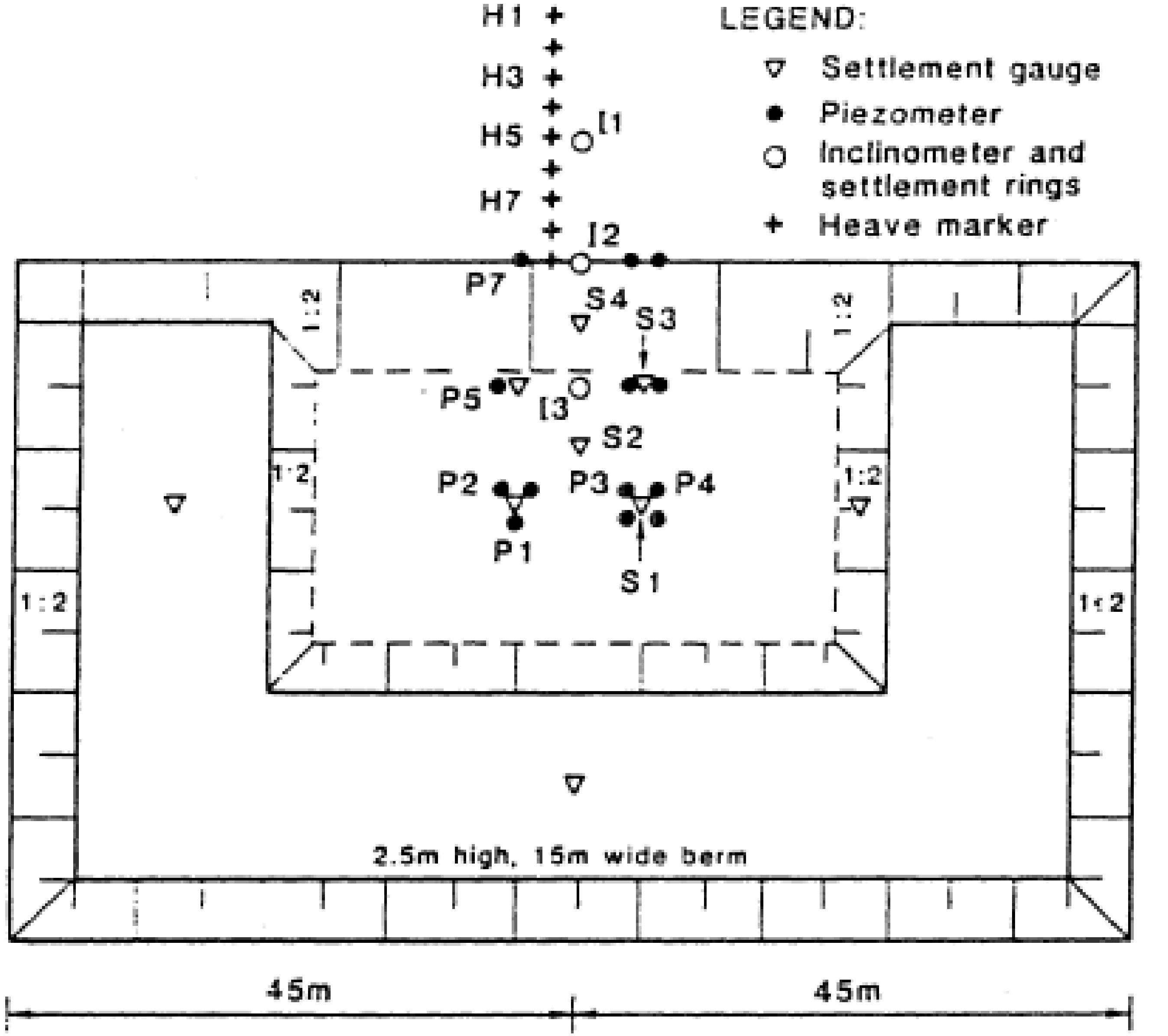


Fig. 3 The test embankment showing the positions of key instrumentation: (a) plan

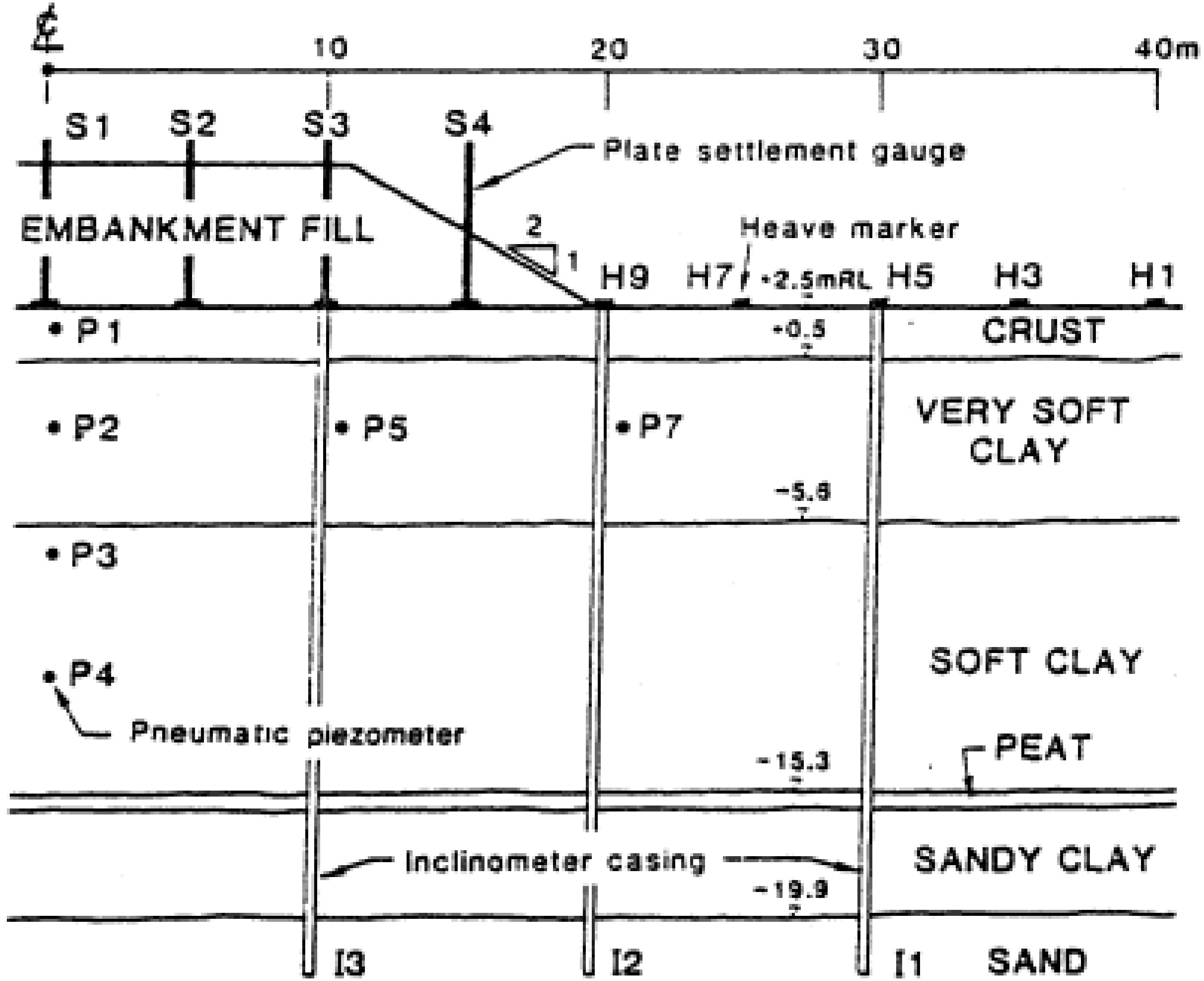


Fig. 3 The test embankment showing the positions of key instrumentation: (b) cross-section through centre

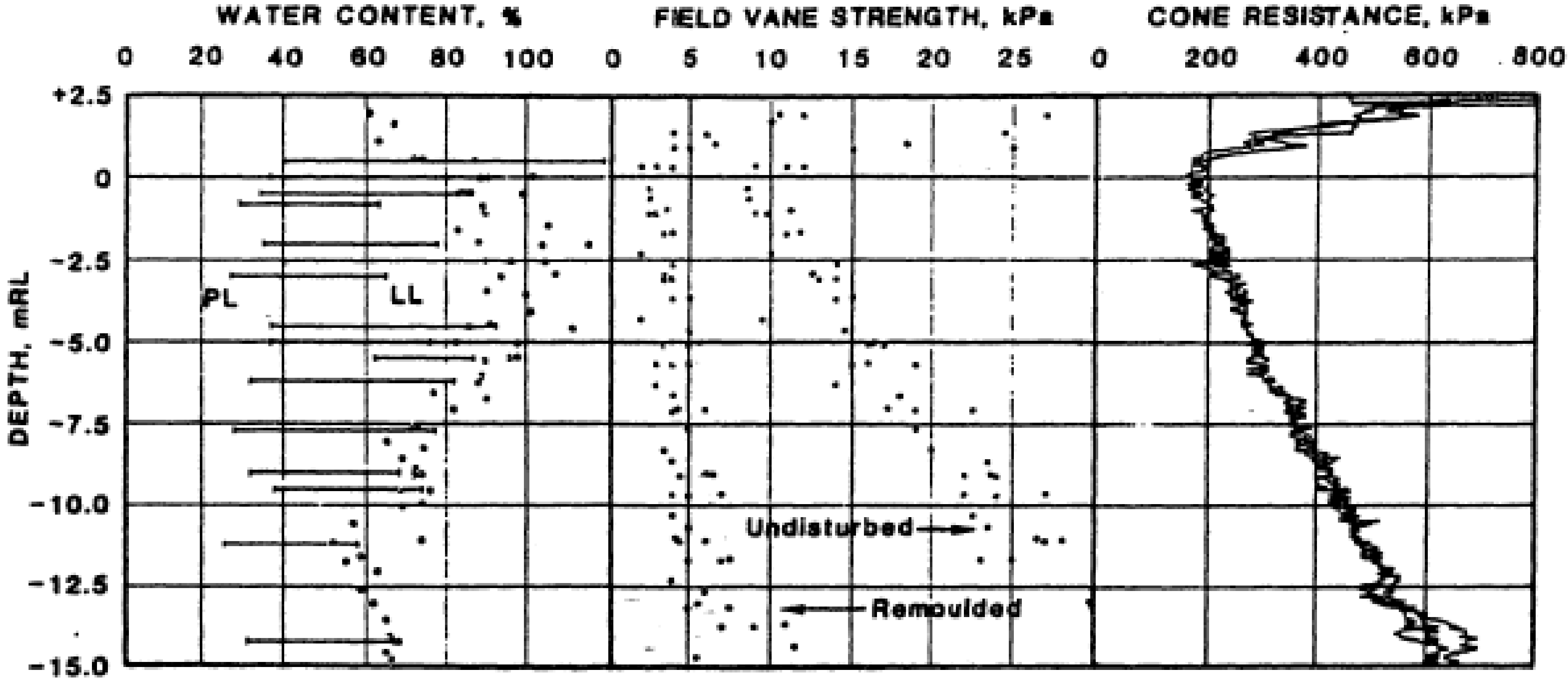
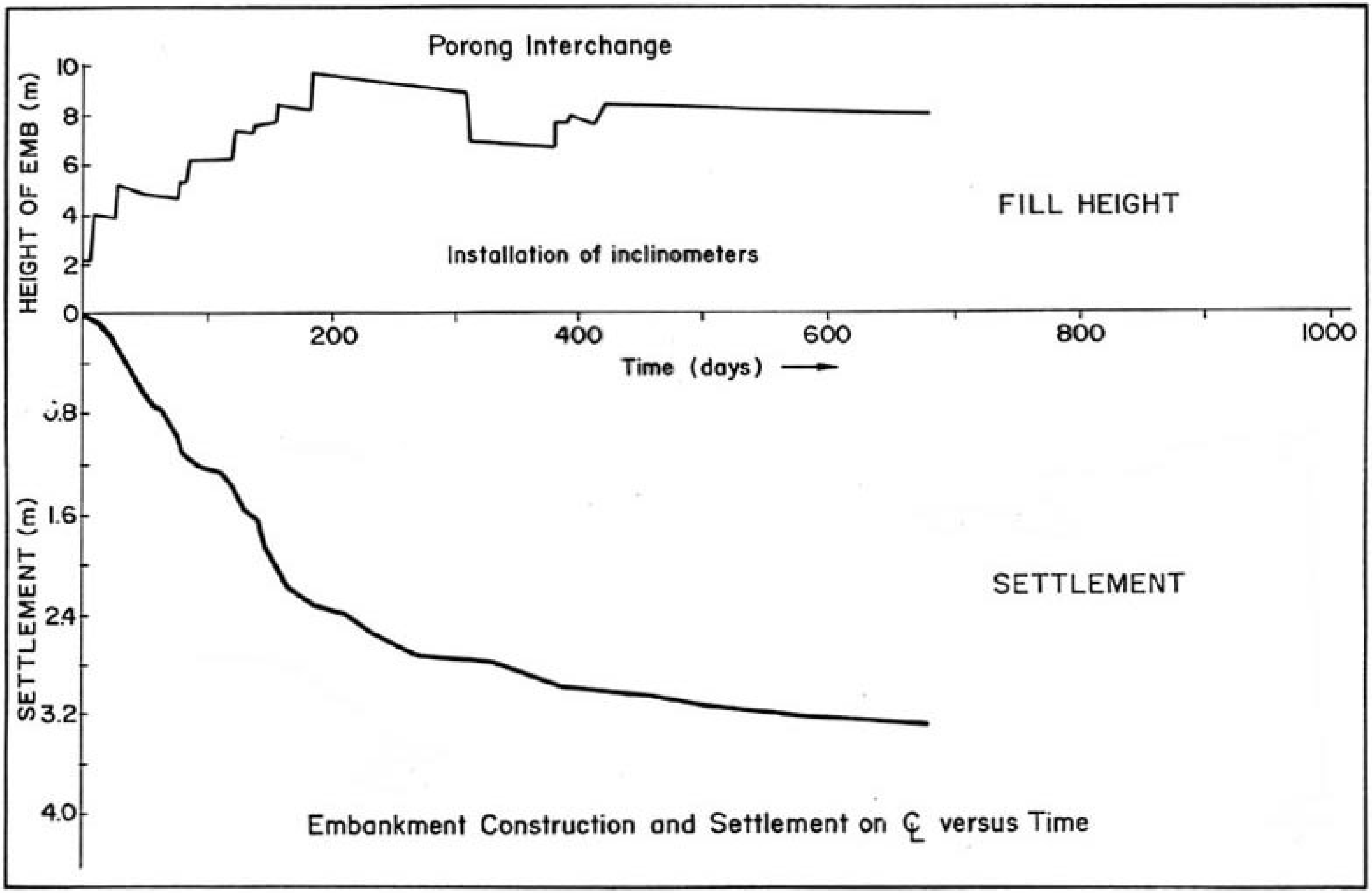


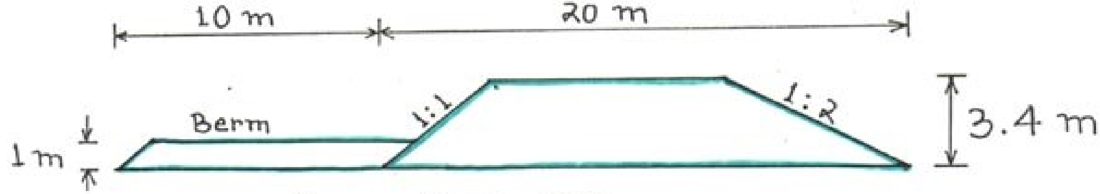
Fig. 2 Water contents, Atterberg Limits, vane strengths and Dutch cone resistances at the test site

DEPTH, m +2.5 mRL	SOIL DESCRIPTION		p_c (kPa)	$\frac{C_c}{1+e_o}$	k_h (m/sec)
+0.5	CRUST	Yellowish brown mottled red CLAY with roots, root holes and laterite concretions	110	0.3	-
-5.6	UPPER CLAY	Light greenish grey CLAY with a few shells, very thin discontinuous sand partings, occasional near-vertical roots and some decaying organic matter (<2%)	40	0.5	4×10^{-9}
-15.3	LOWER CLAY	Grey CLAY with some shells, very thin discontinuous sand partings and some decaying organic matter (<2%)	60	0.3	1×10^{-9}
-15.9	PEAT	Dark brown PEAT with no smell			
-19.9	SANDY CLAY	Greyish brown sandy CLAY with a little decaying organic matter	60	0.1	2×10^{-7}
	SAND	Dark grey, very silty medium-to-coarse SAND (SPT >20)	-	-	-

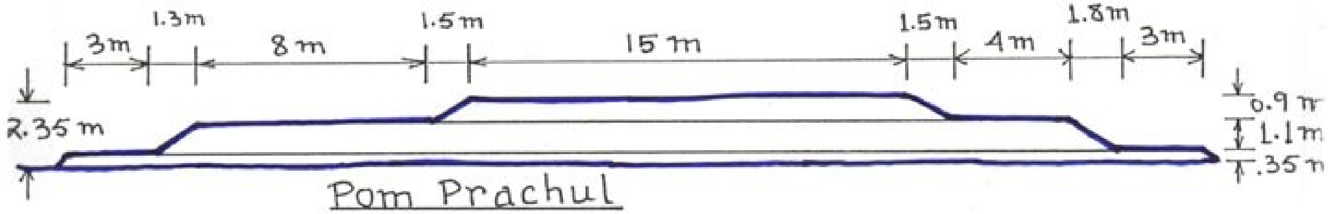
Fig. 1 Soil profile and some soil properties at the site of the Muar test embankment



