Analysis of Test Embankments in Soft Clays with and without Ground Improvement

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Geotechnical Parameters, Stability & Settlement Evaluations

Stability
Lateral Movement
Settlement
Rate of Settlement
Residual Settlement

Geotechnical Investigation at Nong Ngo Hao Airport Site

Phase	Year	Title
I	1972 - 1974	Geotechnical Investiga-tions by Asian Institute of Technology and N.D. Lea and Associates, Kampsax
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IV	1993 - 1995	Full scale Field test of Prefabricated vertical drains by the Asian Institute of Technology

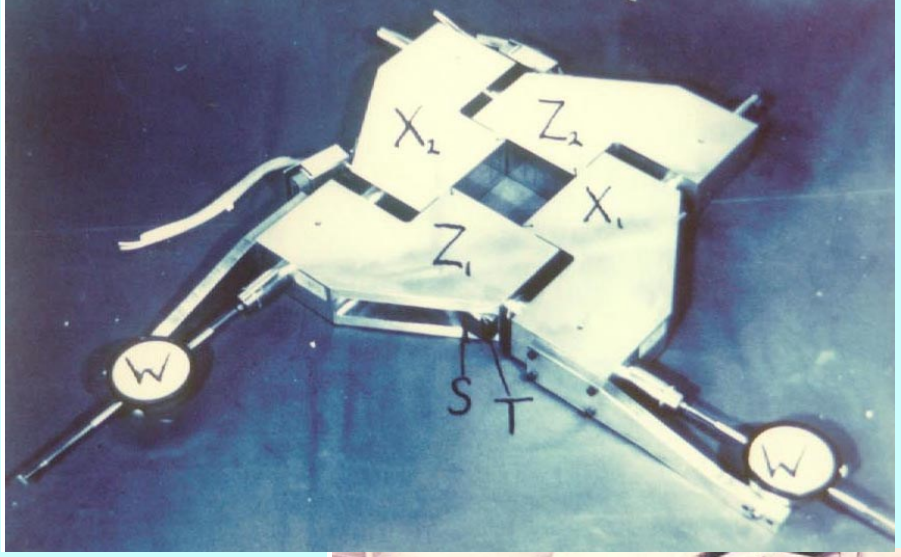
Full scale fields with prefabricated vertical drains (PVD) at the Second Bangkok International Airport

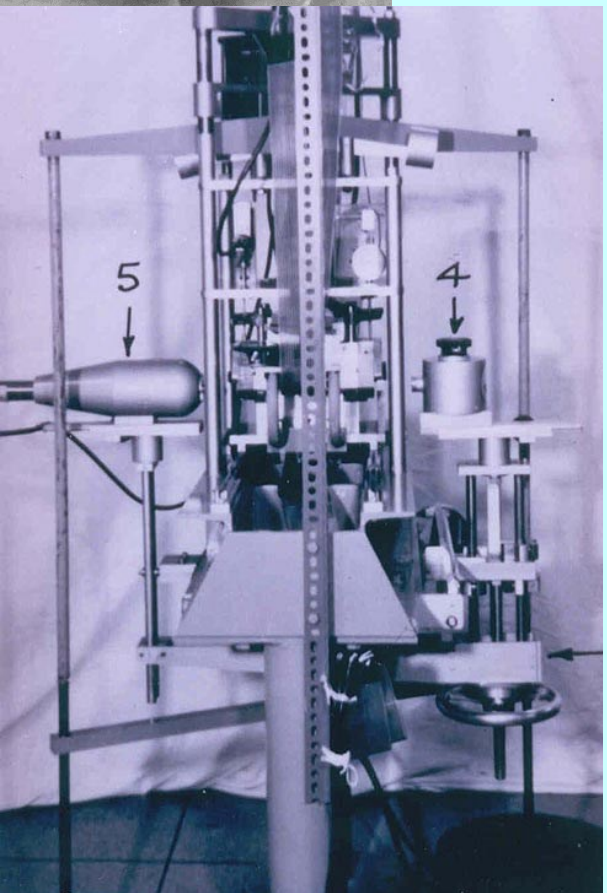
Laboratory testing

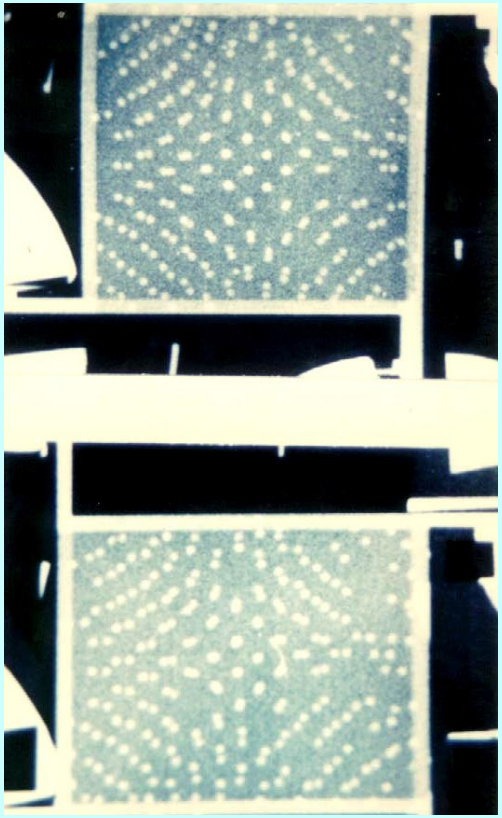
Apparently Karl Terzaghi had little faith in single element tests, theories, and model tests both 1g and centrifugal tests with multiple g

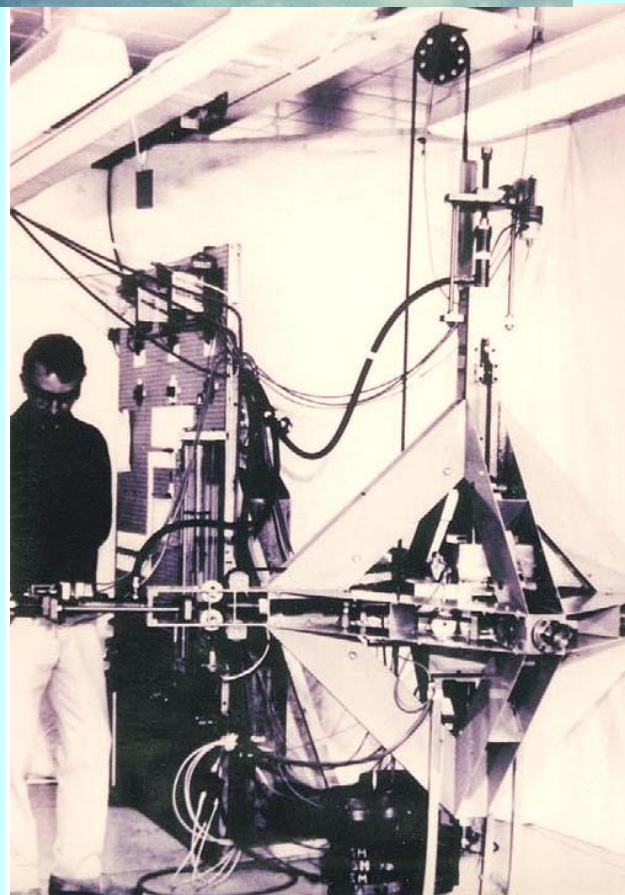
Karl Terzaghi

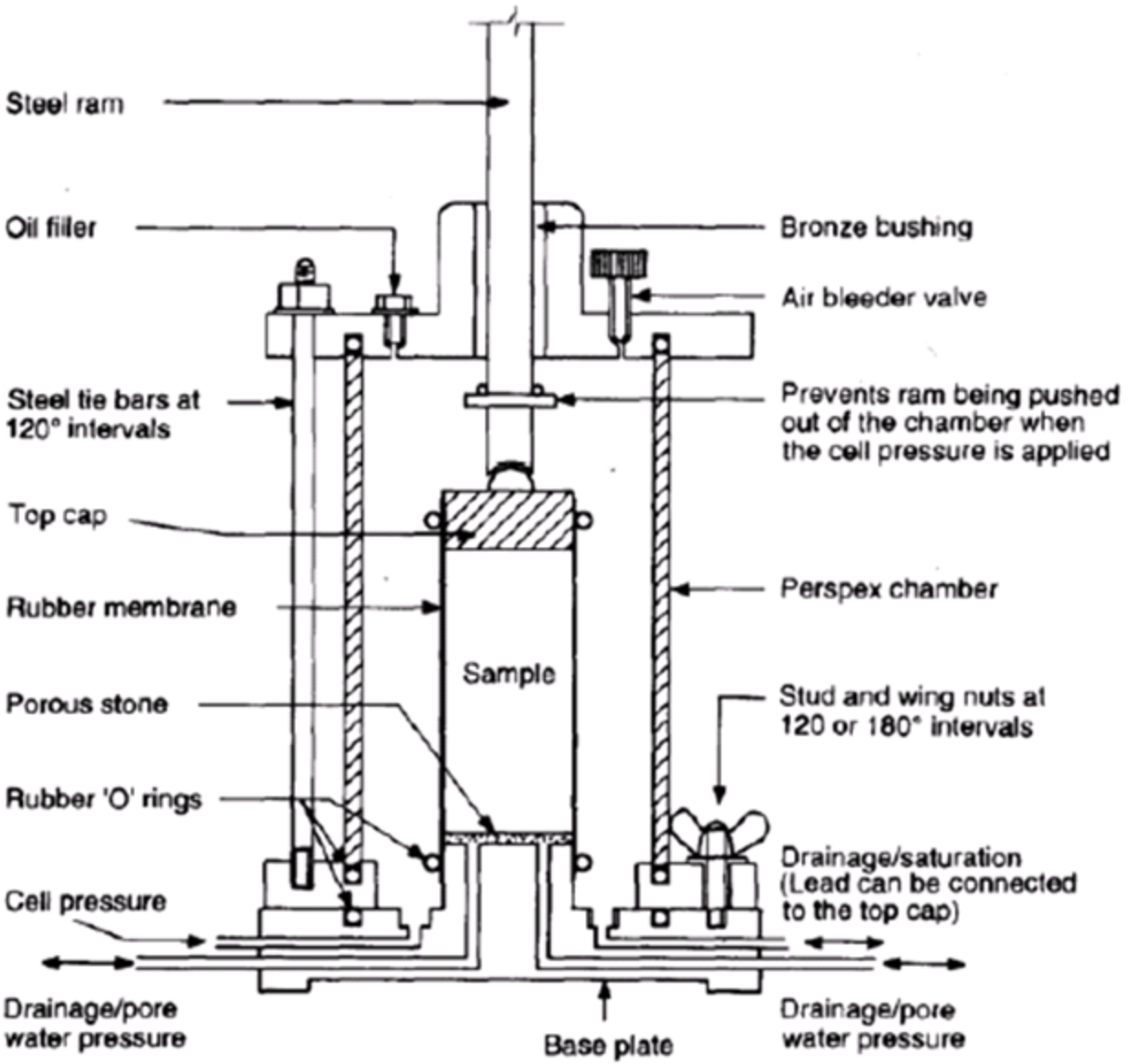
Laboratory testing



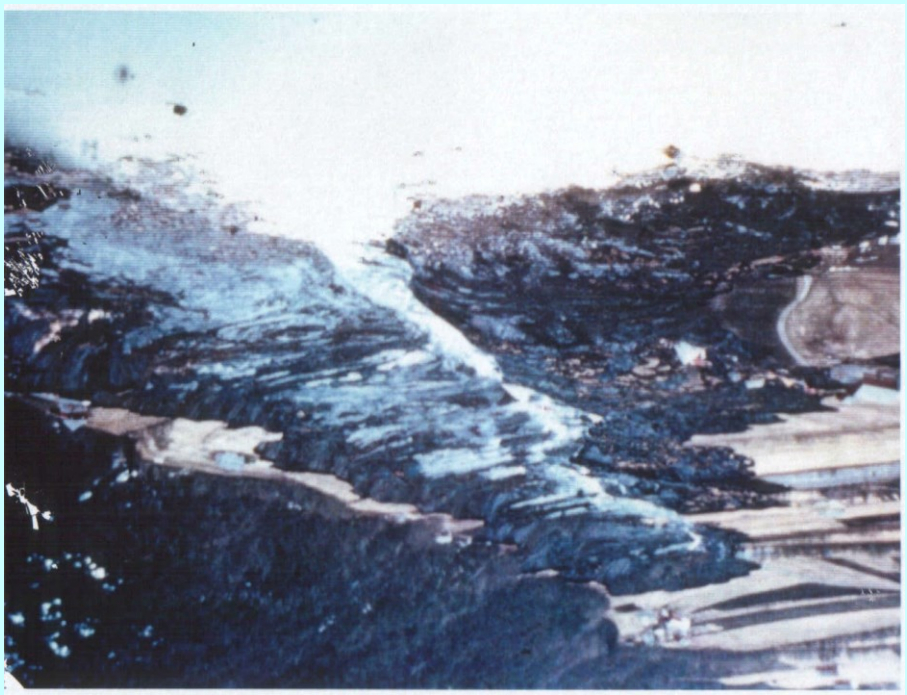


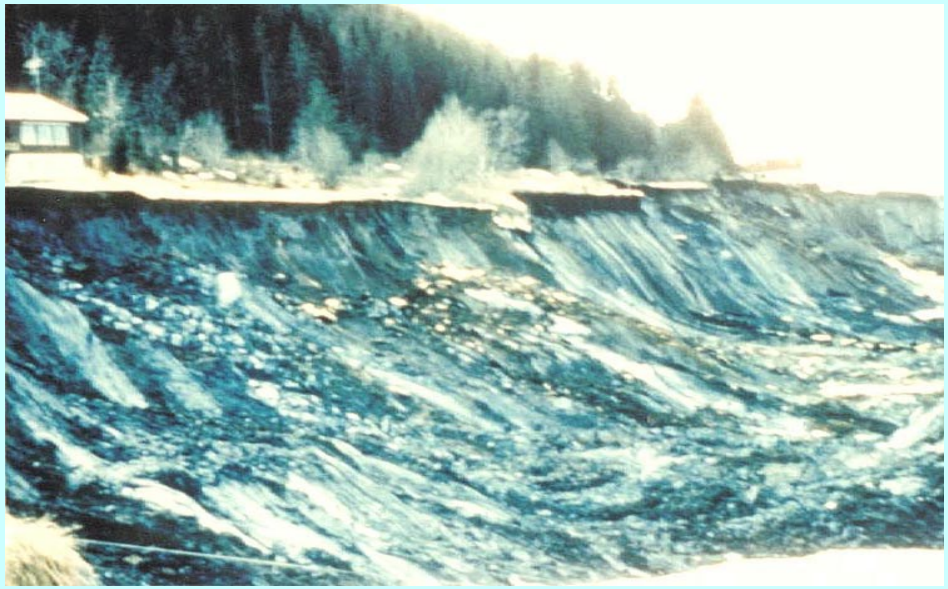


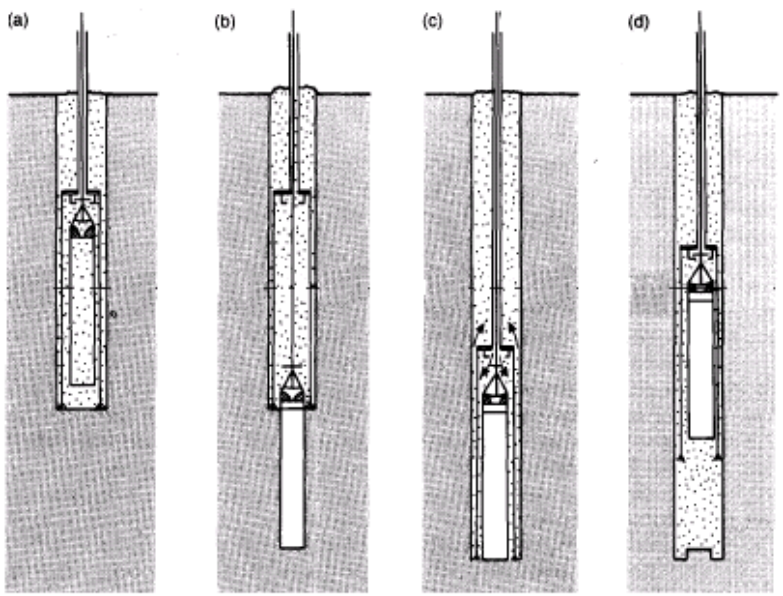




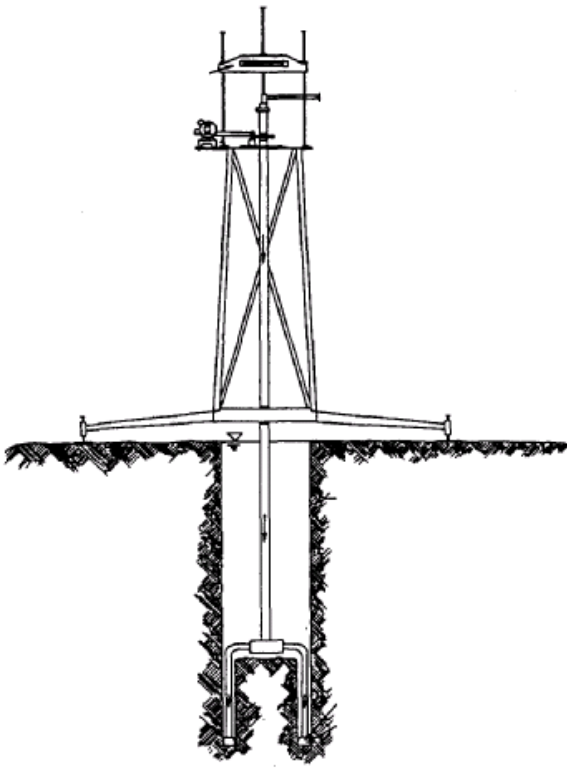
Triaxial cell

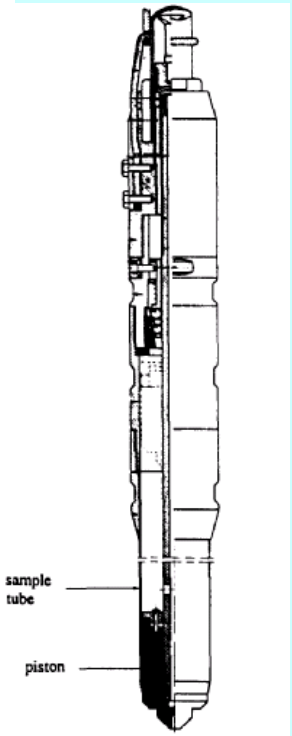