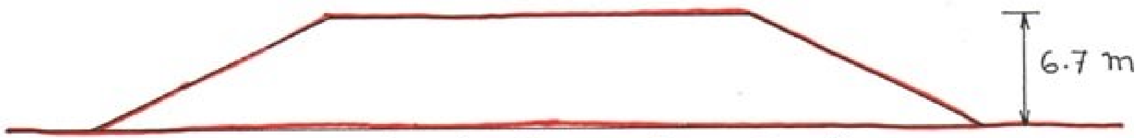
Settlement v Time



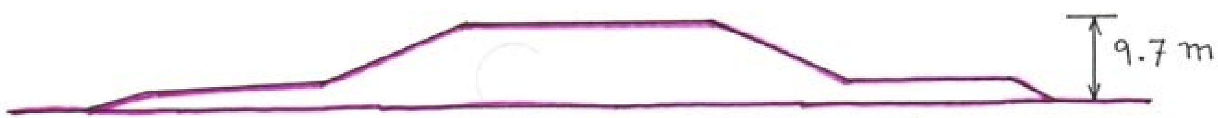
Nong Ngoo Hao



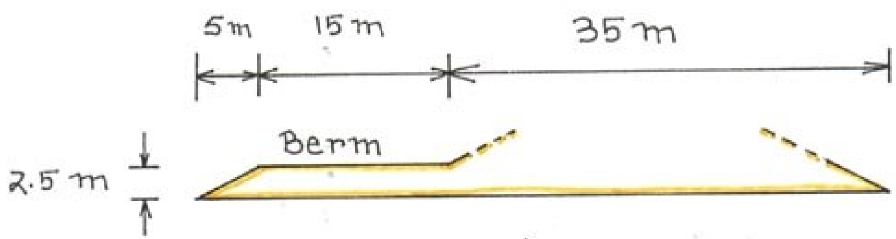
Pom Prachul



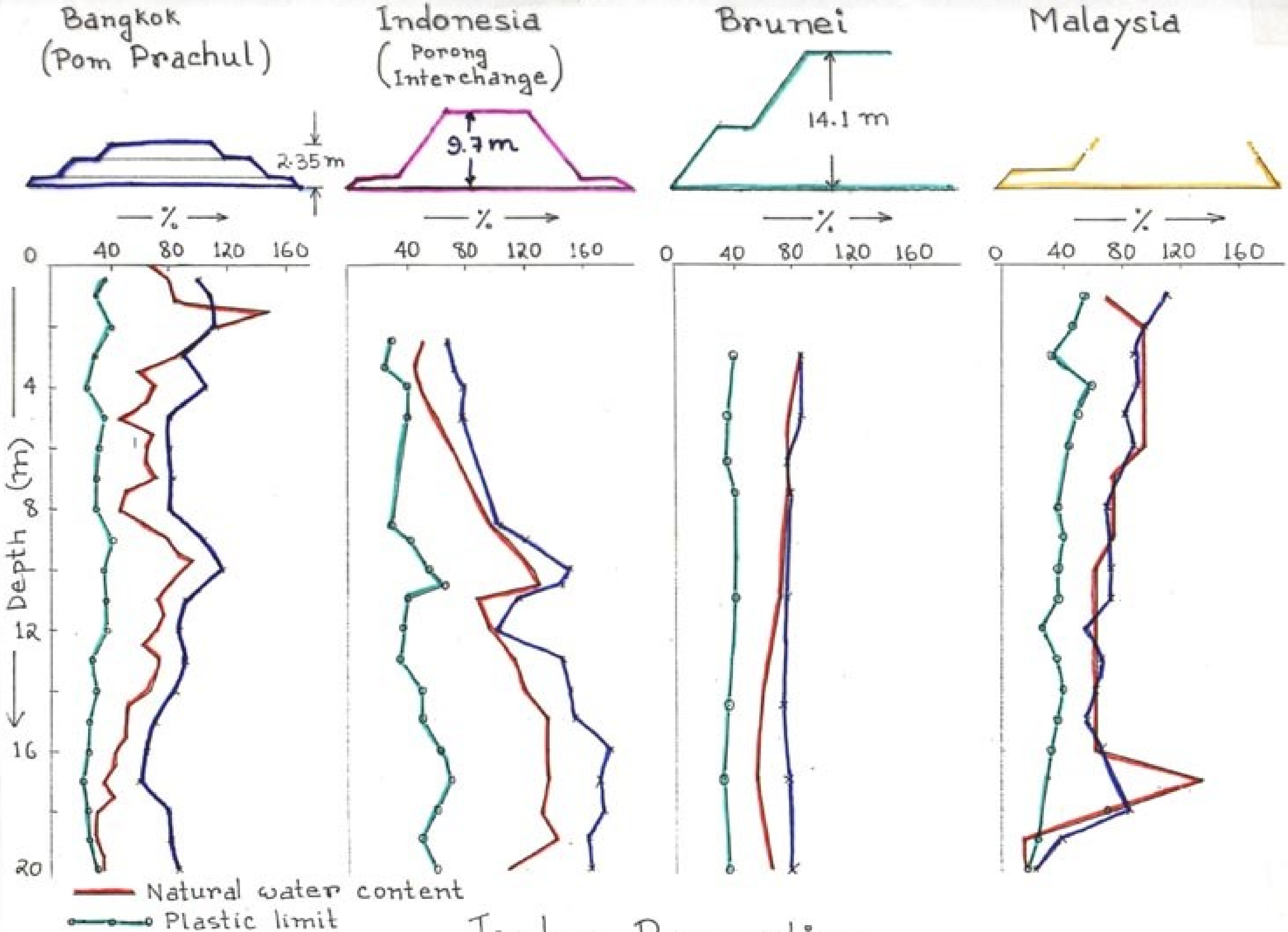
Indonesia (Railway Crossing)



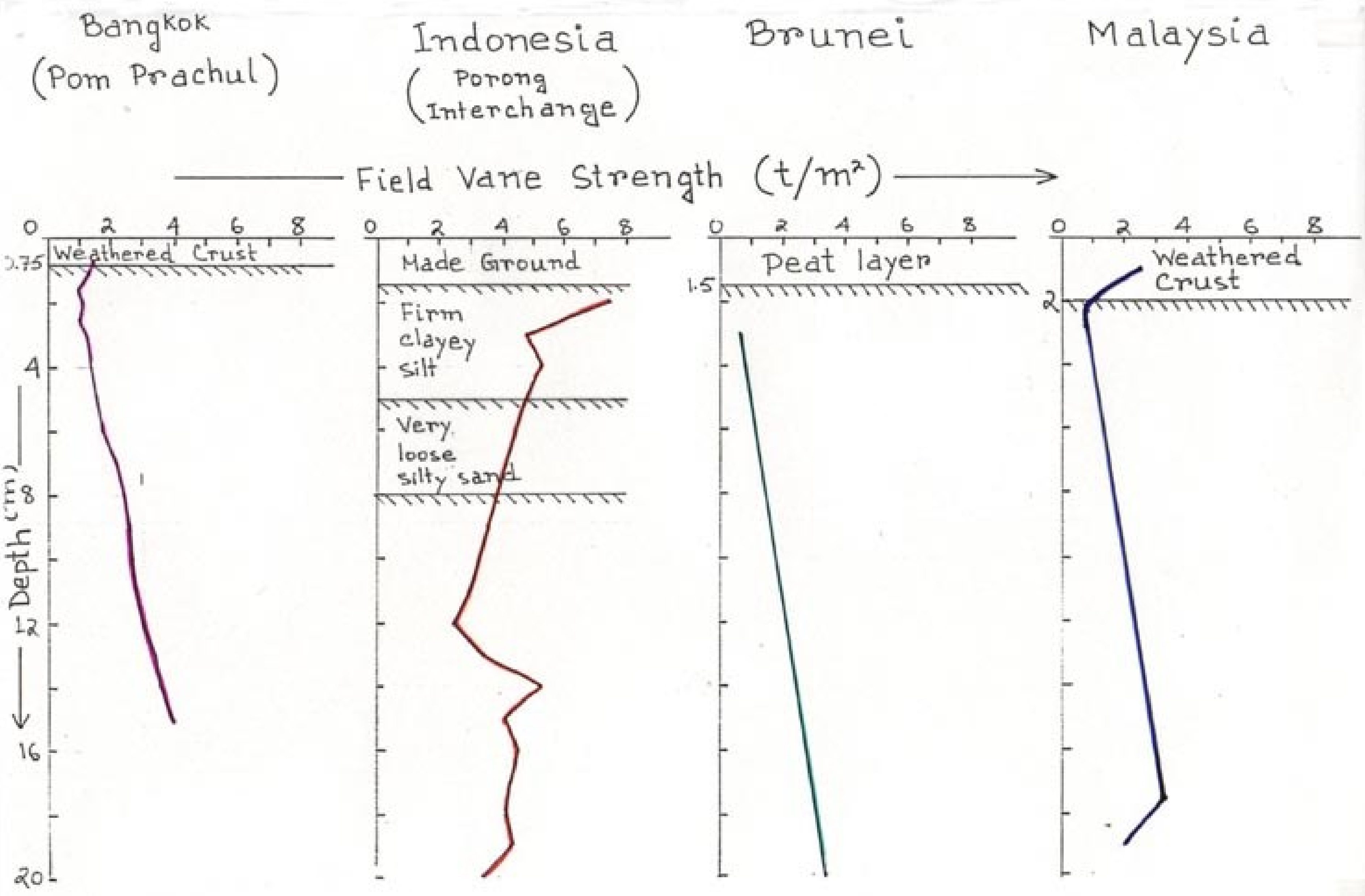
Indonesia (Porong Interchange)



Malaysia



Index Properties


Bangkok
(Pom Prachul)
Indonesia
(Porong Interchange)
Brunei
Malaysia
Field Vane Strength (t/m²)
0 2 4 6 8
0.75
Weathered Crust
Made Ground
Firm clayey silt
Very loose silty sand
1.5
Peat layer
2
Weathered Crust
Depth (m)
4
8
12
16
20