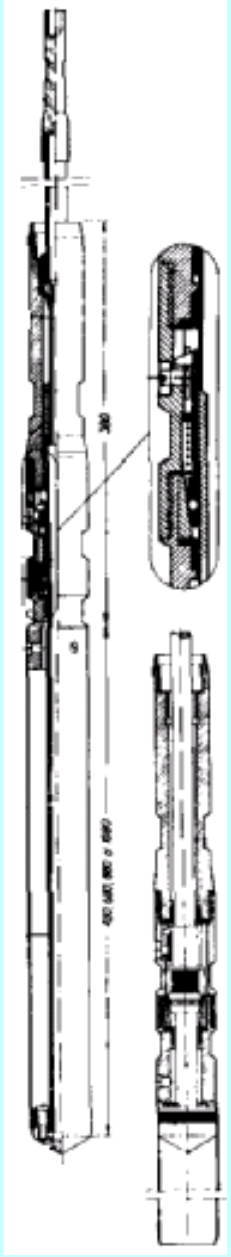


General operation of the Laval sampler

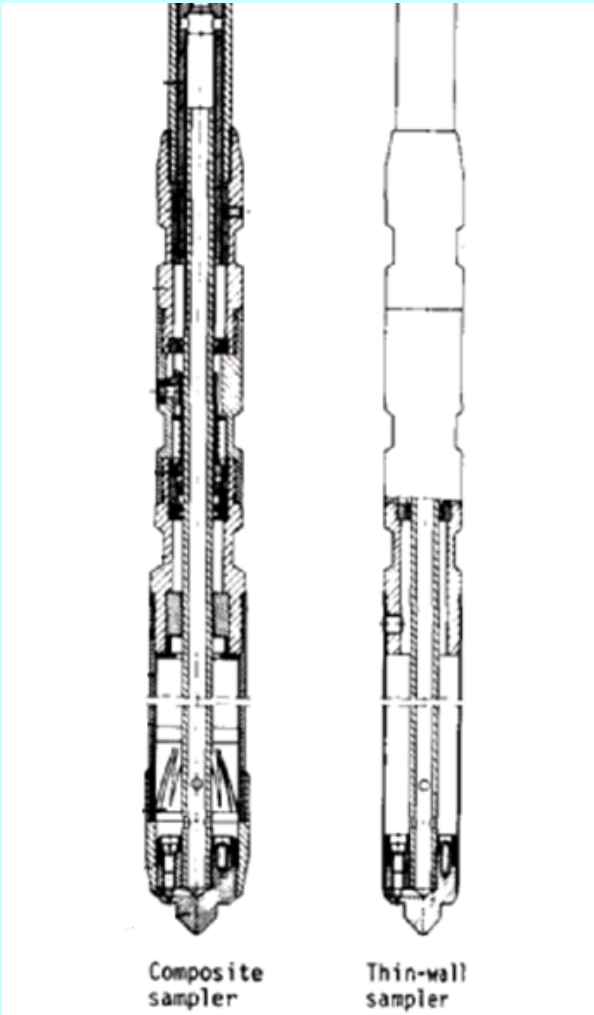




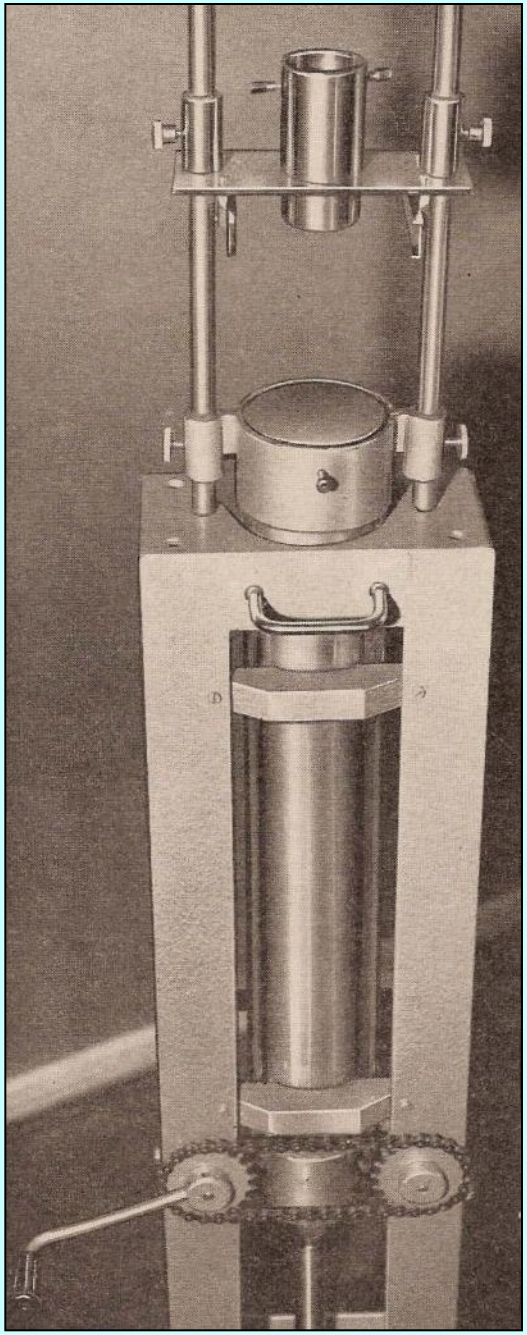
NGI 95 mm piston sampler

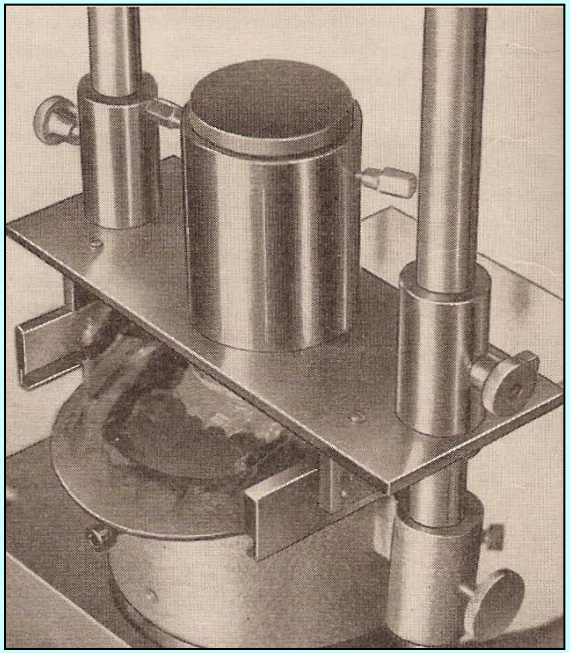


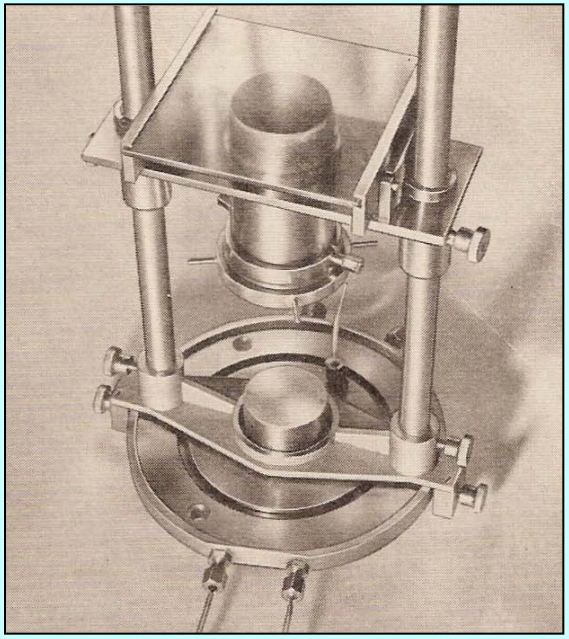
Original NGI 54-mm piston sampler.

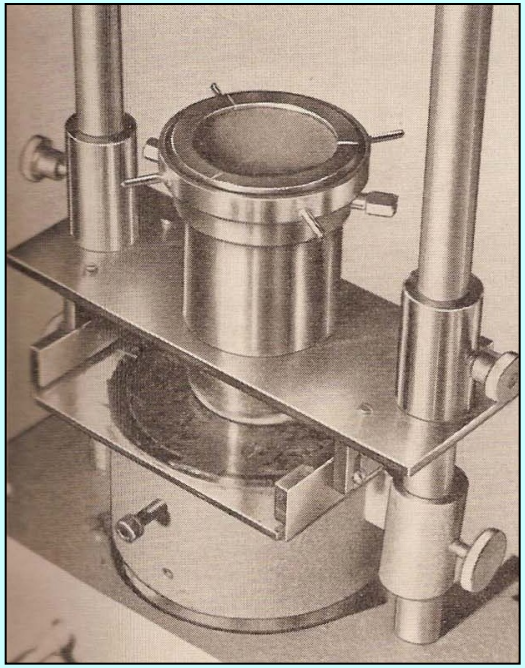


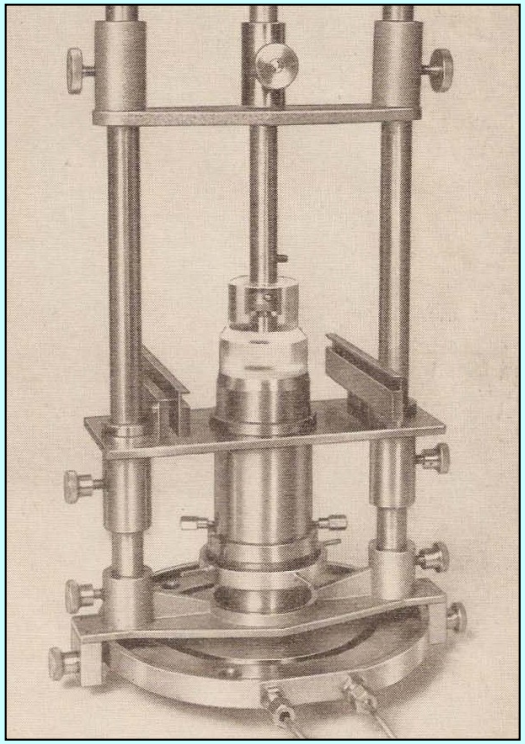
Modified NGI 54-mm piston sampler

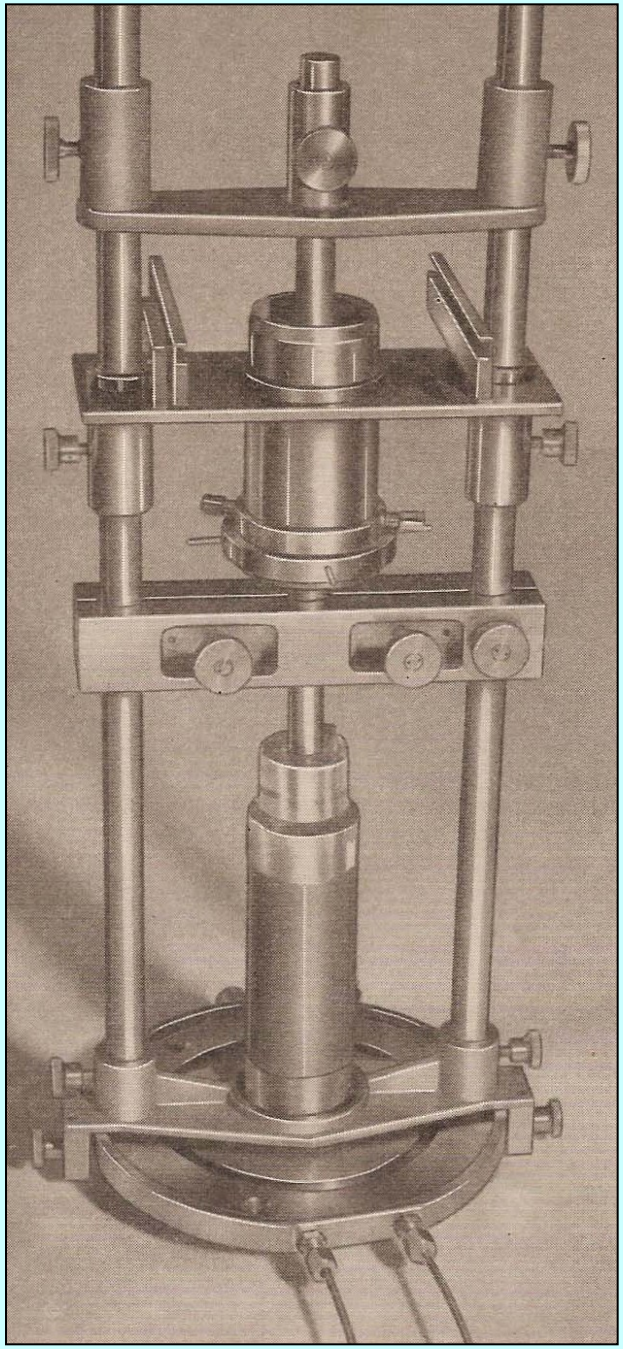


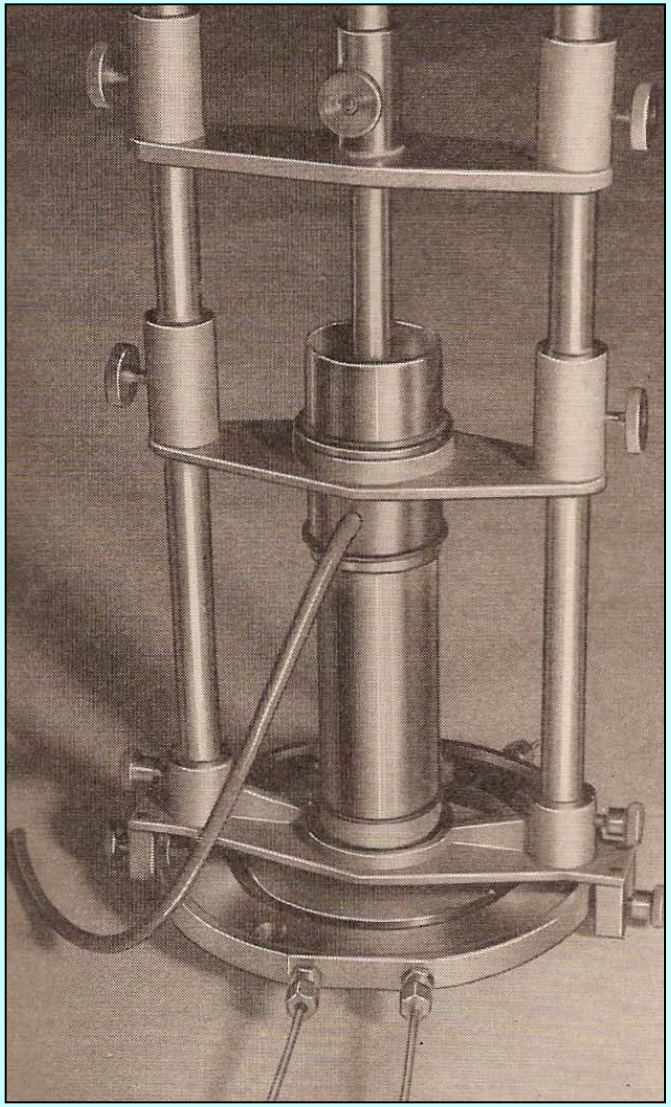


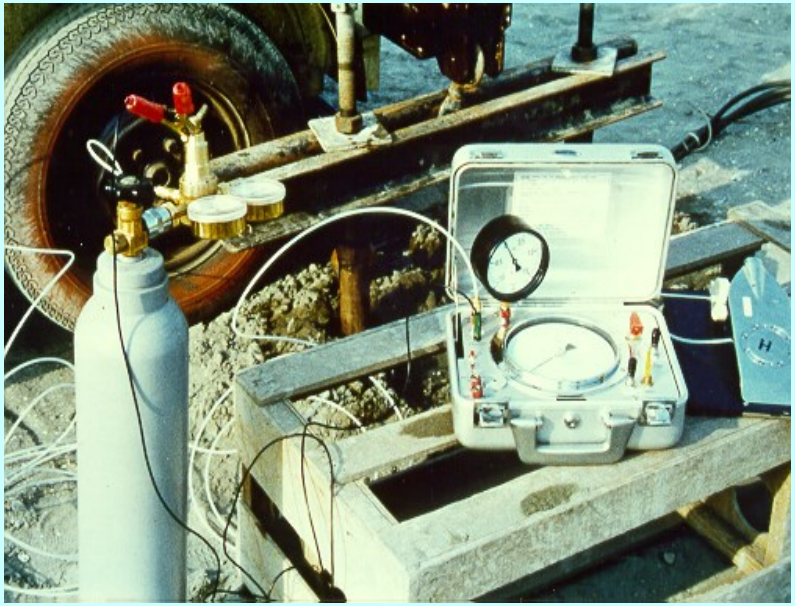












Vane tests

CPT-CPTu

Pressuremeter

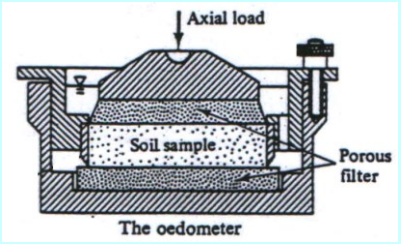
Dilatometer

Large Scaled Tests and Instrumentation

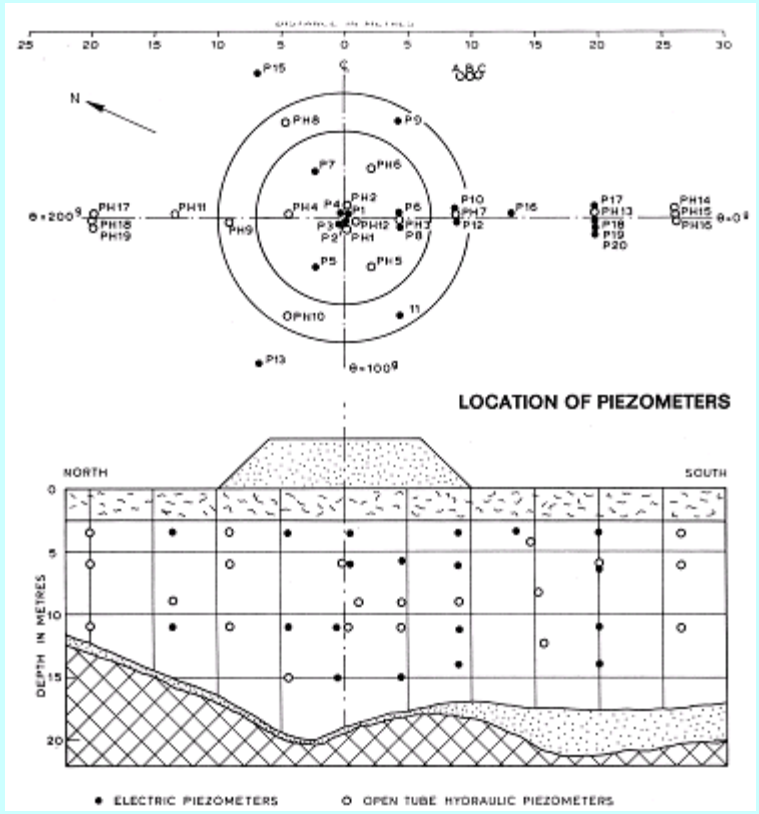
Major projects

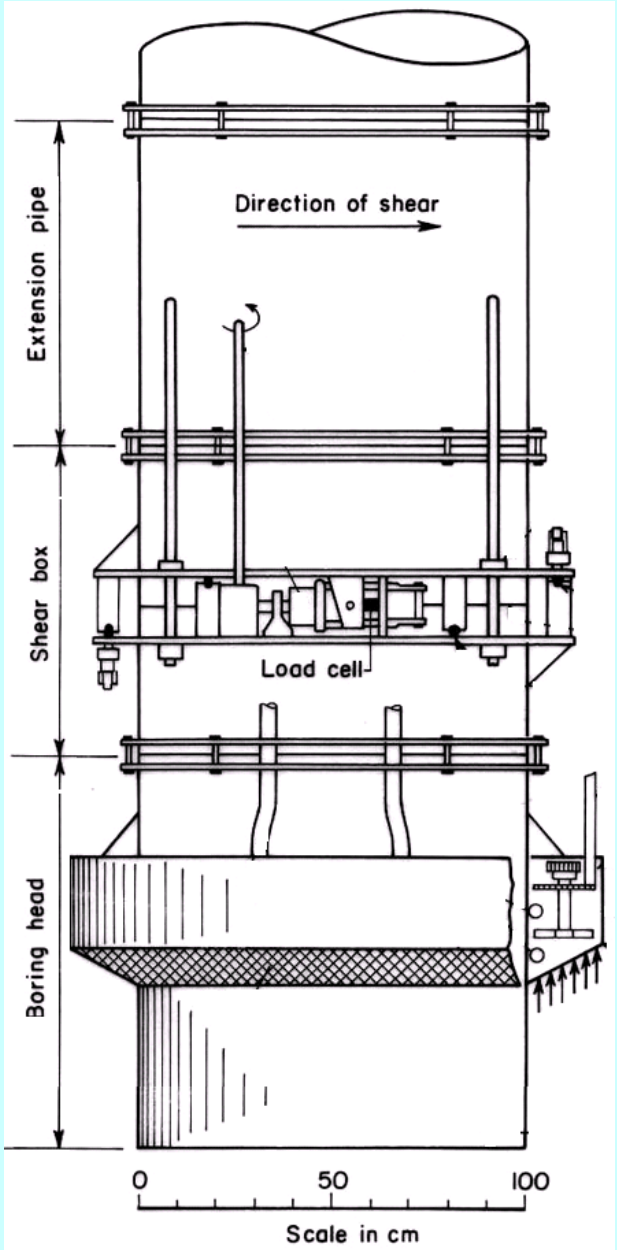
- **Oslo, Norway**
- **Bangkok, Thailand**
- **Muar site, Malaysia**
- **Southeast Queensland**