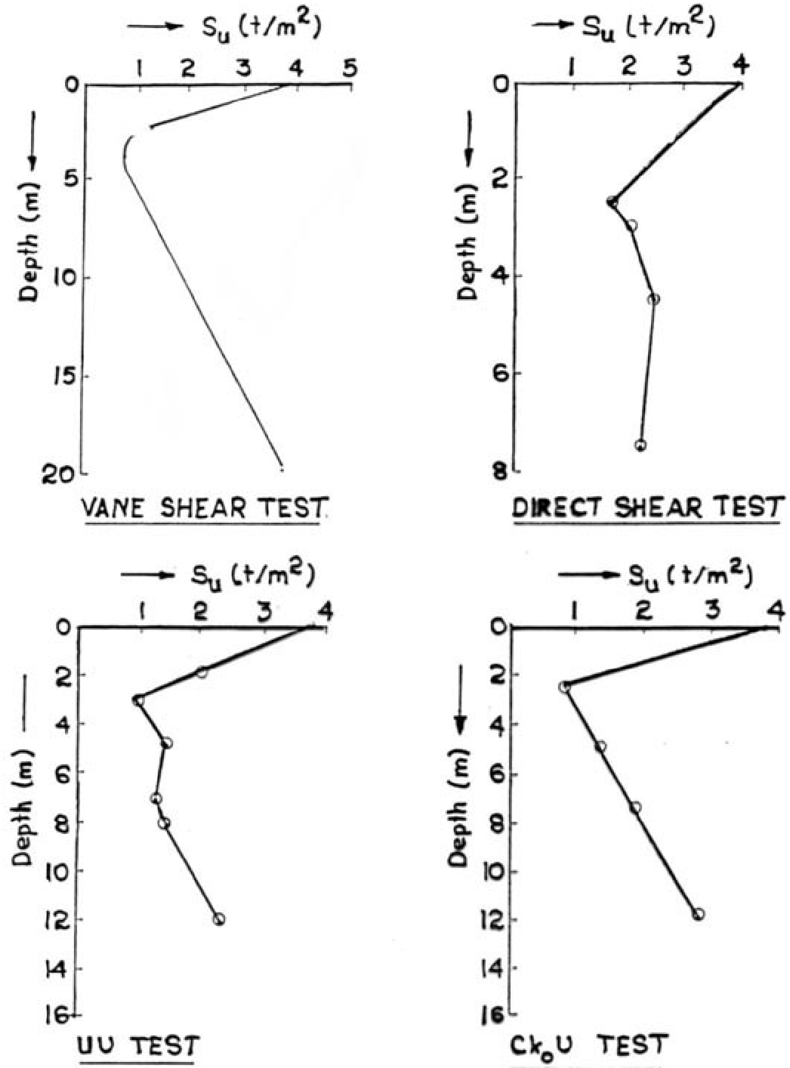


FIG. A.51 : Strength Profiles from different types of test

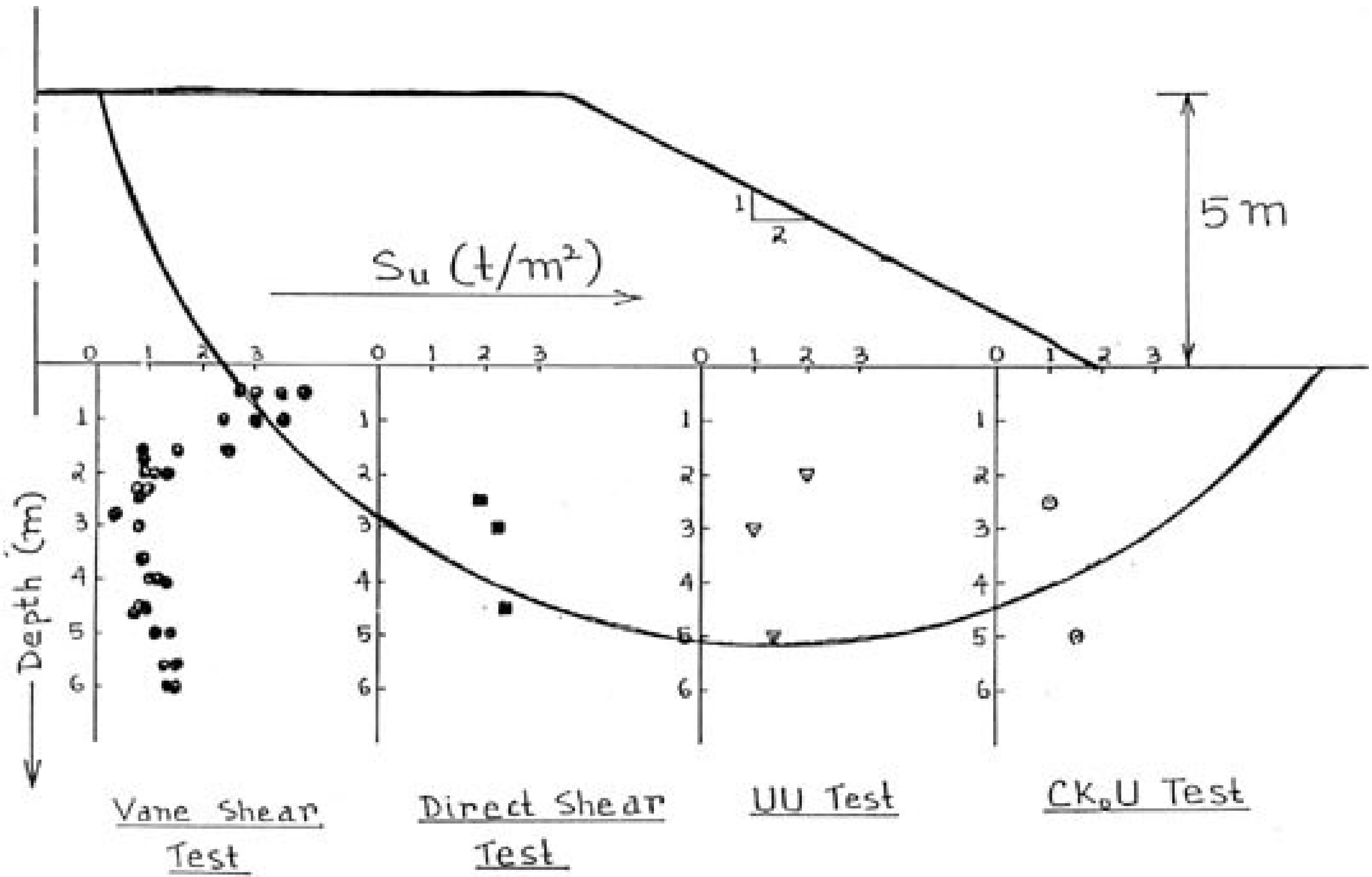


FIG. A.53 : Slip Surface and Strangth Profiles

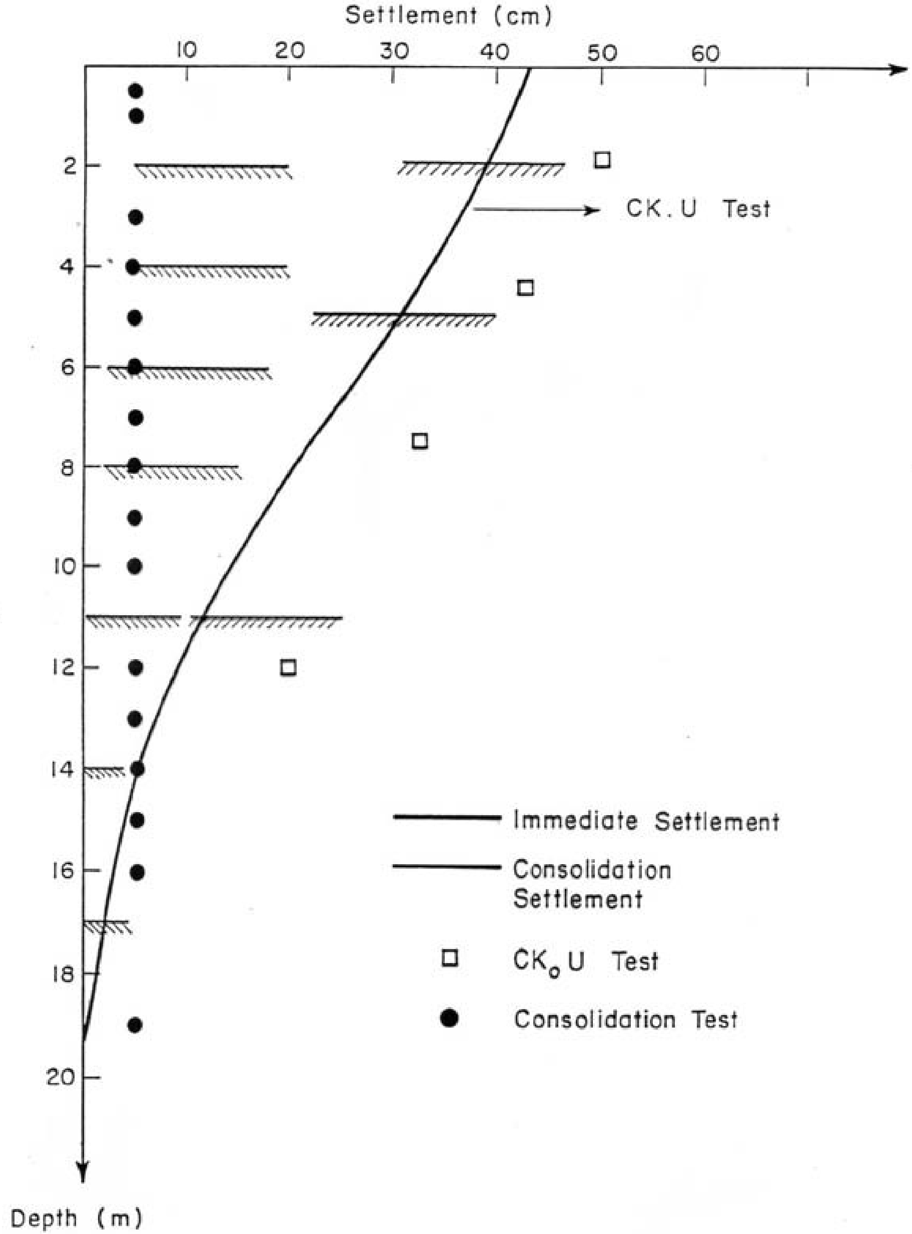
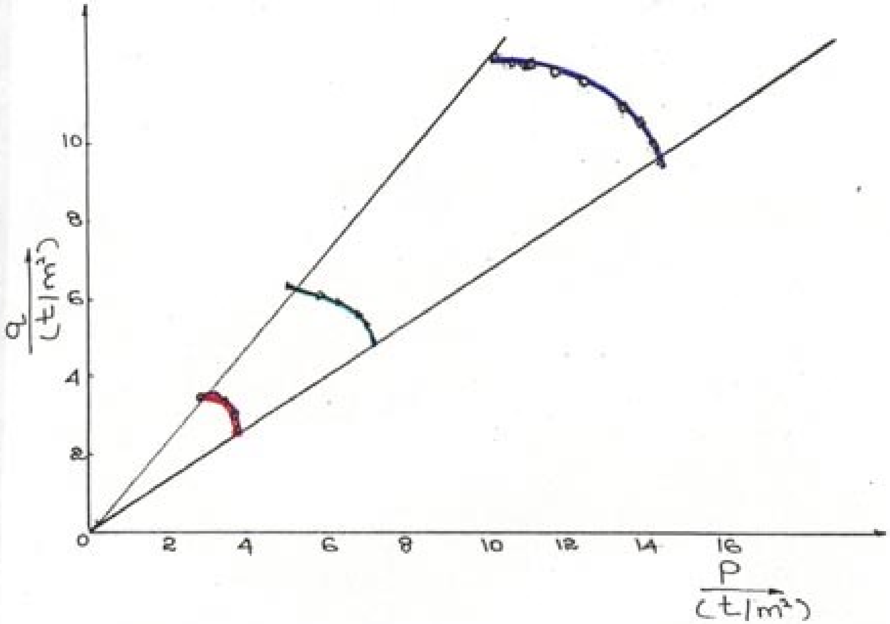
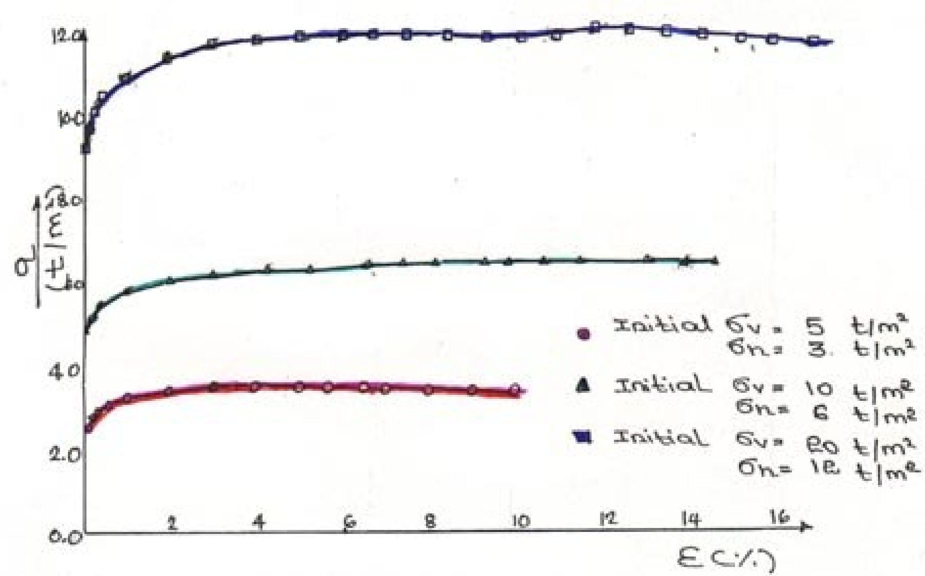


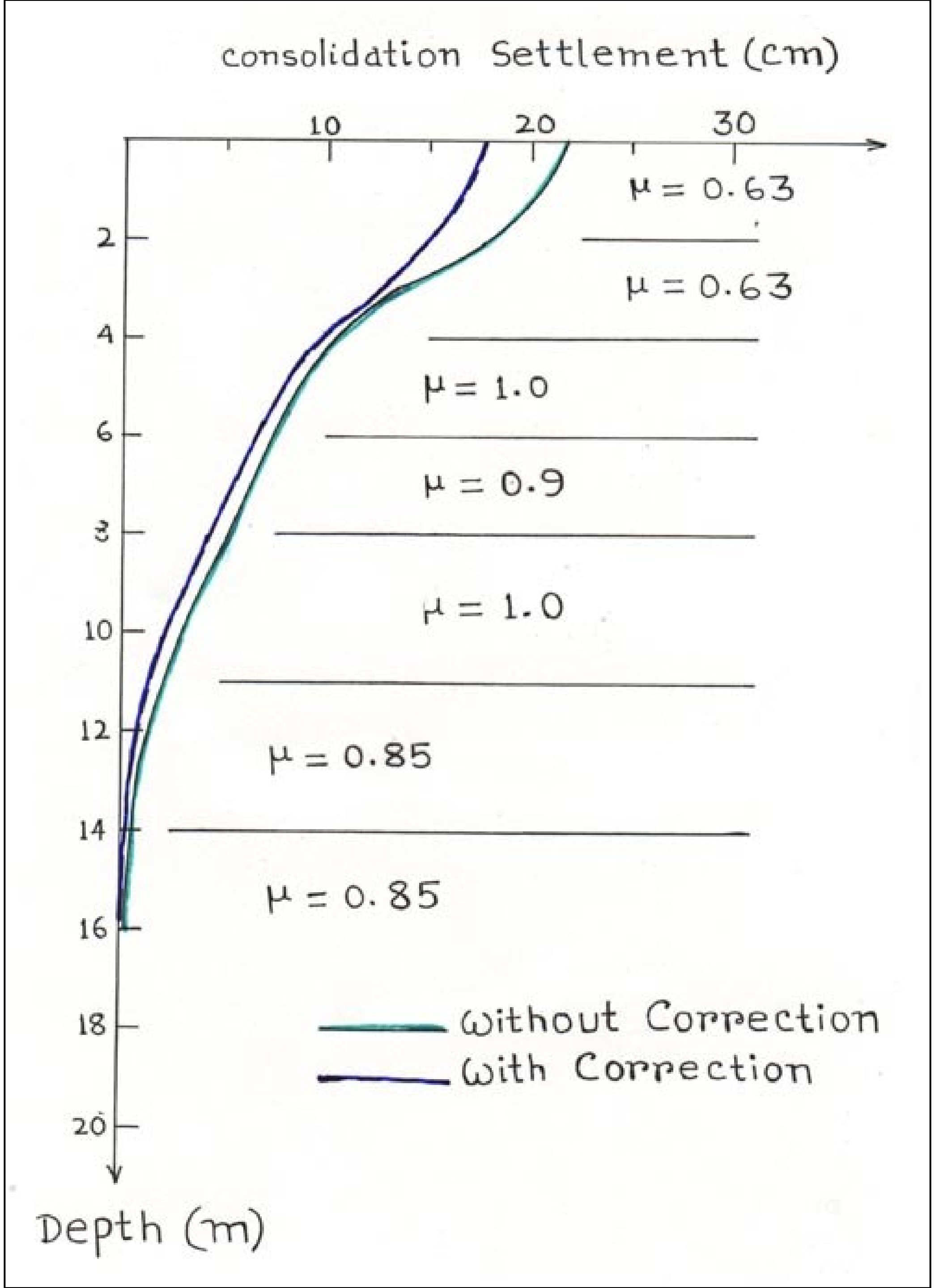
FIG. A.52 : Consolidation Test Locations

Sample Depth 8_3 m

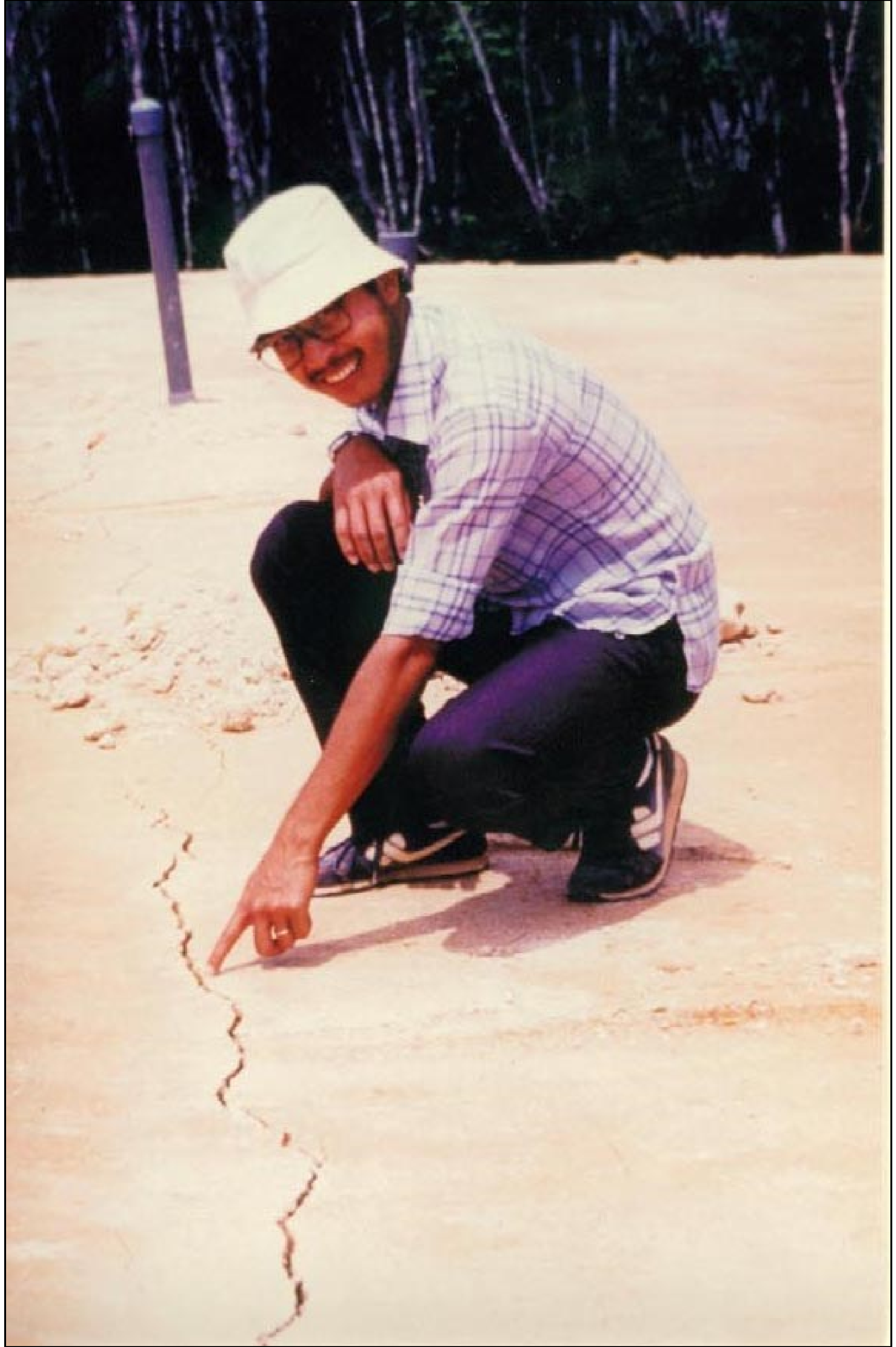


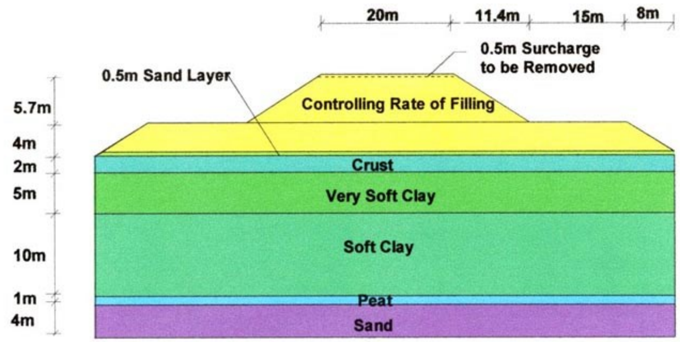


CK_oU TEST RESULTS



consolidation Settlement (cm)
10
20
30
μ = 0.63
μ = 0.63
μ = 1.0
μ = 0.9
μ = 1.0
μ = 0.85
μ = 0.85
2
4
6
8
10
12
14
16
18
20
without Correction
with Correction
Depth (m)





Typical cross section of Muar Clay Embankment

Soil Parameters Used for Reinforced Embankments

Depth (m)	κ	λ	*M*
0-2	0.06	0.35	1.20
2-7	0.10	0.61	1.07
7-12	0.06	0.28	1.07
12-18	0.04	0.22	1.07
18-22	0.03	0.10	1.20

Soil Parameters used for Embankments

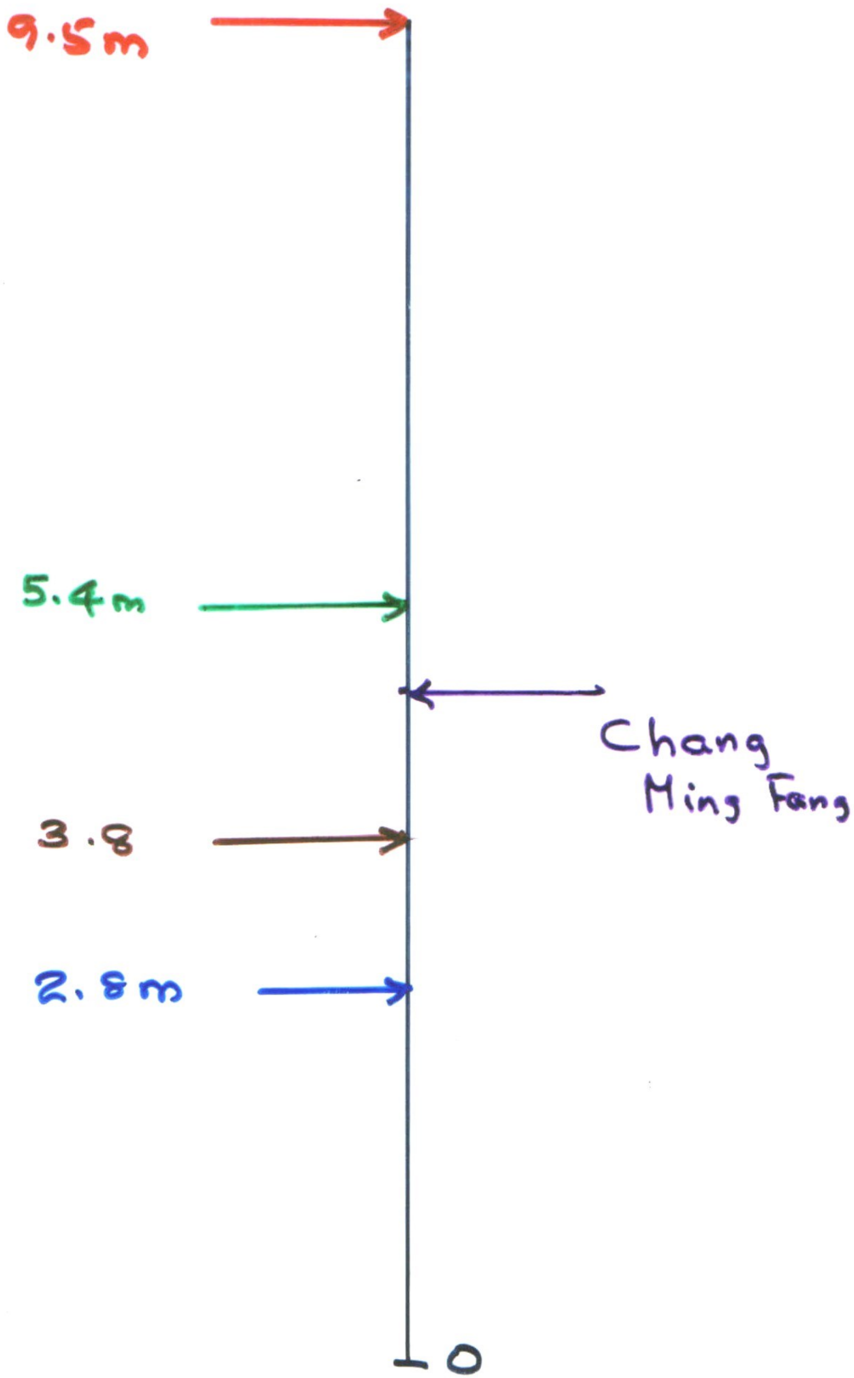
Depth (m)	κ	λ	*M*
0-2	0.06	0.16	1.19
2-6	0.06	0.16	1.19
6-8	0.05	0.15	1.12
8-18	0.04	0.09	1.07

Table 10 Summary of All 31 Predictions Made for the Failure Thickness and Failure Surface of the Muar Test Embankment (Figure 42)