Large Scaled Tests and Instrumentation

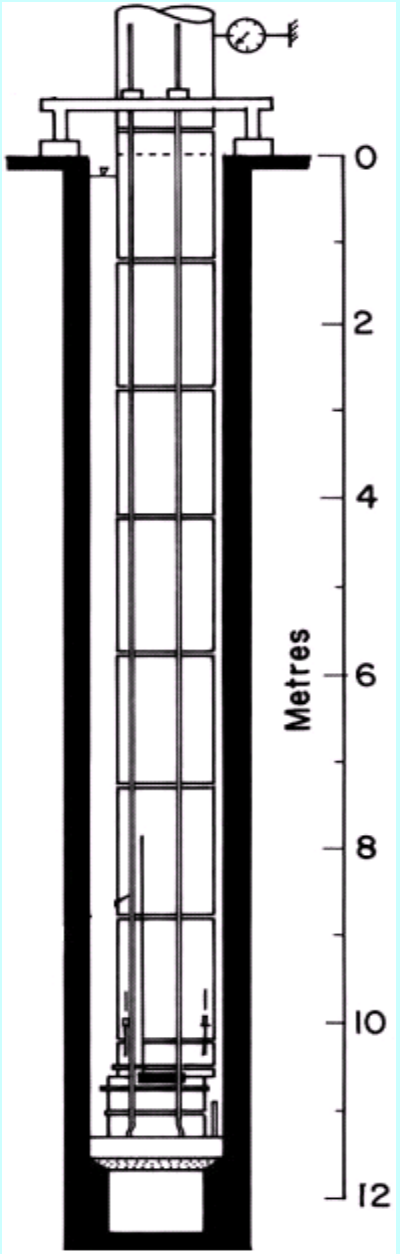


The oedometer





Large scaled Direct Shear Test

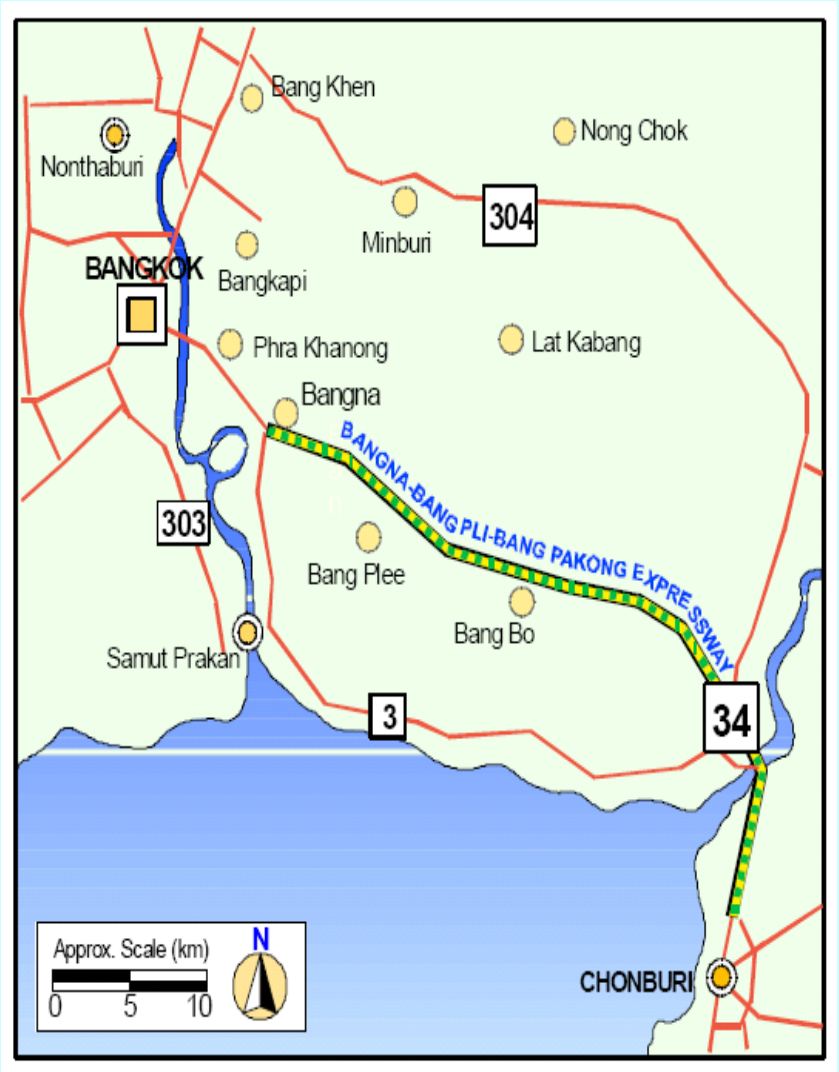


Test in progress

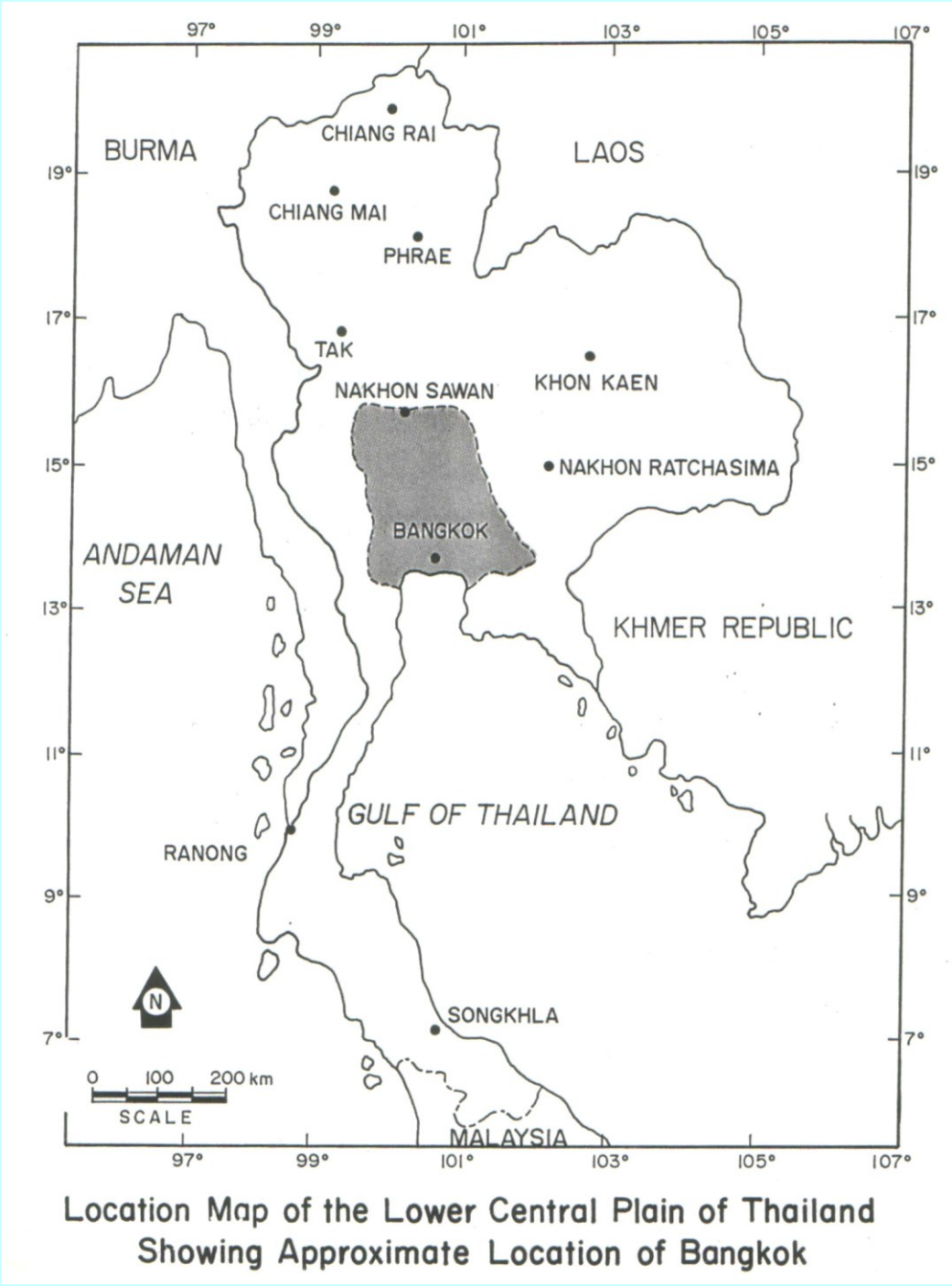
1. Gold Coast Highway
   (stone columns)
2. Sunshine Coast Motorway
   (Vertical drains-PVD)
3. Port of Brisbane Motorway
   (Vertical drains-PVD)

Major Projects

1. Instrumented Piles in Sand, Holmen, Drammen, Norway
2. Slurry trench excavations for Oslo Subway
3. Changi International Airport, Singapore
4. Royal Thai Navy Dockyard, Bangkok, Thailand
5. Swarnabhumi International Airport, Bangkok, Thailand
6. North- South Expressway, Malaysia
7. Colombo-Katunayake Expressway, Sri Lanka
8. Bangpo-Thermal Power Plant, Bangkok, Thailand



Large Scaled Tests and Instrumentation



Location Map of the Lower Central Plain of Thailand
Showing Approximate Location of Bangkok

Geotechnical Investigation at Nong Ngo Hao Airport Site

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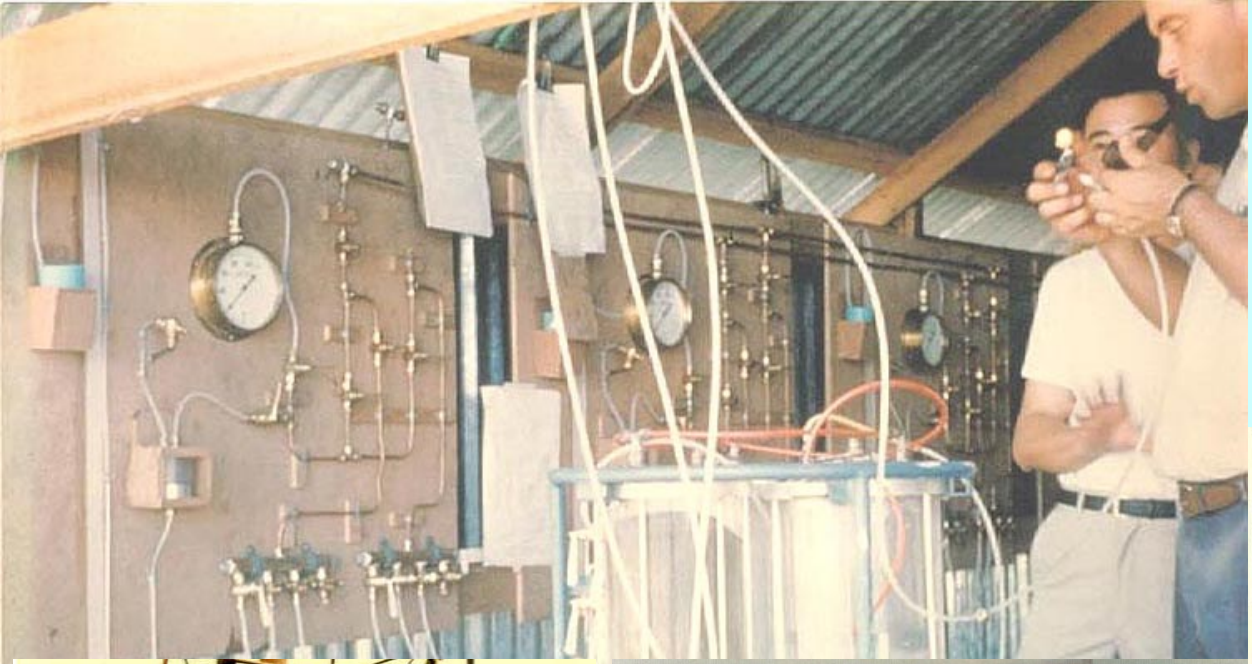
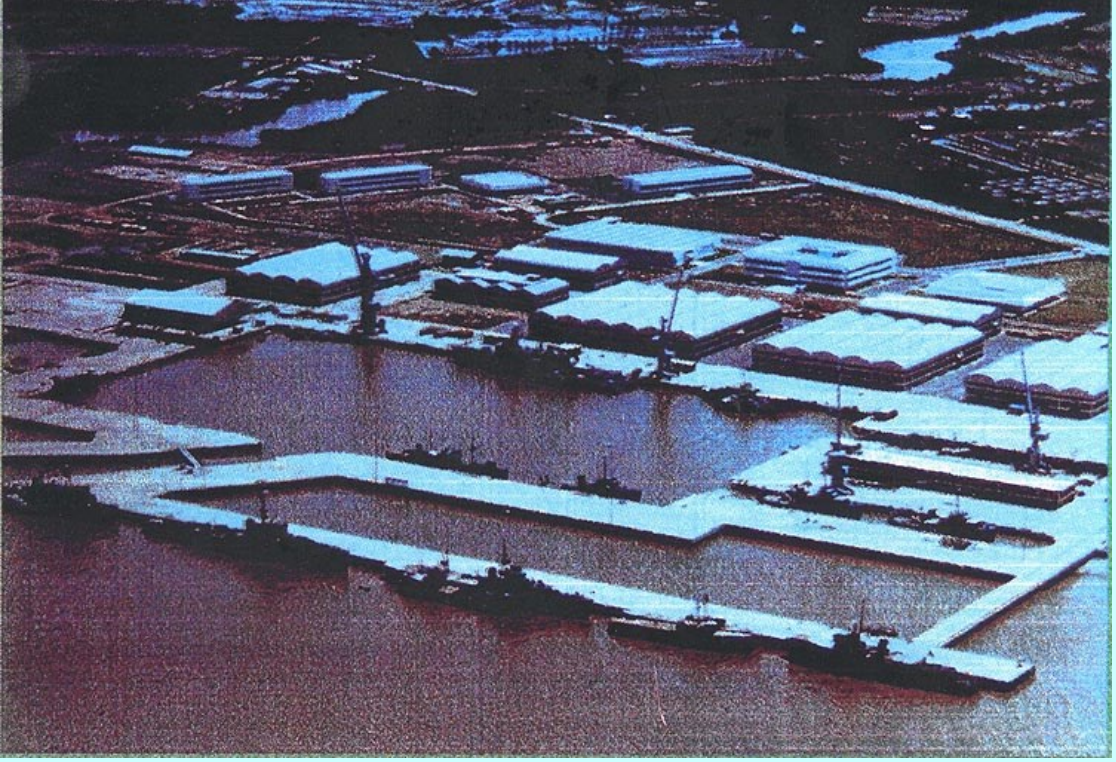
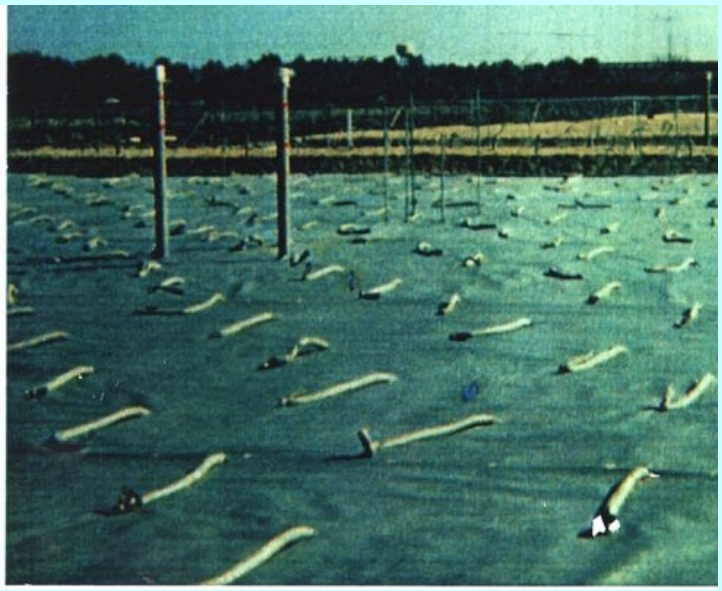
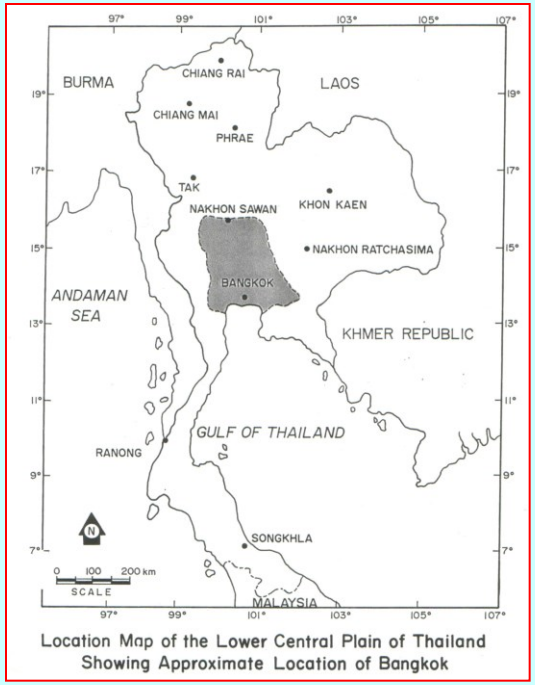