Predictor		Method of Analysis	Strengths Adopted		Predictions for Failure, m		
					Failure Surface		Fill
No.	Name and Organization		Soft Clay	Compacted Fill	Depth	Radius	Thickness
1	J.S. Younger, J. Riyanto & C. Setjadiningrat, Bandung Institute of Technology, Indonesia	Fellenius	0.68 - 0.86 x vane (μ varied with PI)	s_u = 160 kPa (from cone data)	-	-	8.5
2	K.L. Siu, Special Projects Division, GCO, Hong Kong	Bearing capacity	s_u = 20 kPa	Zero	7.0*	14*	8.0*
3	R.A. Fraser, Maunsell Geotechnical Services Ltd, Hong Kong	Empirical chart (Kurihara, 1977)	Kurihara Fig. 6.5, with w = 80%	Zero	13.0*	~25*	6.5
4	'Trainees', Design Division, GCO, Hong Kong	Fellenius	1.2 x vane (consolidation)	c = 14 kPa, Ø = 31° (partially cracked)	8.0*	20*	6.4*
5	'Design Team', Design Division, GCO, Hong Kong	Bishop	1.2 x vane (consolidation)	c = 14 kPa, Ø = 31°	8.0*	18*	6.0*
6	C.O. Lo & Y.C. Lo, Port Works Division, Civil Engineering Department, Hong Kong	Bishop	0.8 x vane	c = 14 kPa, Ø = 31° (partially cracked)	5.2*	9*	6.0
7	'Engineer', Island East Division, GCO, Hong Kong	Stability chart (NHRI, 1982)	0.8 x vane	Zero	4.0	14	6.0
-	ACTUAL FAILURE →				~8.0	~26	5.4
8	J.M. Shen, Materials Division, GCO, Hong Kong	Fellenius	0.8 x vane	c = 50 kPa, Ø = 0	8.0	19	5.3
9	A.S. BALASUBRAMANIAM et al, Asian Institute of Technology, Bangkok, Thailand	Fellenius	0.9 x vane	c = 19 kPa, Ø = 26°	5.0	13	5.0
10	T.S.K. Lam, Special Projects Division, GCO, Hong Kong	Bishop	1.2 x vane (consolidation)	c = 14 kPa, Ø = 31°	6.0*	16*	5.0*
11	P. To, Special Projects Division, GCO, Hong Kong	Stability chart (Low, 1989)	s_u = 12 kPa (consolidation)	c = 14 kPa, Ø = 31°	8.0	19	5.0
12	'E & N Sections', Island East Division, GCO, Hong Kong	Stability chart (Taylor, 1948)	s_u = 15 kPa	Zero	-	-	5.0
13	S.H. Mak, Island East Division, GCO, Hong Kong	Stability chart (Low, 1989)	0.8 x vane	c = 14 kPa, Ø = 37° (Ø from cone data)	5.0	14	5.0
14	M.F. Chang, Nanyang Technological Institute, Singapore	Bearing capacity	s_u = 9 kPa (20 kPa in crust)	c = 0, Ø = 40°	2.5	~9.5	4.7
15	C.P. Wroth, Oxford University, UK	'Judgement' based on Invited Predictors	'Corrected vane'	'Some contribution'	-	-	4.6
16	G. Wong, Greg Wong & Associates, Hong Kong	'Experience' checked by bearing capacity	Average vane, with N_c = 6	Zero (cracked)	-	-	4.4
17	P.K. Chen, Ove Arup & Partners, Hong Kong	Janbu	0.8 x vane	c = 14 kPa, Ø = 31°	5.0*	10*	4.0*
18	S. Buttling, Scott Wilson Kirkpatrick & Partners, Hong Kong	Janbu	0.8 x vane	Zero (cracked)	4.5	14	4.0
19	H. Aboshi, Fukken Co. Ltd, Hiroshima, Japan	Fellenius	0.85 x vane (+2.5 kPa consolidation)	Zero	4.5*	10*	3.9*
20	J.P. MAGNAN, Laboratoire Central des Ponts et Chaussées, Paris, France	Bishop	0.85 x vane	c = 20 kPa, Ø = 0	7.2-11.0	13-21	3.8
21	H.G. POULOS, C.Y. Lee & J.C. Small, Sydney University, Australia	Fellenius	Vane, lower bound	Zero (cracked)	5.9	13	3.8
22	M.H. Goldsworthy, Howard Humphries & Partners, Johore, Malaysia	Jakobson (1948)	s_u = 13.4 kPa (average vane to 8 m)	Zero (cracked)	8.0	13	3.8
23	H.S. Tan, H. Ernst & C.C. Ladd, Massachusetts Institute of Technology, USA	Bishop	Vane/SHANSEP (Ladd & Foott, 1974)	c = 0, Ø = 31°	5.8	12	3.8
24	B.N. Leung, S.H. Tse & Y.C. Chan, Mainland West Division, GCO, Hong Kong	Bishop	0.85 x vane	c = 14 kPa, Ø = 31° (partially cracked)	6.0	11	3.7
25	'S Section', Island East Division, GCO, Hong Kong	Bishop	0.8 x vane	Zero (cracked)	8.0	13	3.6
26	A. NAKASE & J. Takemura, Tokyo Institute of Technology, Japan	Fellenius	$s_u = 0.27\sigma'_v$	c = 0, Ø = 30°	4.6	9.5	3.5
27	D.T. Bergado, Asian Institute of Technology, Bangkok, Thailand	Bishop	0.8 x vane	c = 19 kPa, Ø = 26°	8.0*	17*	3.5*
28	G. Aas & O. Eide, Norwegian Geotechnical Institute, Oslo, Norway	Bishop (effective in crust)	~0.83 - 0.7 x vane (μ varied with PI)	Zero (cracked)	5.0	12	2.9
29	'Student', Development & Airport Division, Civil Engineering Department, Hong Kong	Fellenius	Vane	c = 14 kPa, Ø = 0	5.0*	11.5*	2.8*
3	T.W. Lo, Nanyang Technological Institute, Singapore	Fellenius	0.75 x vane	Zero	3.5*	16*	2.8*
31	K.S. Wong, Nanyang Technological Institute, Singapore	Finite elements	0.8 x vane, E_u = 200 s_u	c = 14 kPa, Ø = 31°	4.0*	16*	2.8*

Notes : (1) All stability analyses were carried out in terms of total stress unless otherwise stated.

(2) * Signifies that the predicted failure surface is not the critical one for the assumptions adopted, usually because the mid-point circle principl 's violated.



9.5m
5.4m
Chang
Ming Fang
3.8
2.8m
0

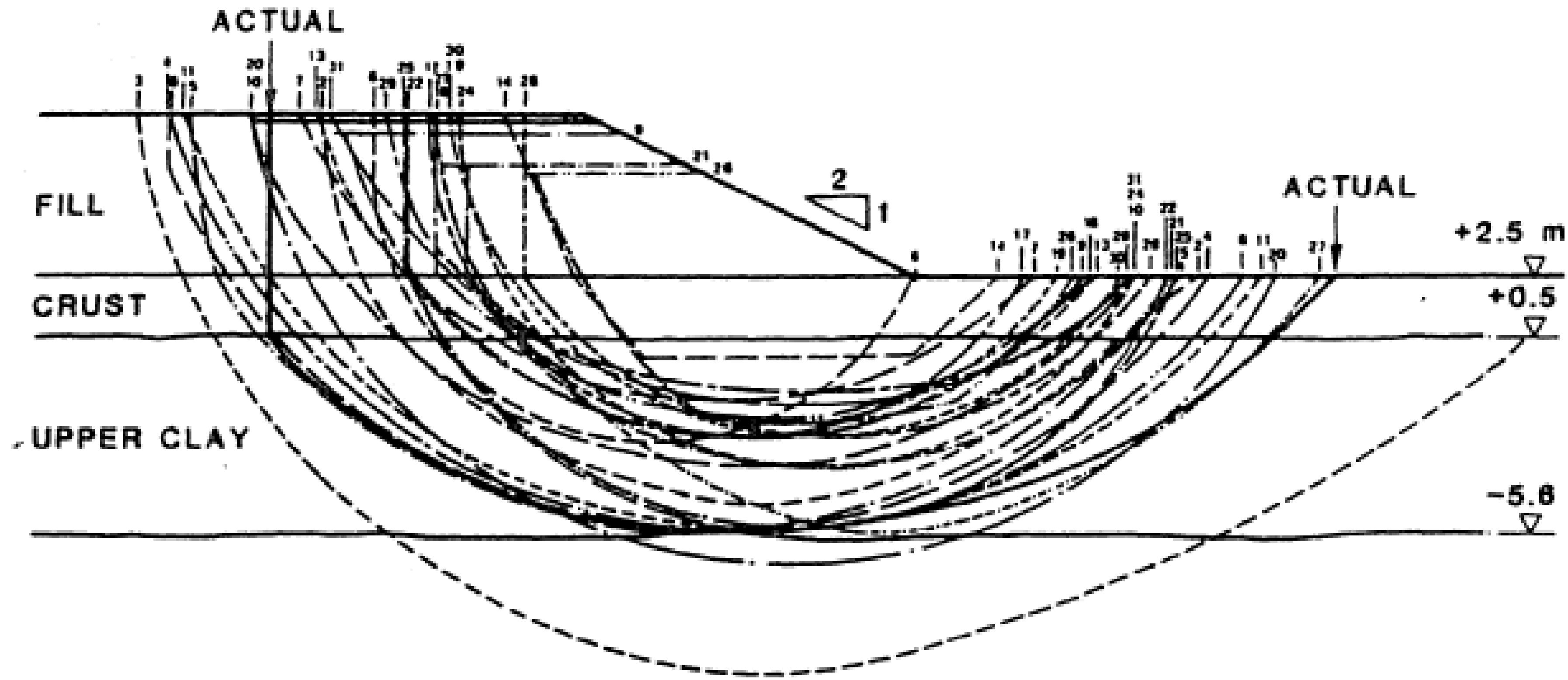


Fig. 6 The failure surfaces predicted by all 31 predictors (see Table 1 for surface numbers)

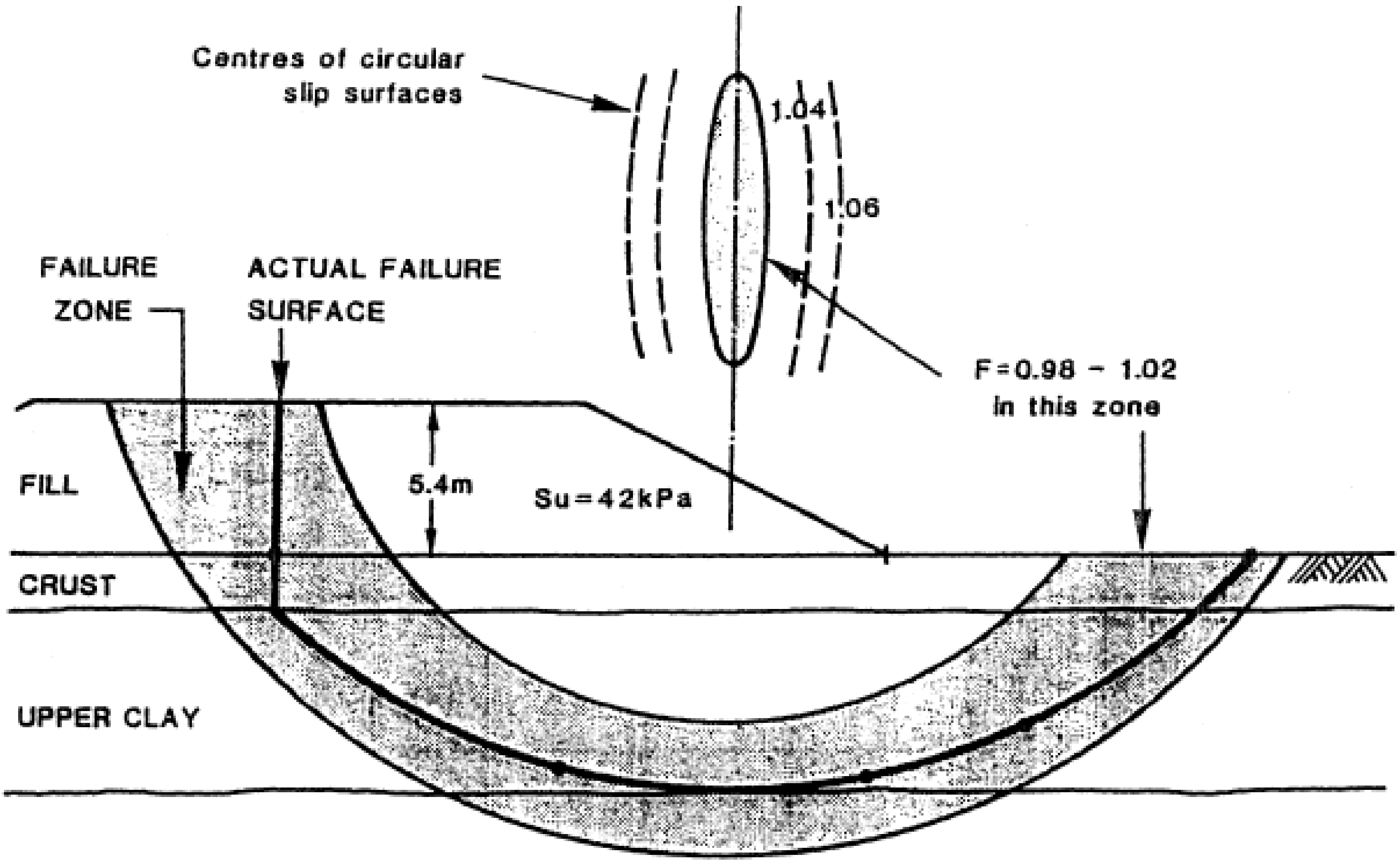


Fig. 10 Reanalysis of the embankment failure, showing the possible failure zone

Table 2 Actual failure conditions for the test embankment compared with predictions of 'major' predictors (all values in metres)

Predictor	Fill Thickness	Fill Height	Slip Surface Depth	Maximum Embankment Settlement	Maximum Surface Heave	Maximum Lateral Movement	Excess Pore Pressure in Piezometer P2
Bala	5.0	4.35	5.0	0.65	0.18	0.35	1.0
Nakase	3.5	3.20	4.6	0.30	0.25	0.50	6.2
Poulos	3.8	3.45	5.9	0.35	0.05	0.16	5.7
Magnan	3.8	3.25	7.2–11.0	0.55	0.09	0.12	7.0 ?
Average	4.0	3.55	6.1	0.50	0.14	0.28	5.0
ACTUAL	5.4	4.70	8.2	0.70	0.15	0.37	9.3

Effective stress analysis:

Limit equilibrium analysis

The method of slices

In this method the potential failure surface, in section, is again assumed to be a circular arc with center O and radius r. The soil mass (ABCD) above a trial failure surface (AC) is divided by vertical planes into a series of slices of width b, as shown in Fig. 6.5.

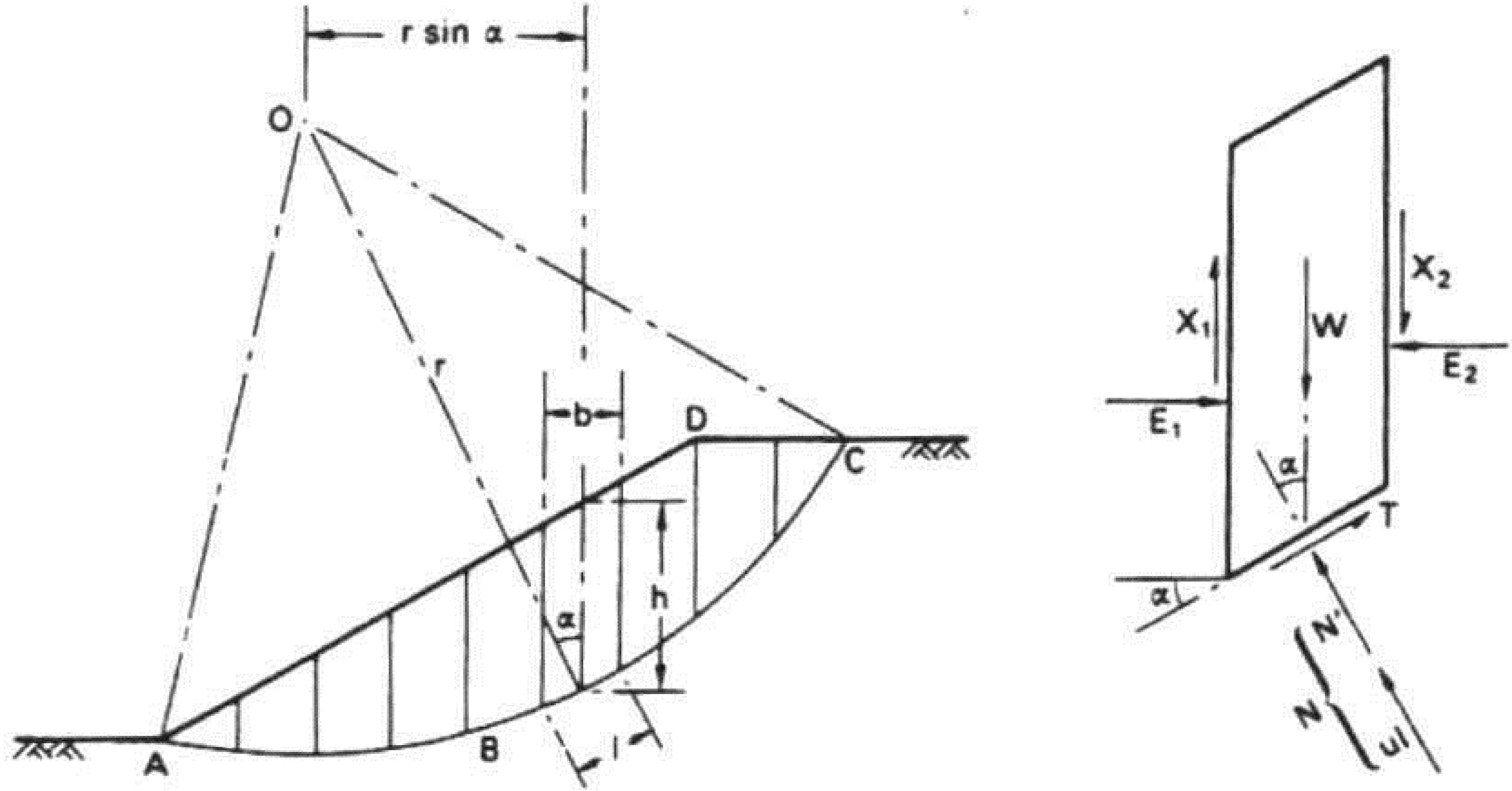


Fig. 6.5 The method of slices

The base of each slice is assumed to be a straight line. For any slice the inclination of the base to the horizontal is α and the height measured on the center-line, is h. The analysis is based on the use of a lumped factor of safety (F), defined as the ratio of the available shear strength (τ_f) to the shear strength (τ_m), which must be mobilized to maintain a condition of limiting equilibrium.

That is,

$$F = \frac{\tau_f}{\tau_m}$$

The factor of safety is taken to be the same for each slice, implying that there must be mutual support between slices that is forces must act between the slices.

The forces per unit dimension normal to the section) acting on a slice are:

The total unit weight of the slice, W = γbh (γ_{sat}). The total normal force on the base, N (equal to σl). In general this force has two components, the effective normal force N' (equal to $\sigma' l$) and the boundary water force U (equal to, ul), where u is the pore water pressure at the center of the base and l is the length of the base.

The shear force on the base, T= $\tau_m l$

The total normal force on the sides, E_1 and E_2.

The shear forces on the sides X_1 and X_2.

Any external forces must also be included in the analysis.

The problem is statically indeterminate and in order to obtain a solution assumptions must be made regarding the inter-slice forces E and X: in general the resulting solution for factor of safety is not exact.