Royal Thai Navy Dockyard



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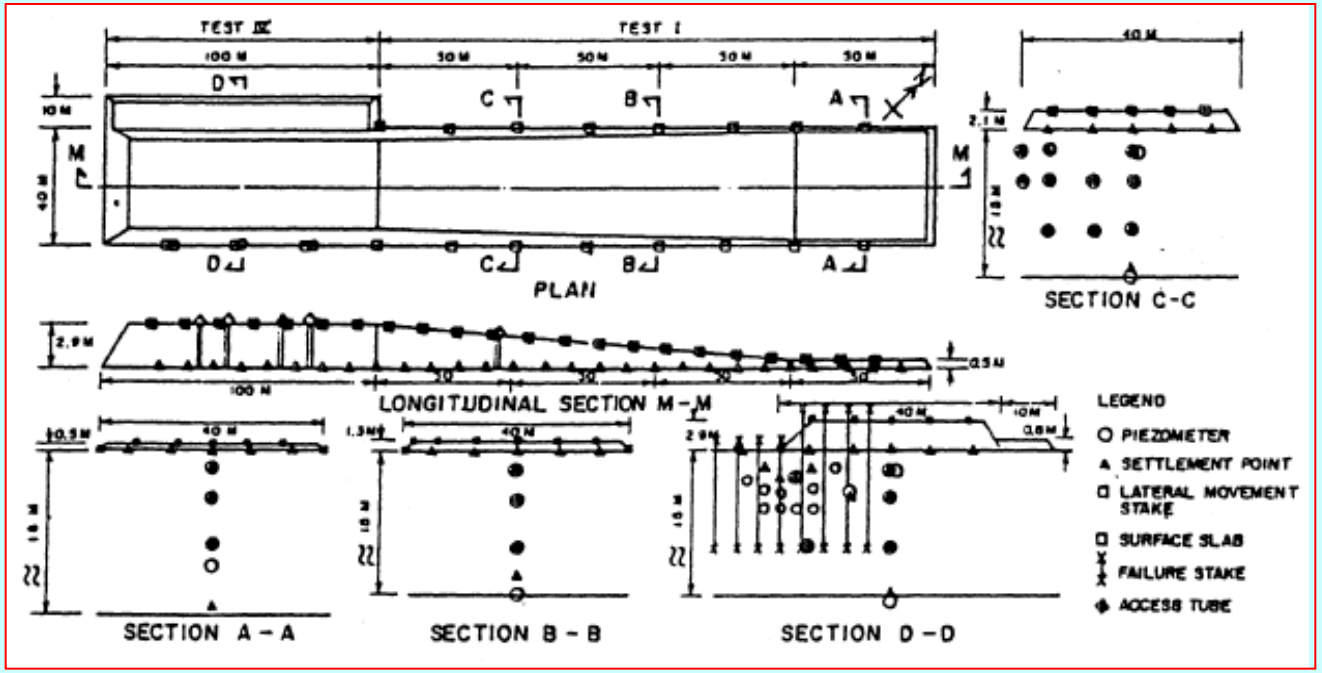


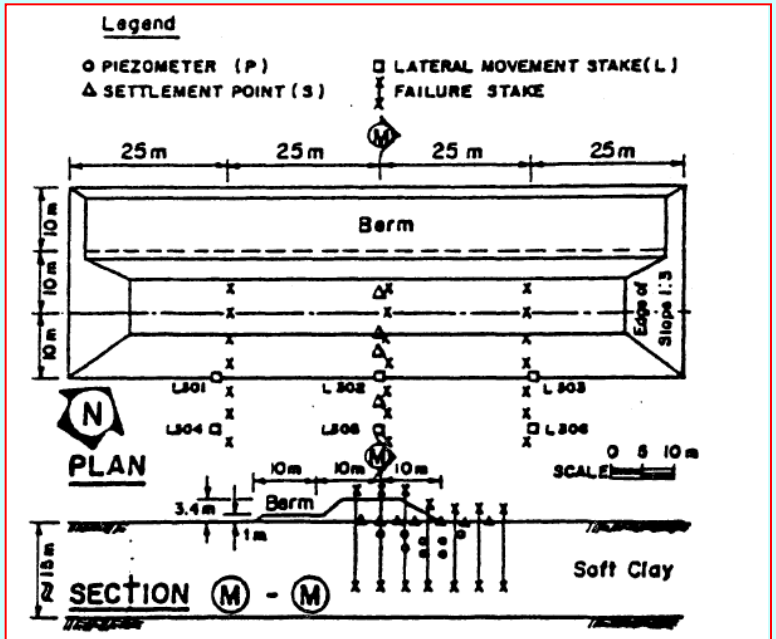
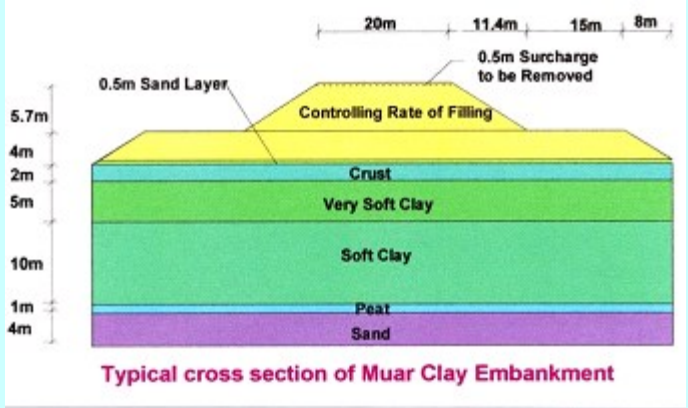
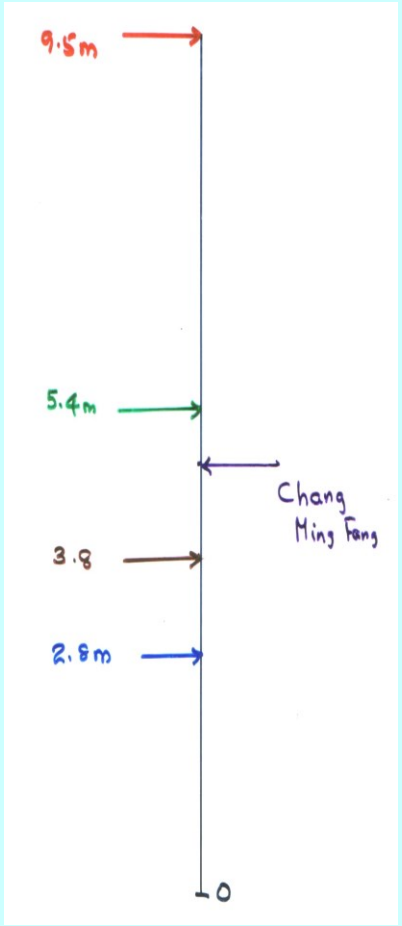
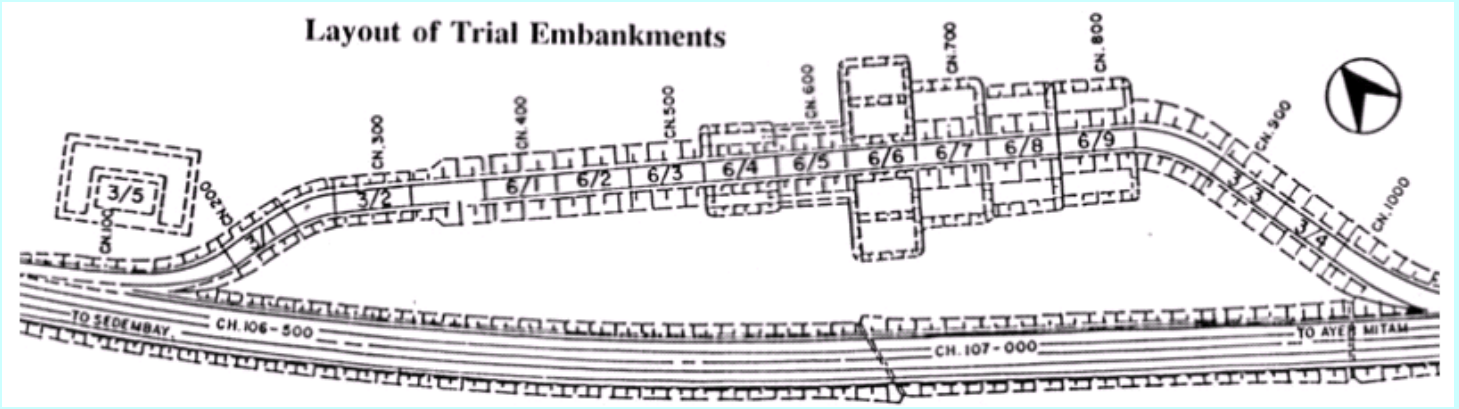


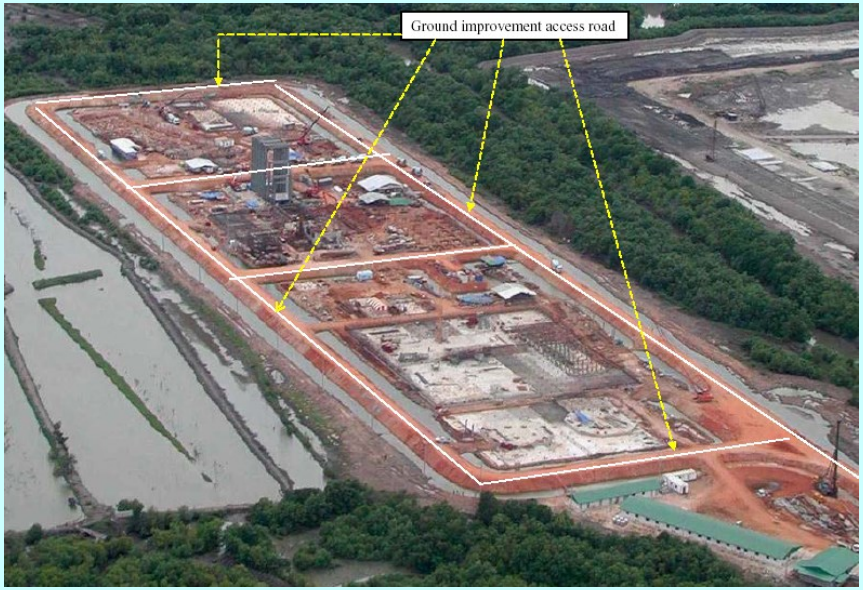
Fig. 1. Embankment I Taken to Failure.