Considering moments about O, the sum of the moments of the shear forces T on the failure arc AC must equal the moment of the weight of the soil mass ABCD.
For any slice the lever arm of W is r sin α, and therefore

$$\Sigma Tr = \Sigma Wr \sin \alpha$$

$$T = \tau_m l = \frac{\tau}{F} l$$

$$\Sigma \frac{\tau_f}{F} l = \Sigma W \sin \alpha$$

$$F = \frac{\sum \tau_f l}{\sum W \sin \alpha}$$

For an analysis in terms of effective stresses

$$F = \frac{\Sigma(c' + \sigma' \tan\phi')l}{\Sigma W \sin\alpha} \qquad F = \frac{c'L + \tan\phi' \Sigma N'}{\Sigma W \sin\alpha}$$

L_a is the arc length AC

Equation 6.3 is exact but approximations are introduced in determining the forces N'.

The Fellenius (or Swedish) solution

In this solution for each slice it is assumed that the resultant of the inter-slice forces is zero. The solution involves resolving the forces on each slice normal to the base, that is

$$N' = W \cos \alpha - ul$$

Hence the factor of safety in terms of effective stress (Equation 6.3) is given by

$$F = \frac{c'L + \tan\phi' \Sigma(W\cos\alpha - ul)}{\Sigma W \sin\alpha}$$

The components Wcosα and Wsinα can be determined graphically for each slice. Alternatively, the value of α cab be measured or calculated. Again, a series of trial failure surfaces must be chosen in order to obtain the minimum factor of safety. This solution underestimates the factor of safety: the error, compared with more accurate methods of analysis is usually within 5 to 20 percent.

For an analysis in terms of total stress the parameters c_u and ϕ_u are used and the value of u in equation 6.4 is zero. If $\phi_u = 0$, the factor of safety is given by

$$F = \frac{c_u L}{\Sigma W \sin\alpha}$$

An exact value of F is obtained

The Bishop Routine Solution

In this solution it is assumed that the resultant forces on the sides of the slices are horizontal, that is

$$X_1 - X_2 = 0$$

For equilibrium the shear force on the base of any slice is

$$T = \frac{1}{F}(c'l + N'\tan\phi')$$

Resolving the forces in the vertical direction:

$$W = N'\cos\alpha + ul\cos\alpha + \frac{c'l}{F}\sin\alpha + \frac{N'}{F}\tan\phi'\sin\alpha$$

$$N' = \frac{\left(W - \frac{c'l}{F}\sin\alpha - ul\cos\alpha\right)}{\left(\cos\alpha + \frac{\left(\tan\alpha\tan\phi'\right)}{F}\right)}$$

It is convenient to substitute

$$l = b\sec\alpha$$

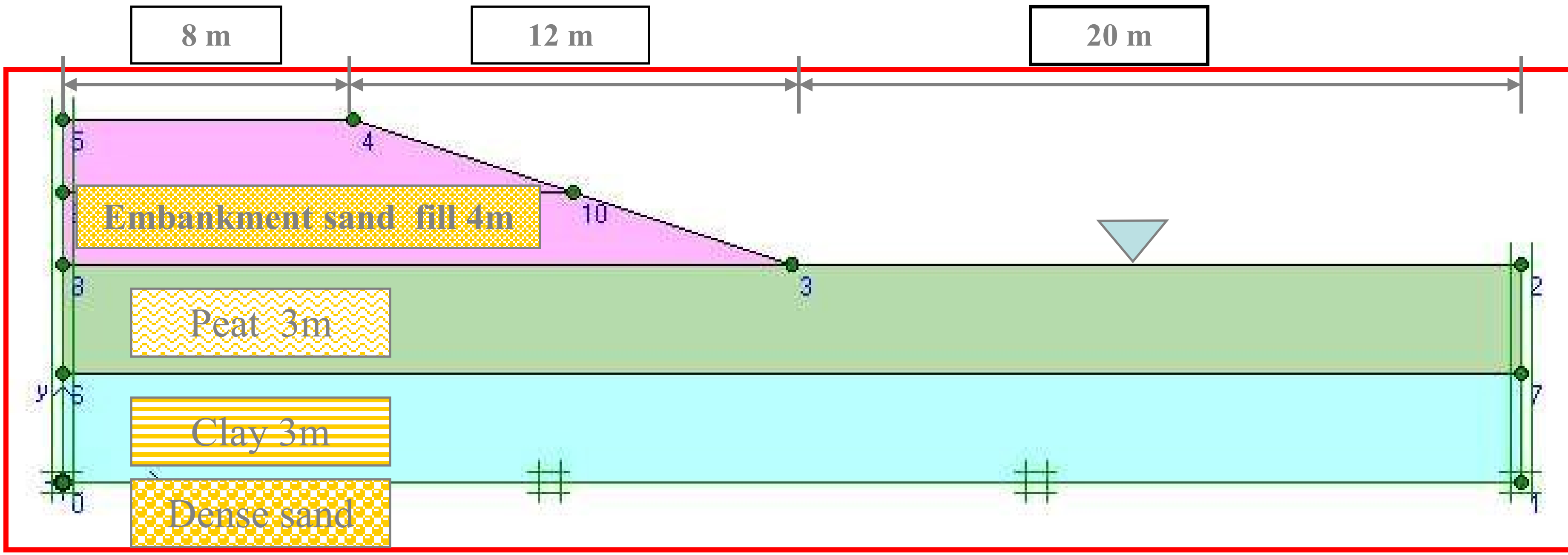
From equation 6.3 after some re-arrangements

$$F = \frac{1}{\Sigma W\sin\alpha}\Sigma\left[\left\{c'b + \left(W - ub\right)\tan\phi'\right\}\frac{\sec\alpha}{1 + \frac{\left(\tan\alpha\tan\phi'\right)}{F}}\right]$$

PLAXIS analysis:

Continuum approach

FINITE ELEMENT METHOD IN STABILITY ANALYSIS



- Backfill Increment = 2m x 2
- *C* - ϕ Reduction Stability Analysis :

(1) Immediate After First and Second Backfill
(2) After Long-Term Consolidation

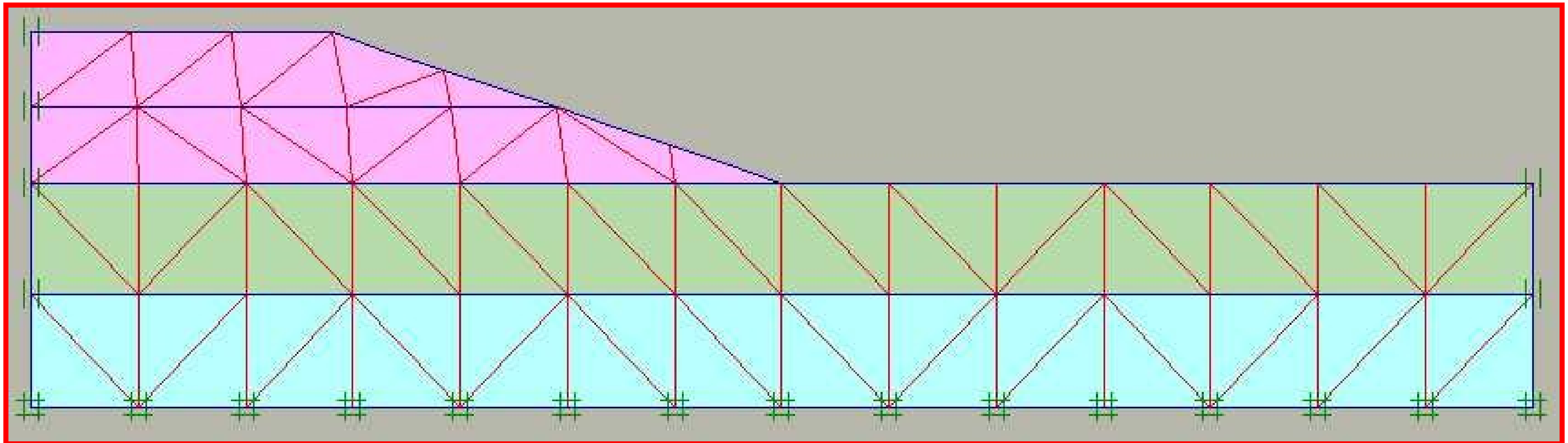
THE CONCEPT OF C ~ ϕ REDUCTIONS SLOPE STABILITY ANALYSIS

$$Safety\ Factor = SF = \frac{\tau_{\text{maximum available}}}{\tau_{\text{required for equilibrium}}} = \frac{c + \sigma_n \tan\phi}{c_r + \sigma_n \tan\phi_r}$$

$$Safety\ Factor = SF = \left(1 + \sum_{i=1}^{n} M_i^{SF}\right)$$

$$c_r = c\left(1 - \sum_{i=1}^{n} M_i^{SF}\right) \quad , \quad \tan\phi_r = \tan\phi\left(1 - \sum_{i=1}^{n} M_i^{SF}\right)$$

FINITE ELEMENT DICRETIZATION



- Material zones
- Element types
- Initial and boundary conditions
- Mesh generation (coarseness)

- Simulation :

(phase 1) 1st stage construction

(phase 5) $C \sim \phi$ reduction

(phase 2) 200 days consolidation

(phase 3) 2nd stage construction

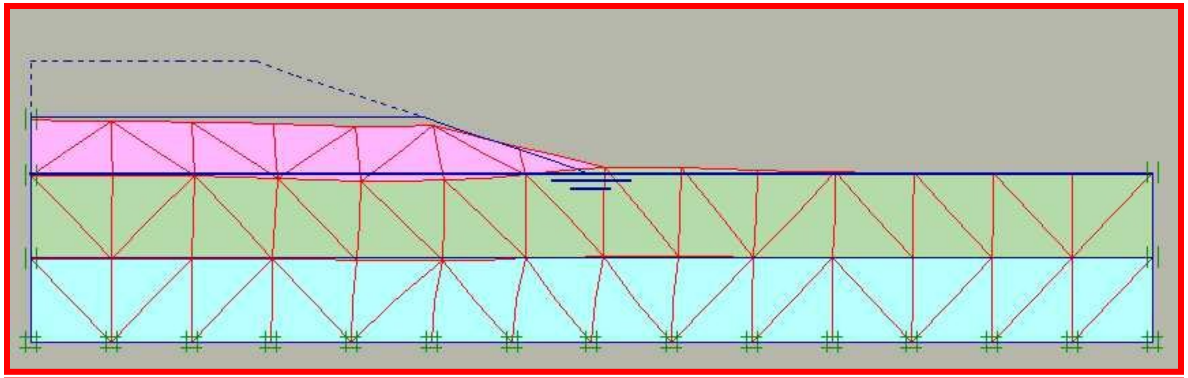
(phase 6) $C \sim \phi$ reduction

(phase 4) consolidation to Δu=1kpa

(phase 7) $C \sim \phi$ reduction

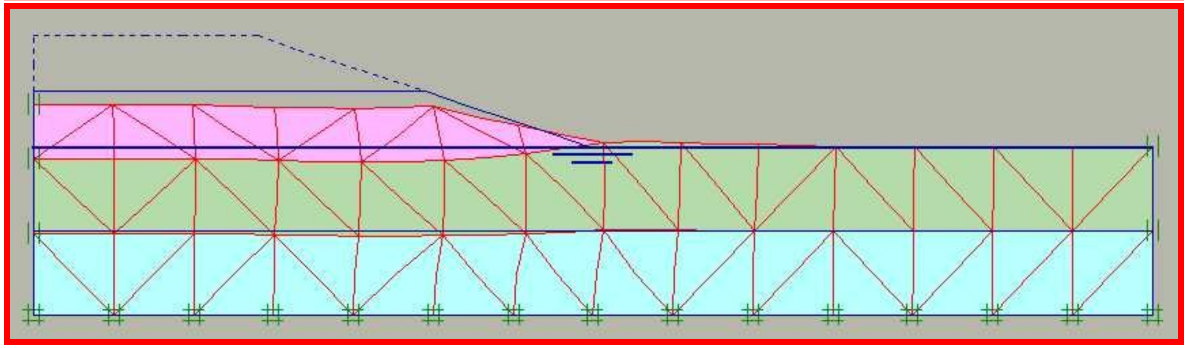
MATERIAL PROPERTIES OF THE ROAD EMBANKMENT AND SUBSOIL

Parameter	Name	Clay	Peat	Sand (Embankment)	Unit
Material model	Model	MC	MC	MC	-
Type of behavior	Type	undrained	unrained	drained	-
Dry soil weight	γ_{dry}	15	8	16	kN/m^3
Wet soil weight	γ_{wet}	28	11	20	kN/m^3
Horizontal permeability	κ_x	$1x10^{-4}$	$2x10^{-3}$	1.0	m/day
Vertical permeability	κ_y	$1x10^{-4}$	$1x10^{-3}$	1.0	m/day
Young's modulus	E_{ref}	1000	350	3000	kN/m^2
Poisson's ratio	ν	0.33	0.35	0.3	-
Cohesion	C_{ref}	2.0	5.0	1.0	kN/m^2
Friction angle	φ	24	20	30	o
Dilatancy angle	Ψ	0.0	0.0	0.0	o