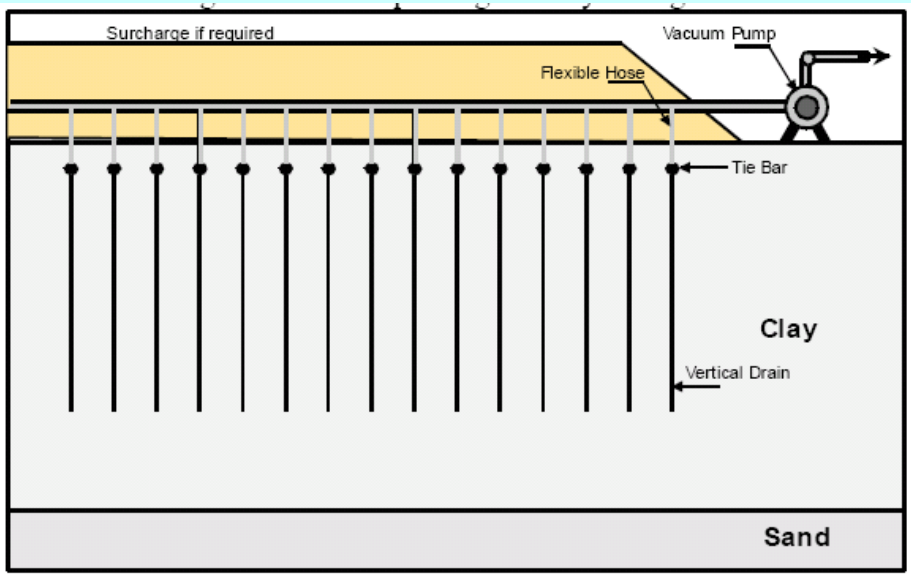


Typical cross section of Muar Clay Embankment

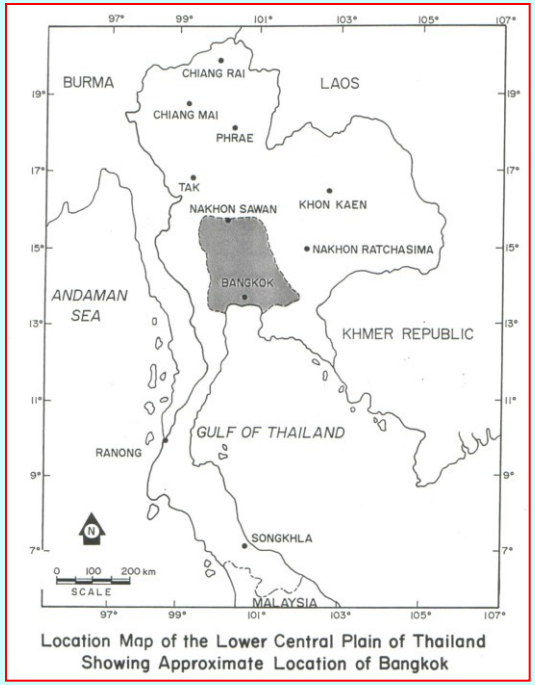




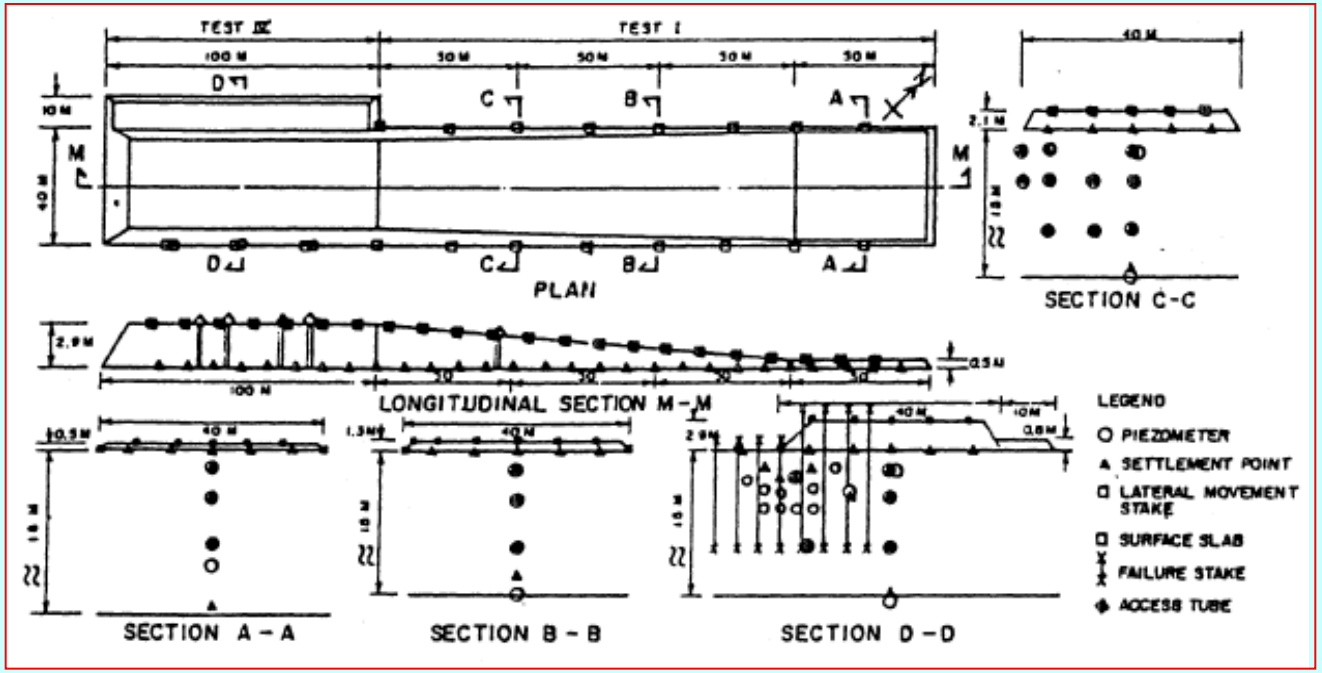


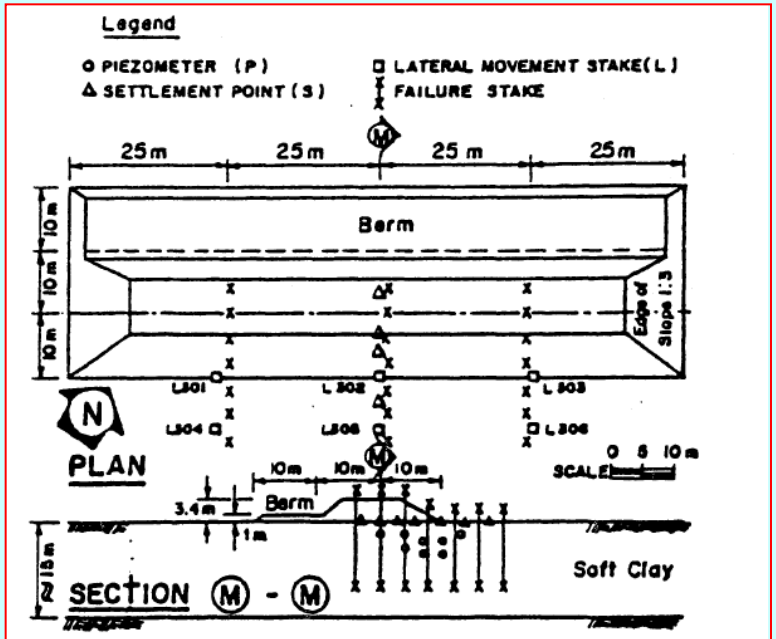


Typical Vacuum Consolidation System



Location Map of the Lower Central Plain of Thailand Showing Approximate Location of Bangkok





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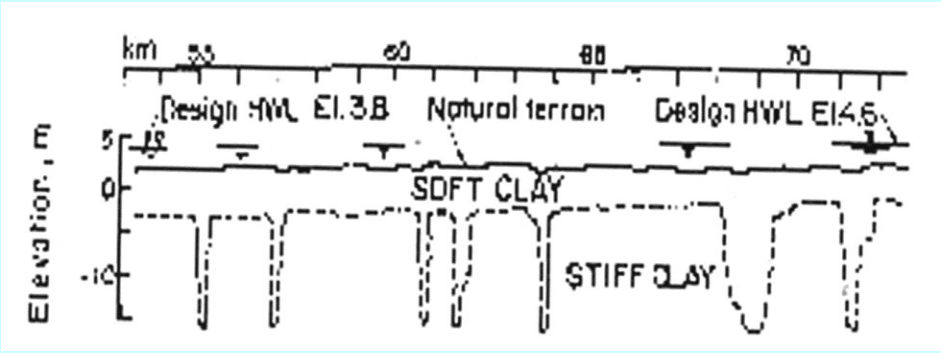
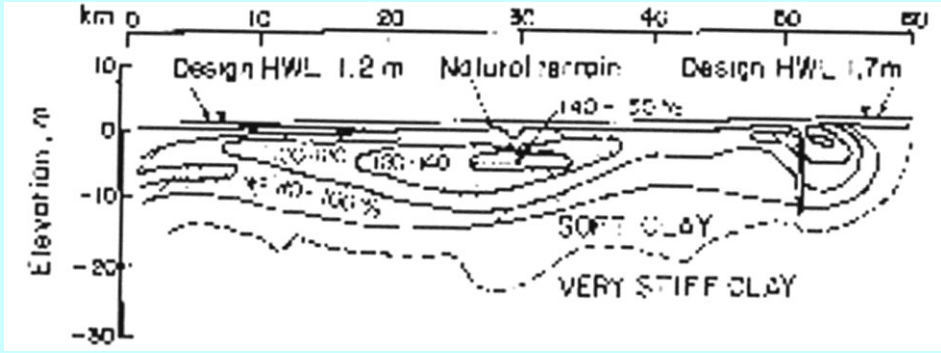
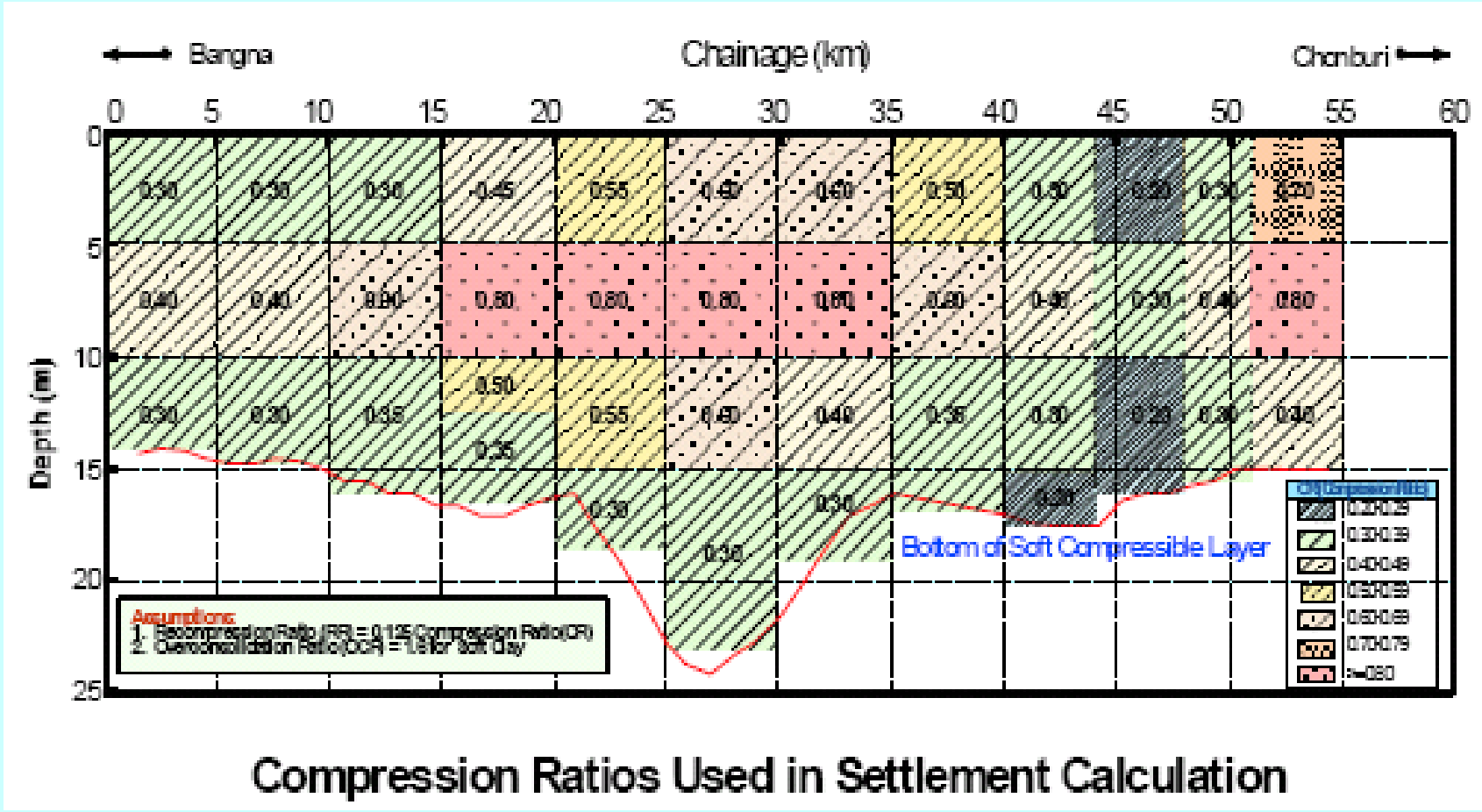
1. **The need to characterize the various soil layers and to identify the possible presence or absence of deep gullies of soft clays would have resulted in having a large number of boreholes and insitu tests.**