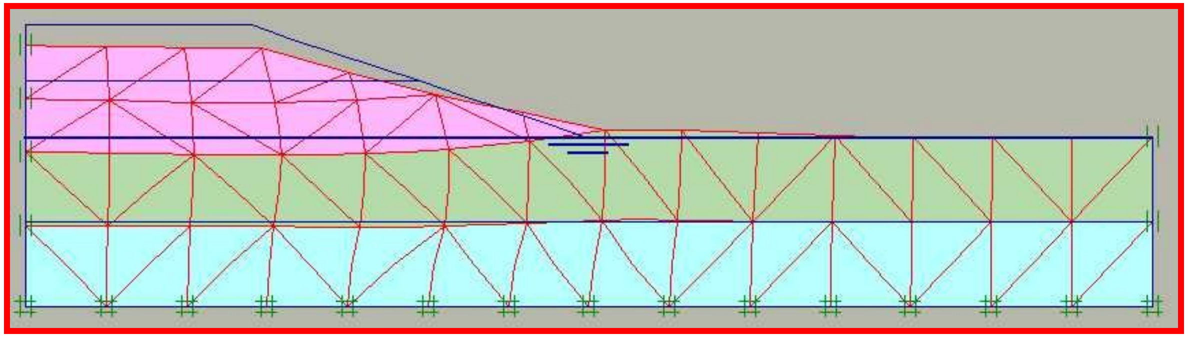
Phase 1-- 1st stage construction

maximum settlement δ_v=1.61cm



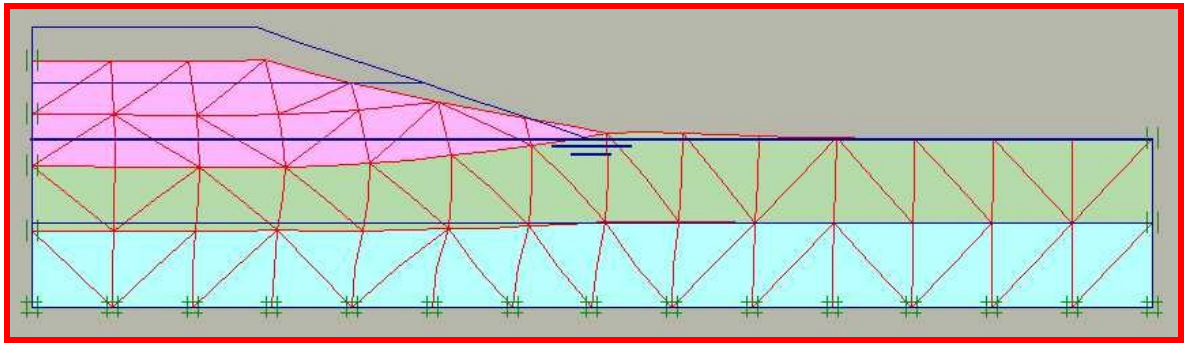
Phase 2 -- 200 days consolidation

maximum settlement δ_v=2.93cm



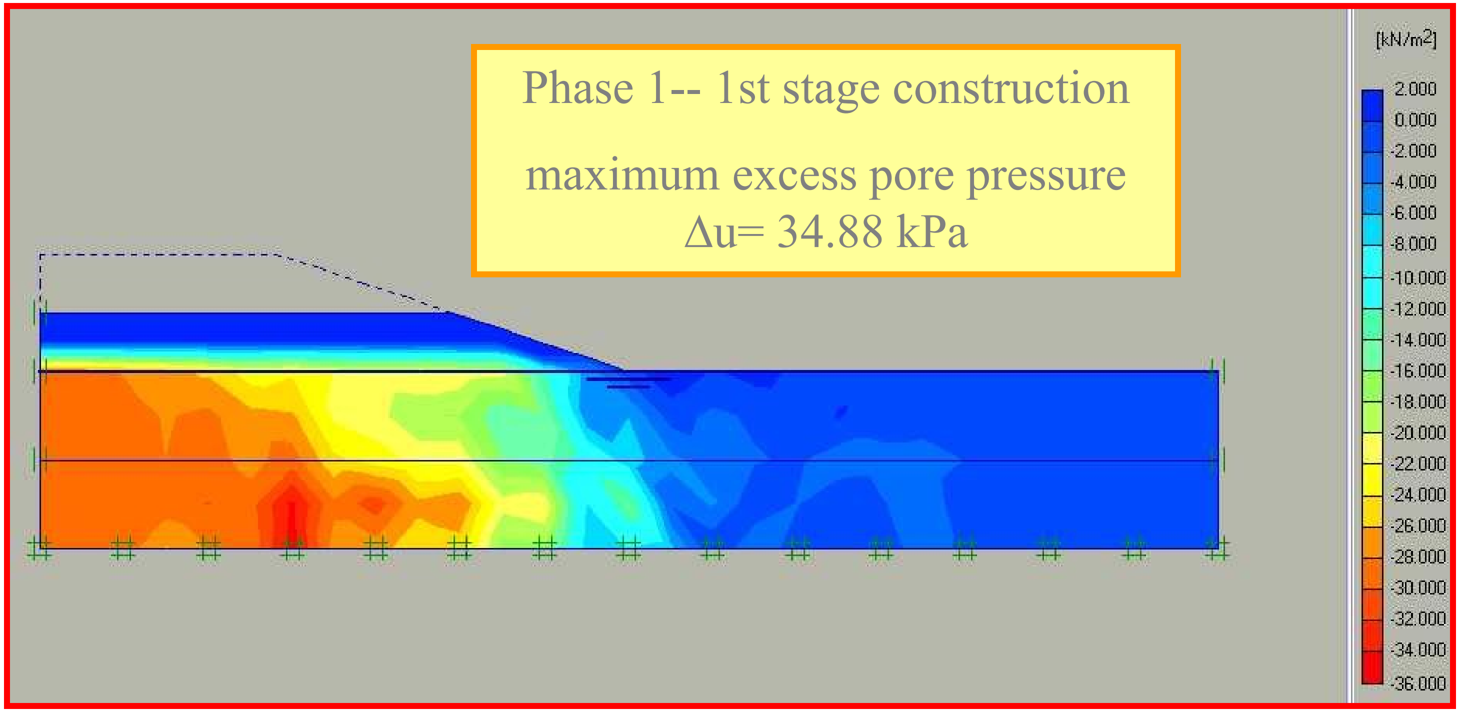
Phase 3-- 2nd stage construction

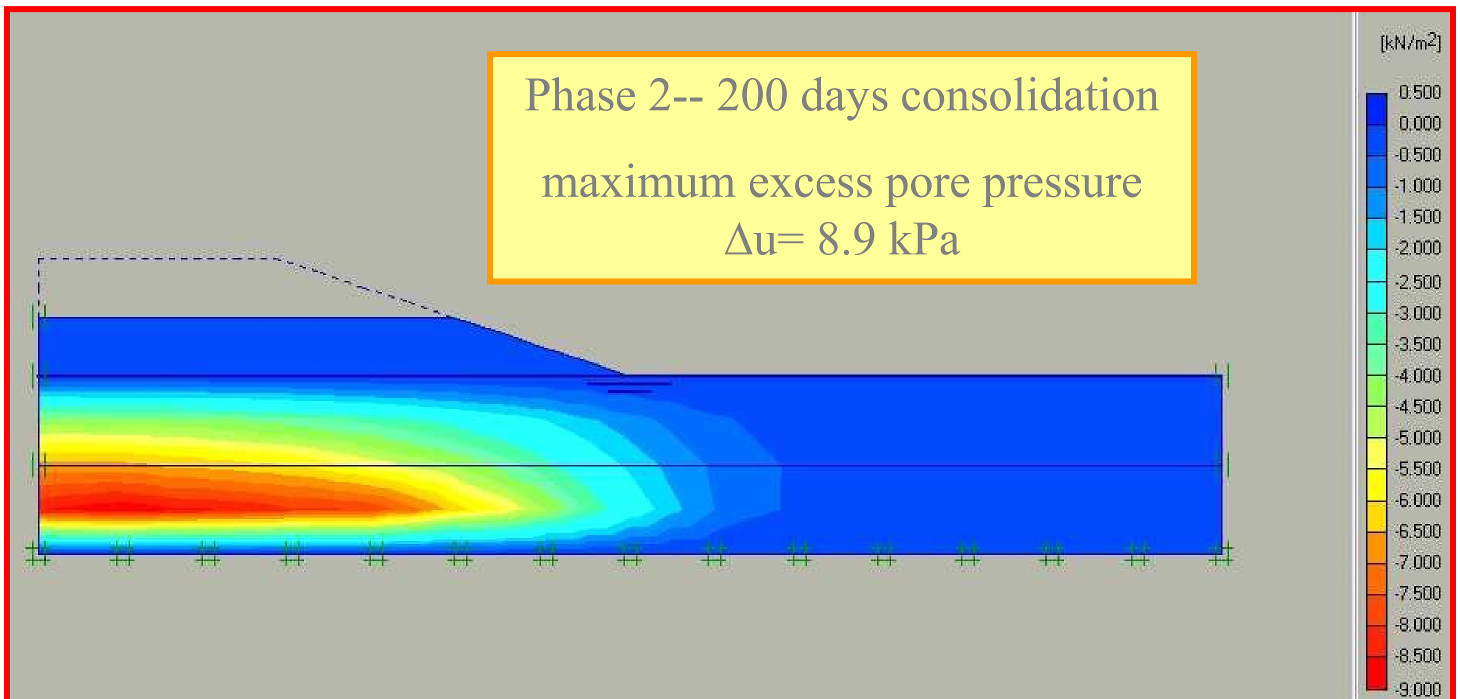
maximum settlement δv=4.14cm



Phase 4 -- consolidation to Δu=1kpa

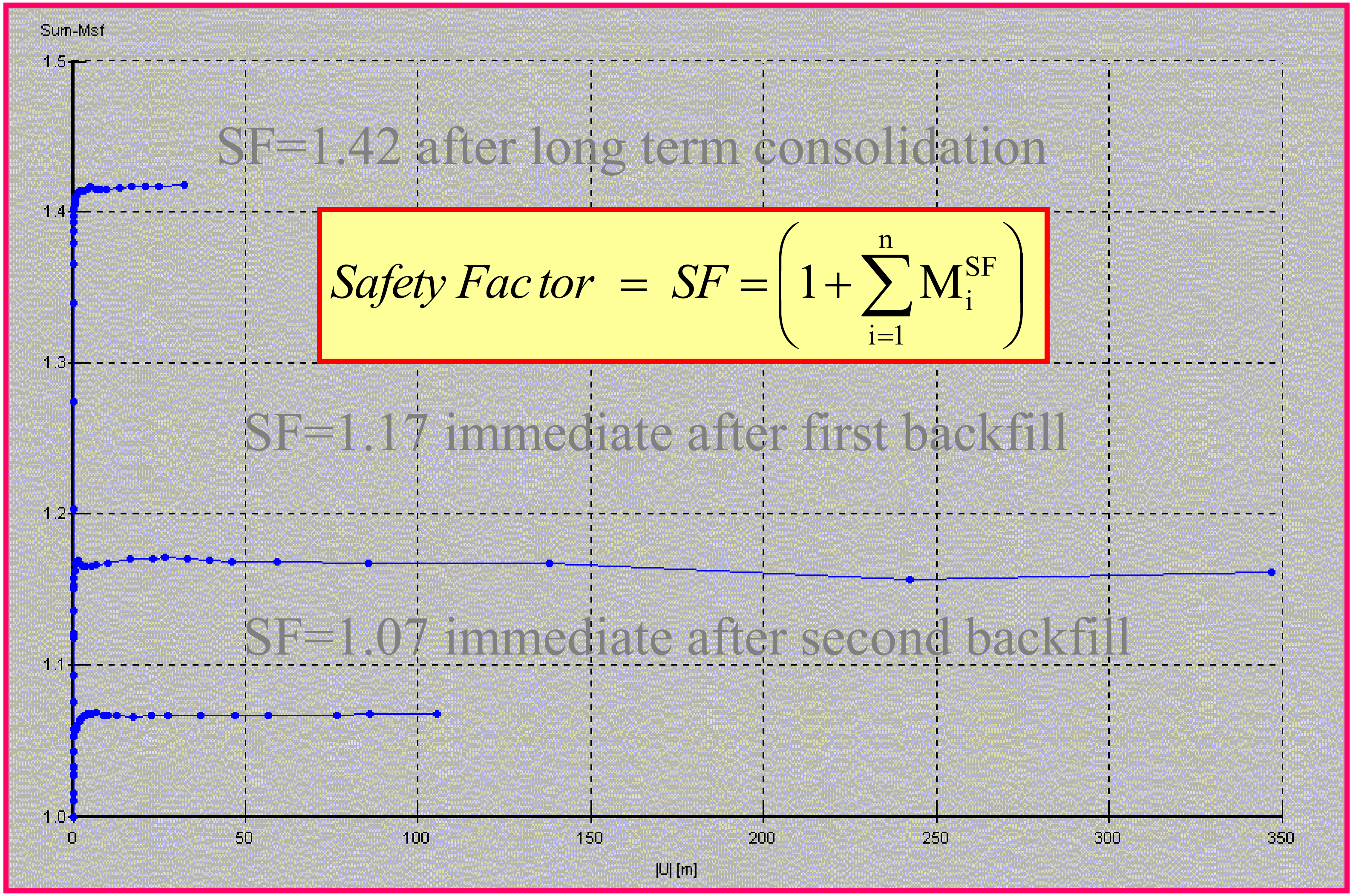
maximum settlement δv=6.10cm


Phase 1-- 1st stage construction
maximum excess pore pressure
Δu= 34.88 kPa
[kN/m2]
2.000
0.000
-2.000
-4.000
-6.000
-8.000
-10.000
-12.000
-14.000
-16.000
-18.000
-20.000
-22.000
-24.000
-26.000
-28.000
-30.000
-32.000
-34.000
-36.000


Phase 2-- 200 days consolidation
maximum excess pore pressure
Δu= 8.9 kPa
[kN/m2]
0.500
0.000
-0.500
-1.000
-1.500
-2.000
-2.500
-3.000
-3.500
-4.000
-4.500
-5.000
-5.500
-6.000
-6.500
-7.000
-7.500
-8.000
-8.500
-9.000

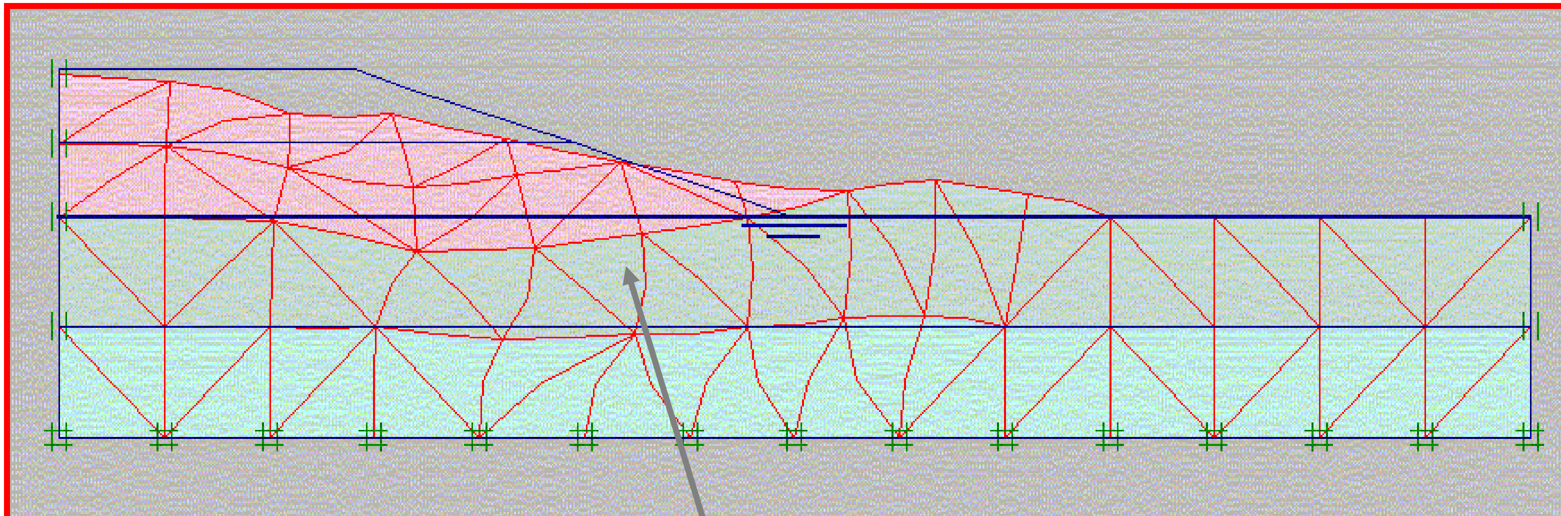
SAFETY FACTOR OF EMBANKMENT AND DISPLACEMENT AT TOE

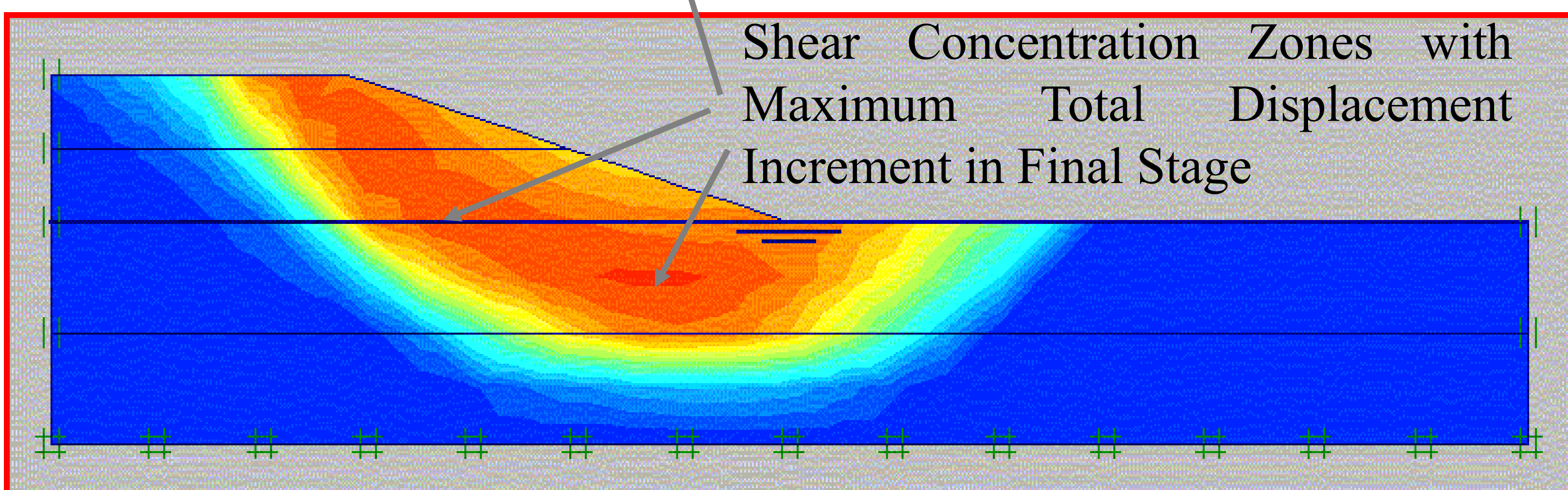
Global Safety Factor *SF*

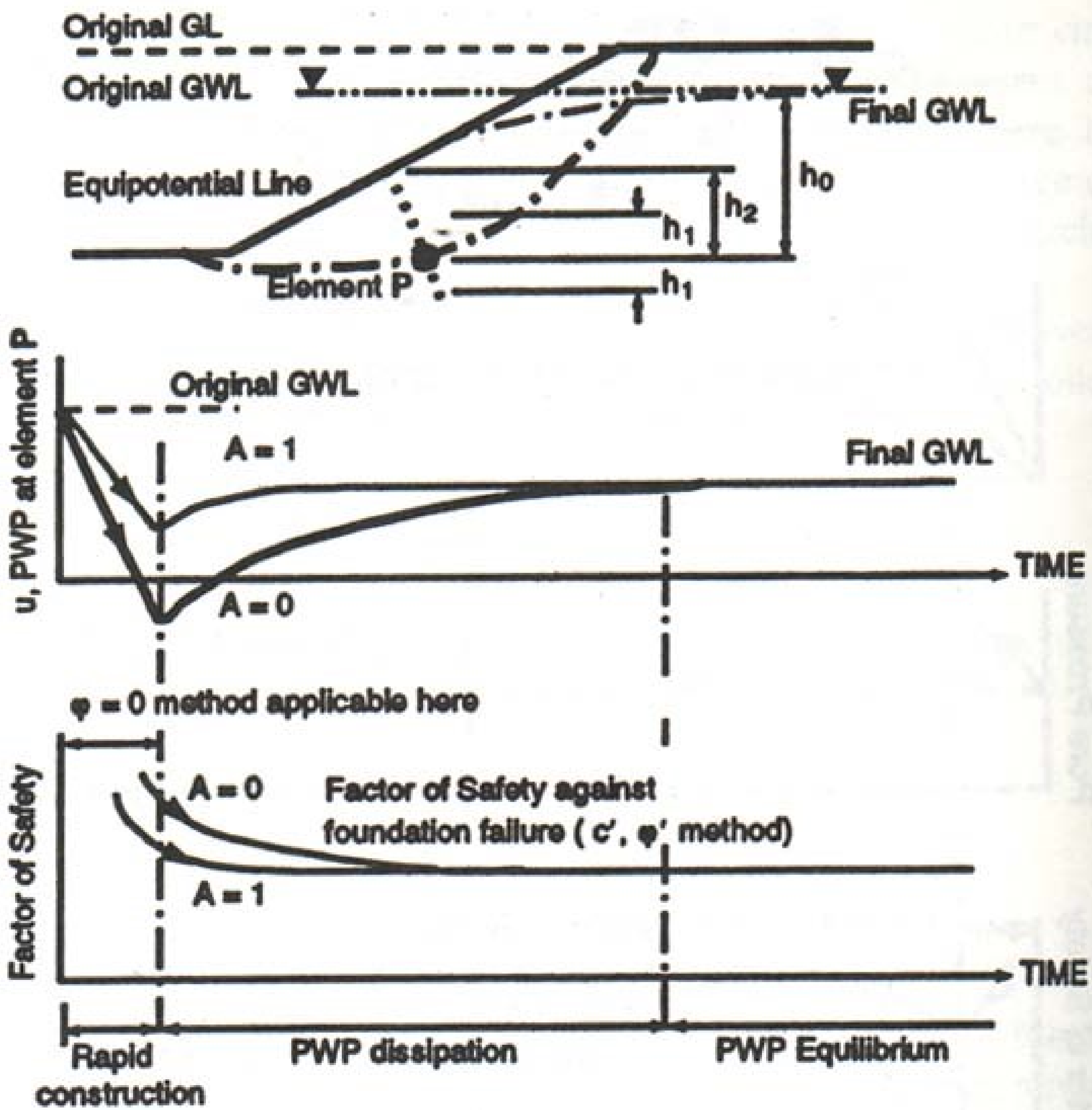


Displacement at embankment toe

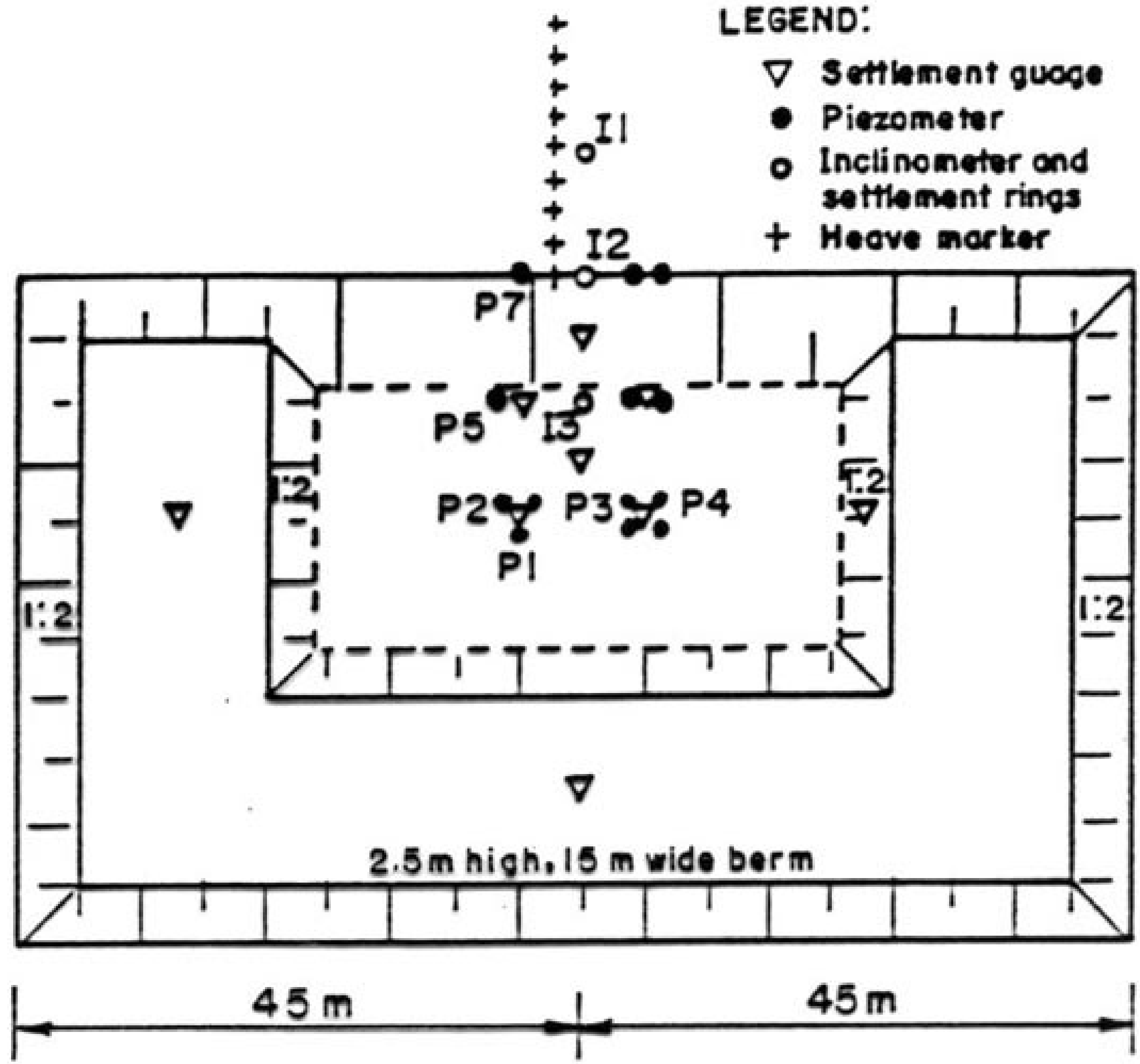
POSSIBLE FAILURE MECHANISM

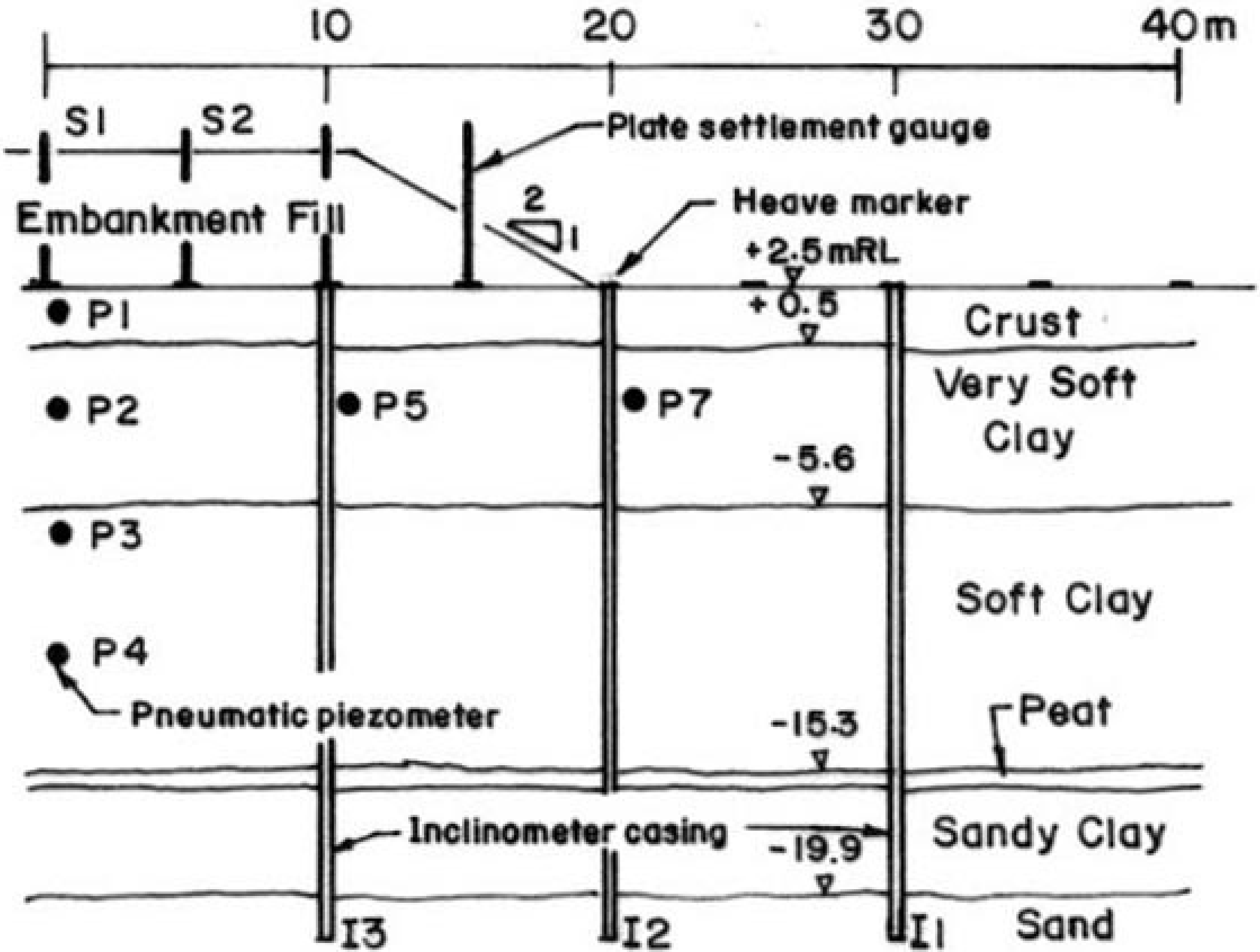




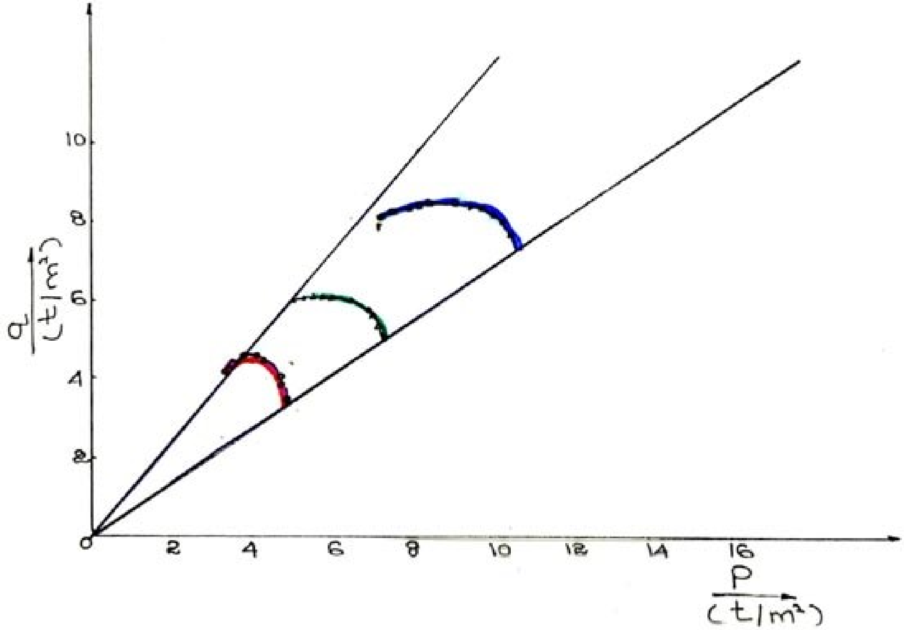


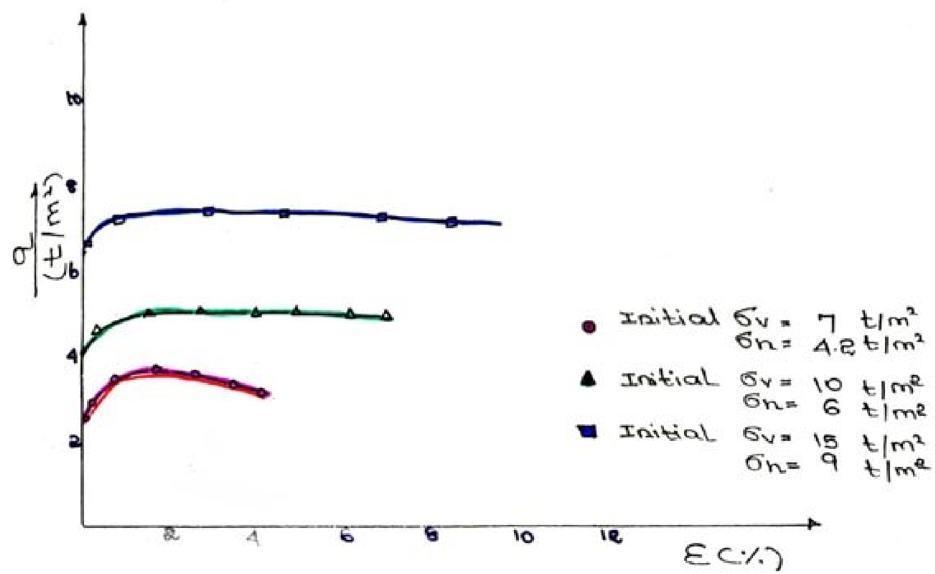
Original GL
Original GWL
Final GWL
Equipotential Line
h_0
h_2
h_1
Element P
h_1
u, PWP at element P
Original GWL
A = 1
Final GWL
TIME
A = 0
φ = 0 method applicable here
Factor of Safety
A = 0
Factor of Safety against
foundation failure (c', φ' method)
A = 1
TIME
Rapid
construction
PWP dissipation
PWP Equilibrium


LEGEND:
Settlement guage
Piezometer
Inclinometer and settlement rings
Heave marker
I1
I2
I3
P1
P2
P3
P4
P5
P7
1:2
1:2
1:2
1:2
2.5m high, 15 m wide berm
45 m
45 m


10
20
30
40m
S1
S2
Plate settlement gauge
Embankment Fill
2
1
Heave marker
+2.5mRL
+0.5
P1
Crust
P2
P5
P7
Very Soft Clay
-5.6
P3
Soft Clay
P4
Pneumatic piezometer
-15.3
Peat
Inclinometer casing
-19.9
Sandy Clay
I3
I2
I1
Sand

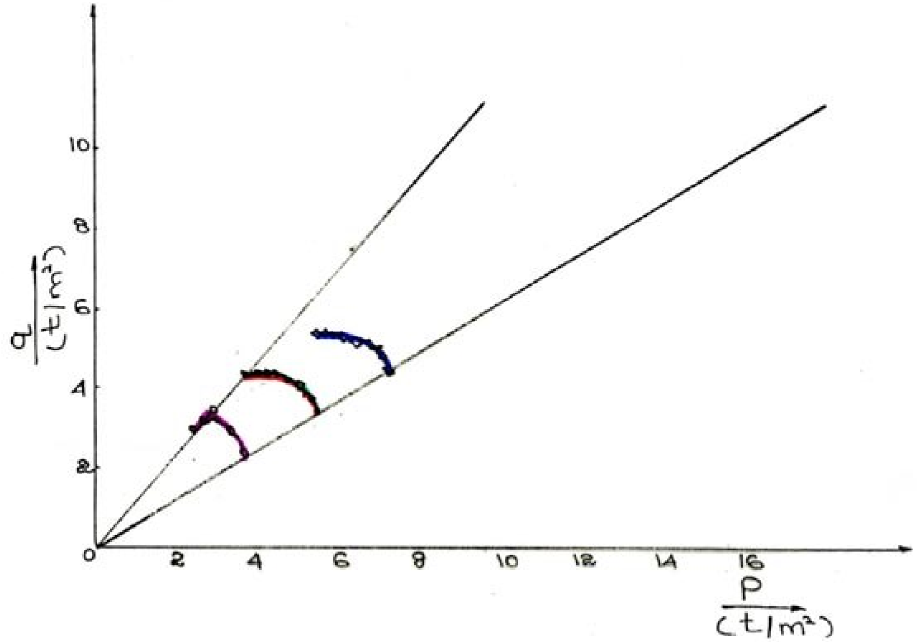
Sample Depth 7-8m

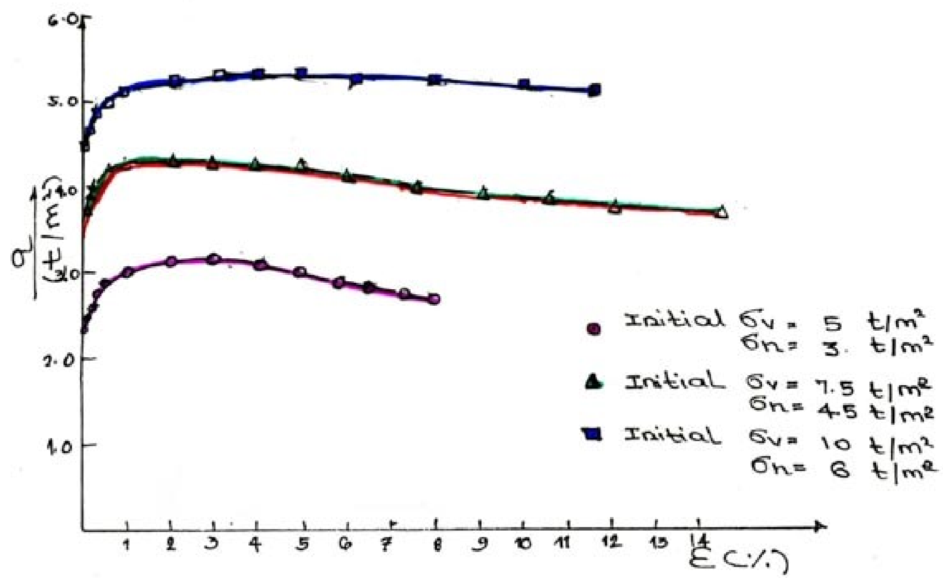




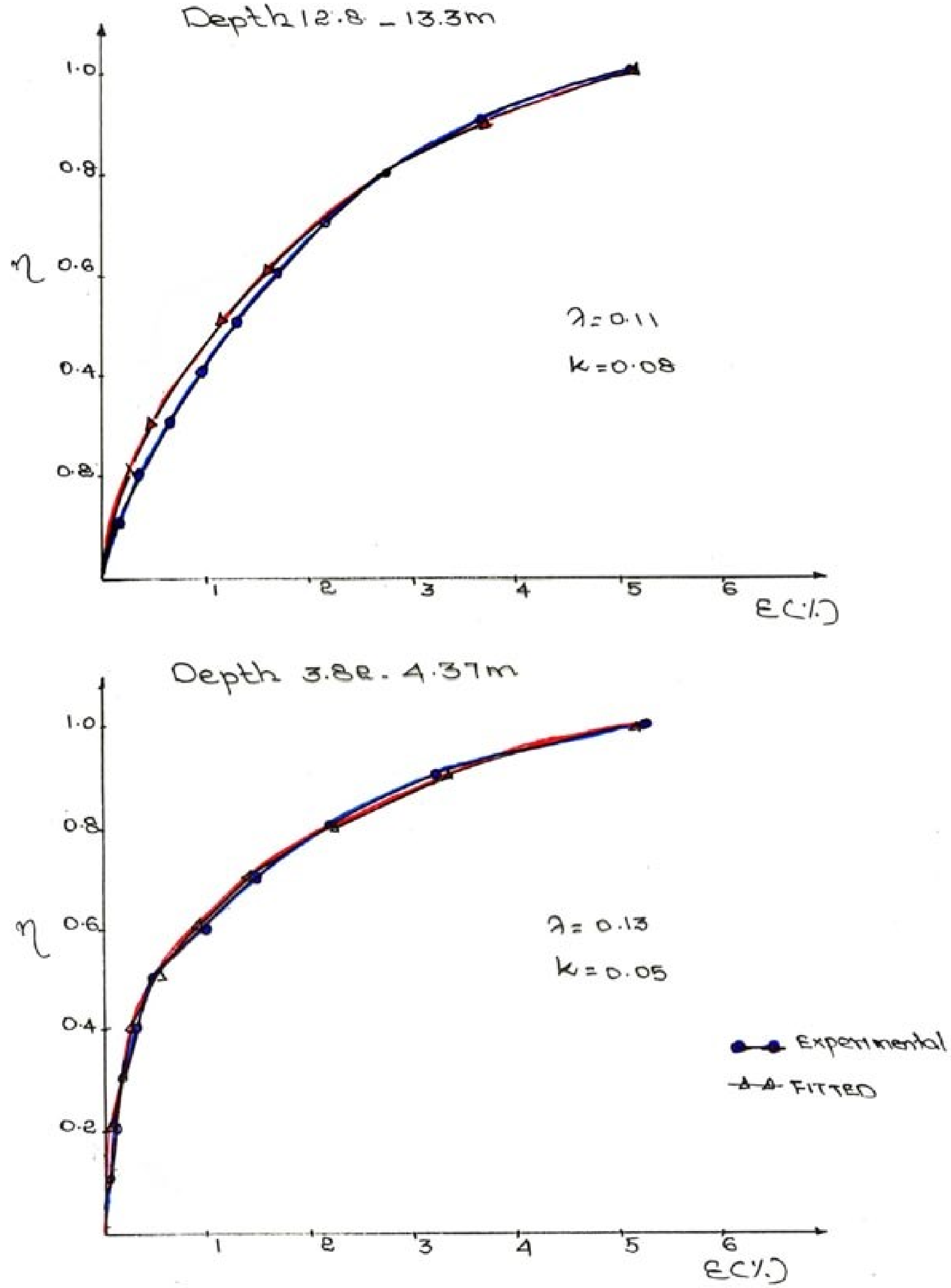
CKoU TEST RESULTS

Sample Depth 4.5 _ 5.5 m

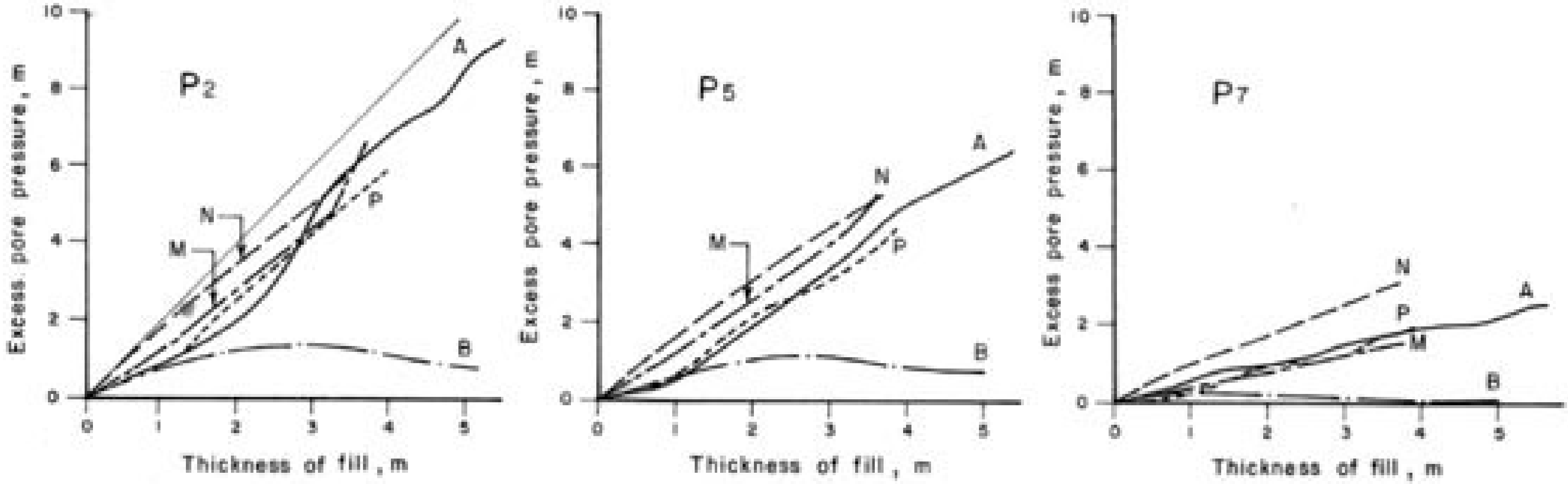




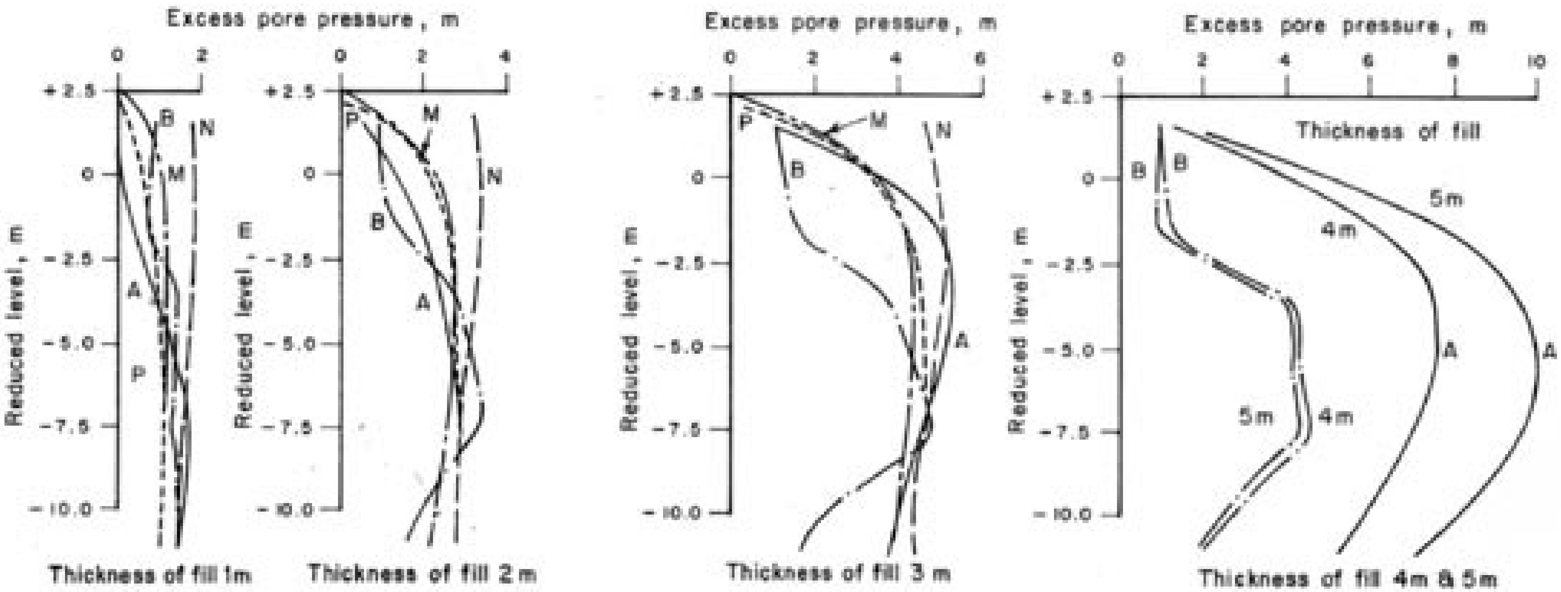
CK$_0$U TEST RESULTS



Experimental & Fitted Curves [CIU TEST]



Comparison of predicted and actual excess pore pressures in piezometers P2, P5, P7 throughout embankment construction (A = Actual, B = Balasubramaniam, M = Magnan, N = Nakase, P = Poulos)



Comparison of predicted and actual excess pore pressure profiles under the embankment center line during construction (A = Actual, B = Balasubramaniam, M = Magnan, N = Nakase, P = Poulos)

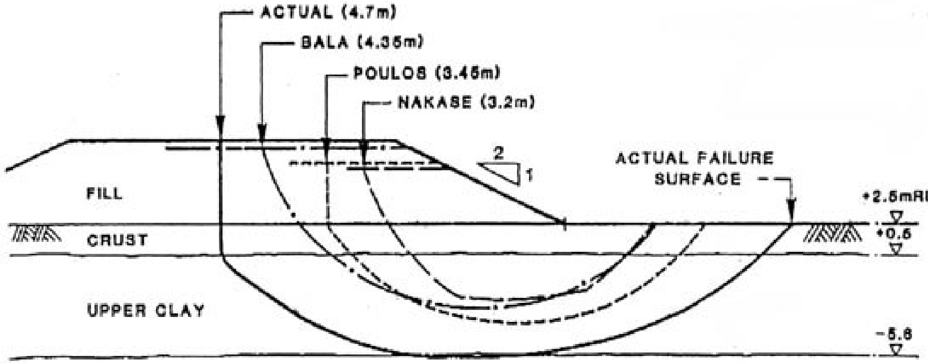


Figure A14 Actual and predicted failure heights and failure surfaces

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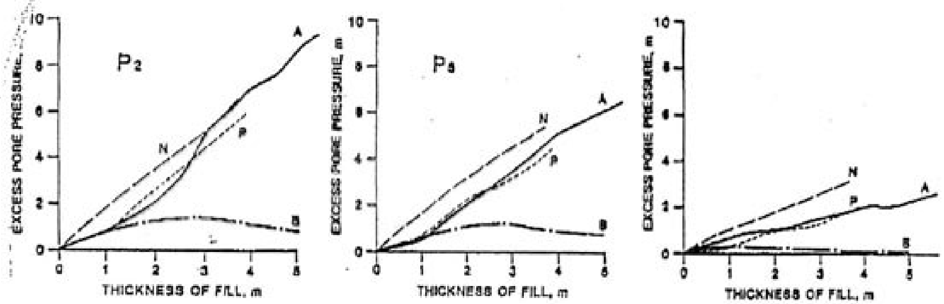


Figure A15 Actual and predicted excess pore pressures at P2, P5 & P7

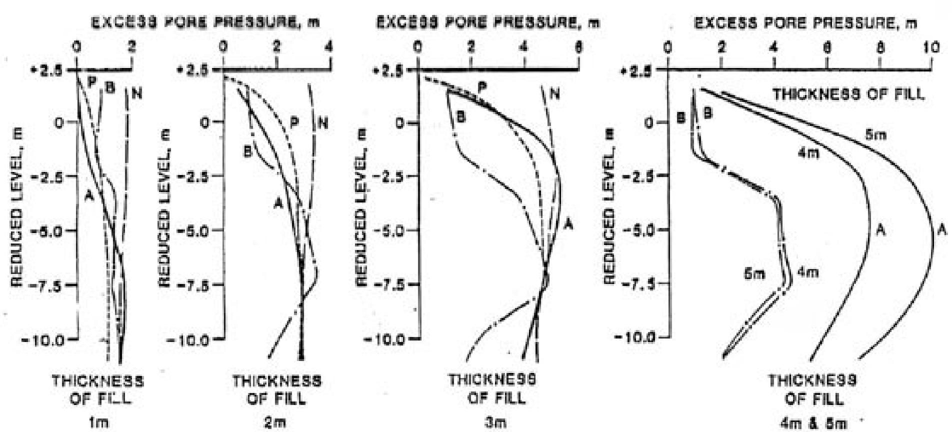
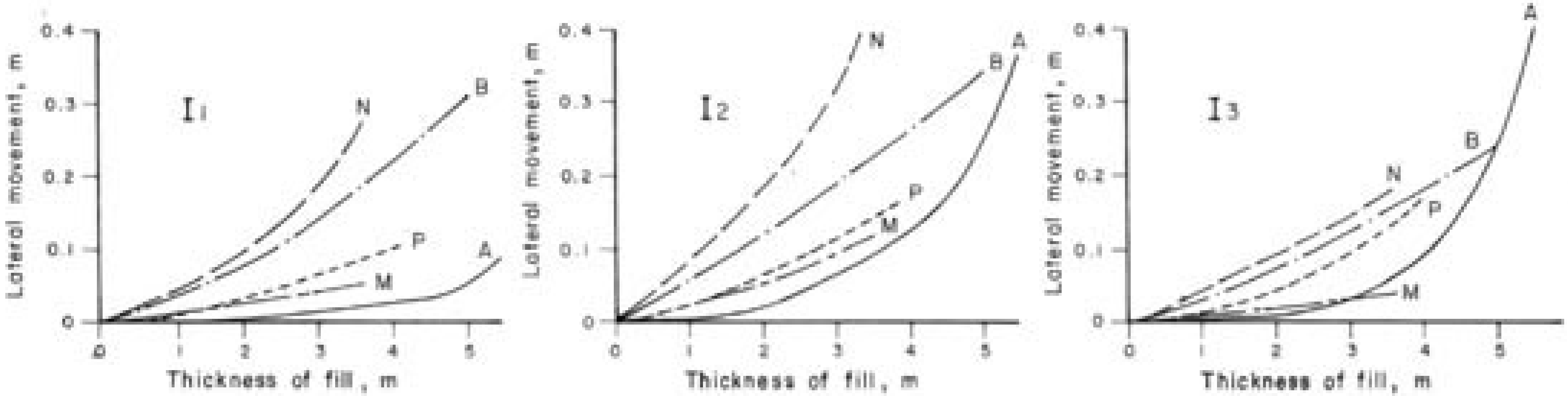
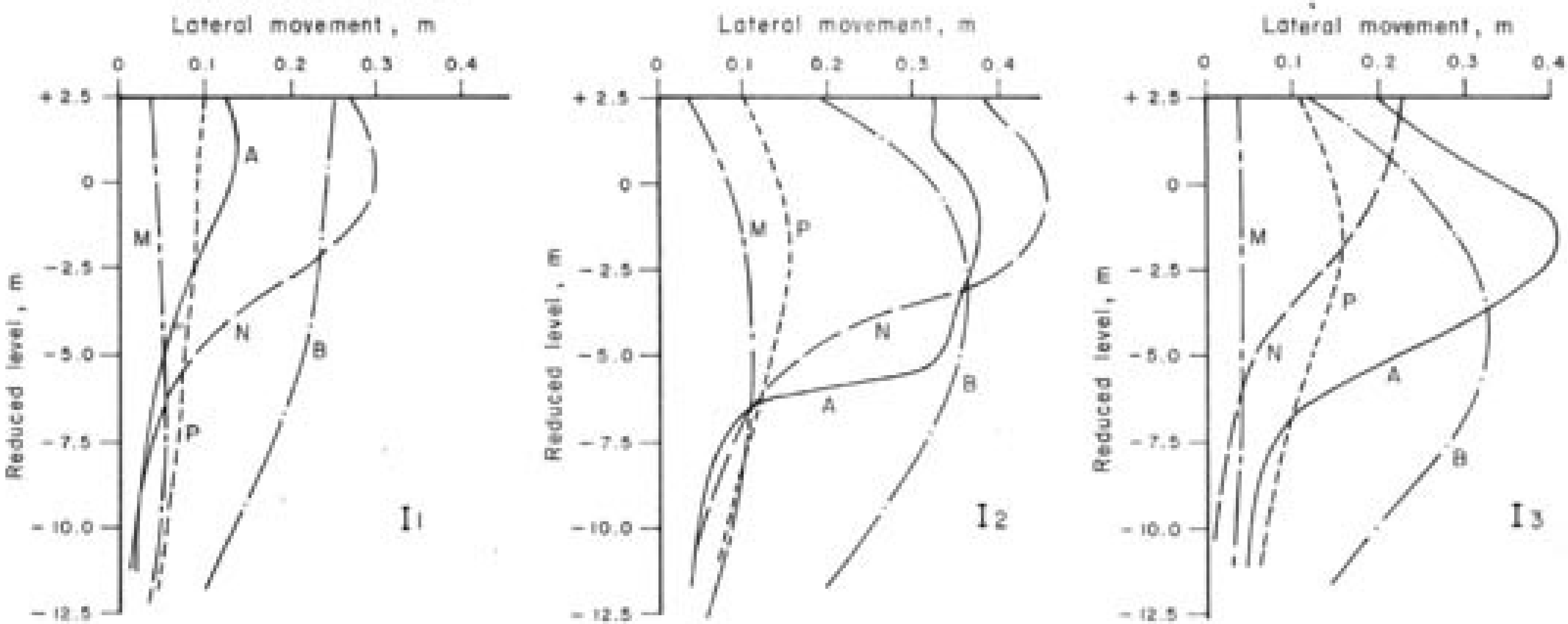


Figure A16 Actual and predicted excess pore pressure profiles under embankment centre



Comparison of predicted and actual horizontal displacements at a 4.5 m depth in inclinometers I1, I2 & I3 throughout embankment construction A = Actual, B = Balasubramaniam, M = Magnan, N = Nakase, P = Poulos)



Comparison of predicted and actual vertical profiles of horizontal displacements in inclinometers I1, I2, & I3 at failure (A = Actual, B = Balasubramaniam, M = Magnan, N = Nakase, P = Poulos)

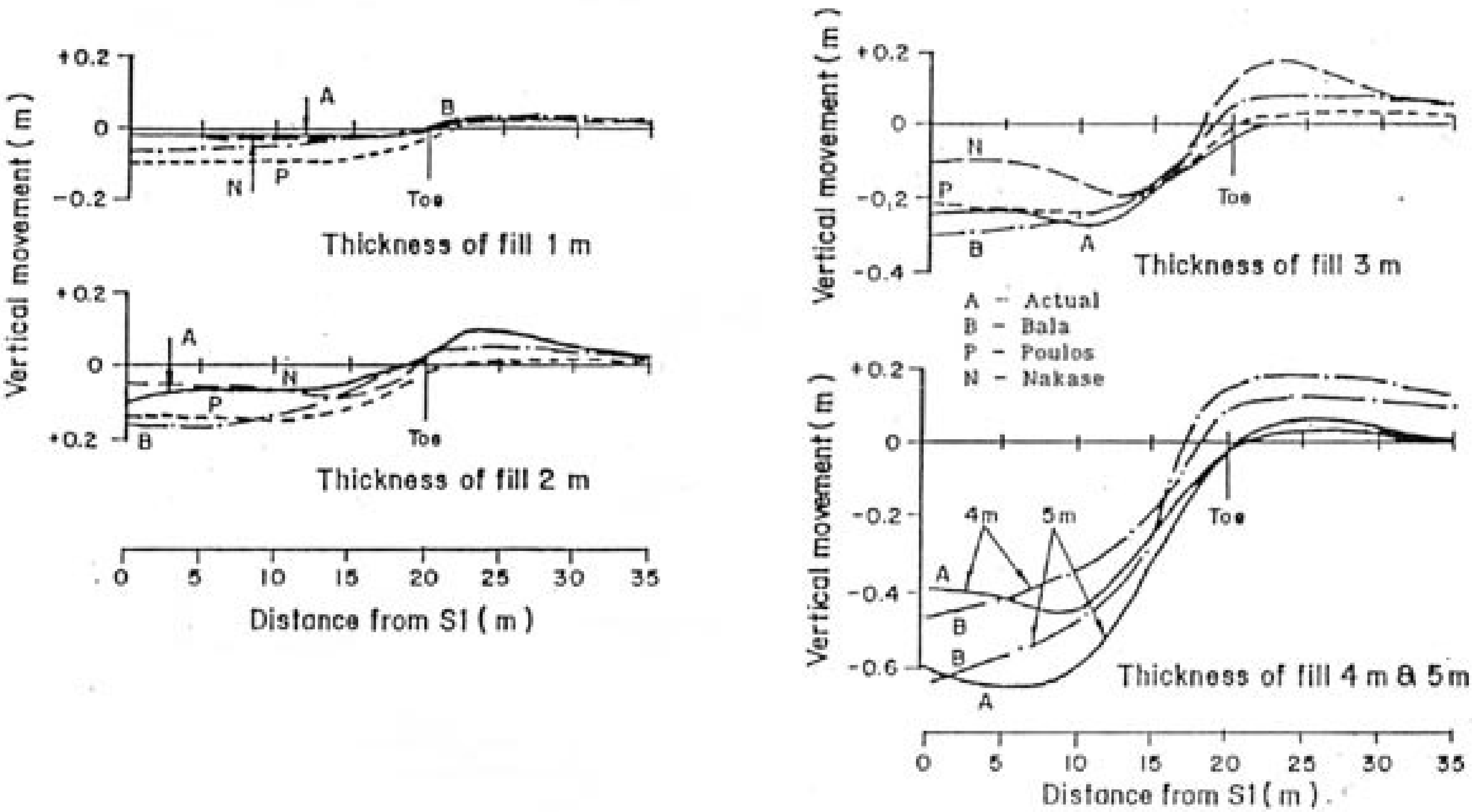


Fig. 10 Actual and predicted surface settlements
(after E.W. Brand & J. Premchitt)