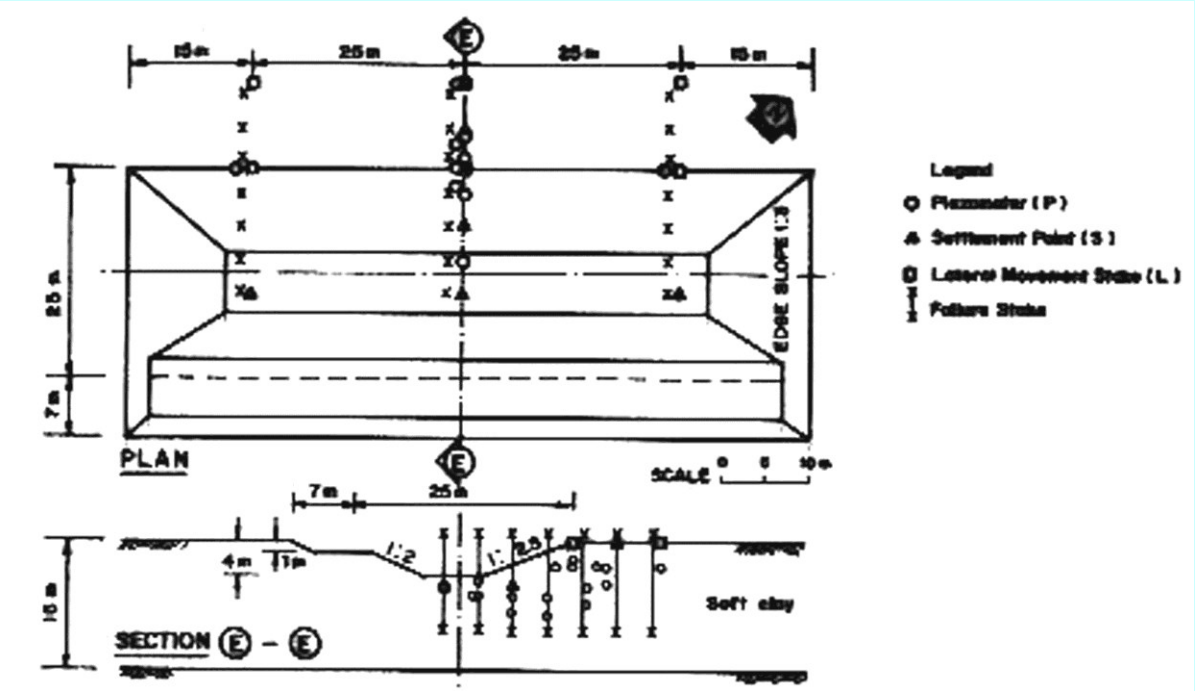
2. **The test embankments and the excavation were carried out to monitor the field settlements under various magnitude of surcharge and to ensure the stability of the embankment and the excavation on a full scale basis and to compare the values estimated from single element laboratory tests and small scale in-situ tests**

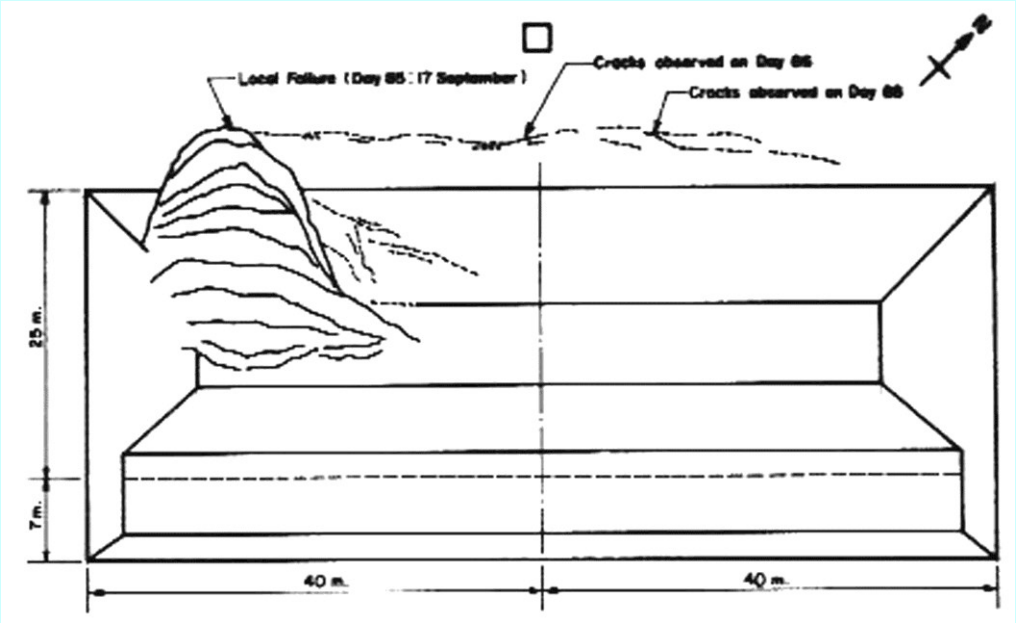


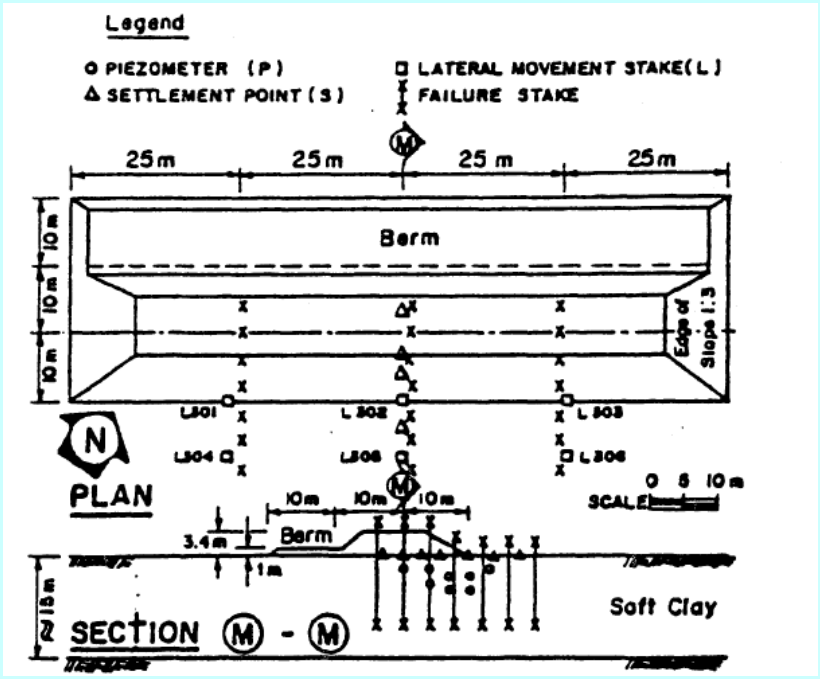


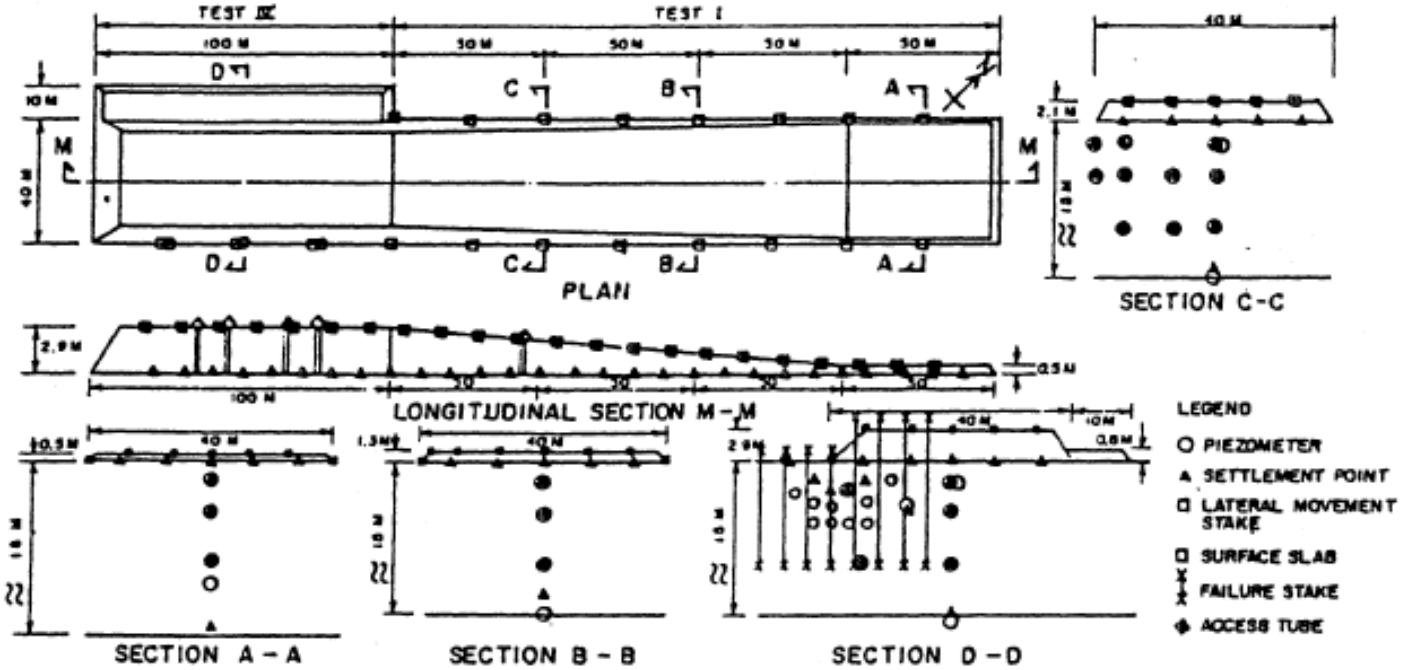


Compression Ratios Used in Settlement Calculation

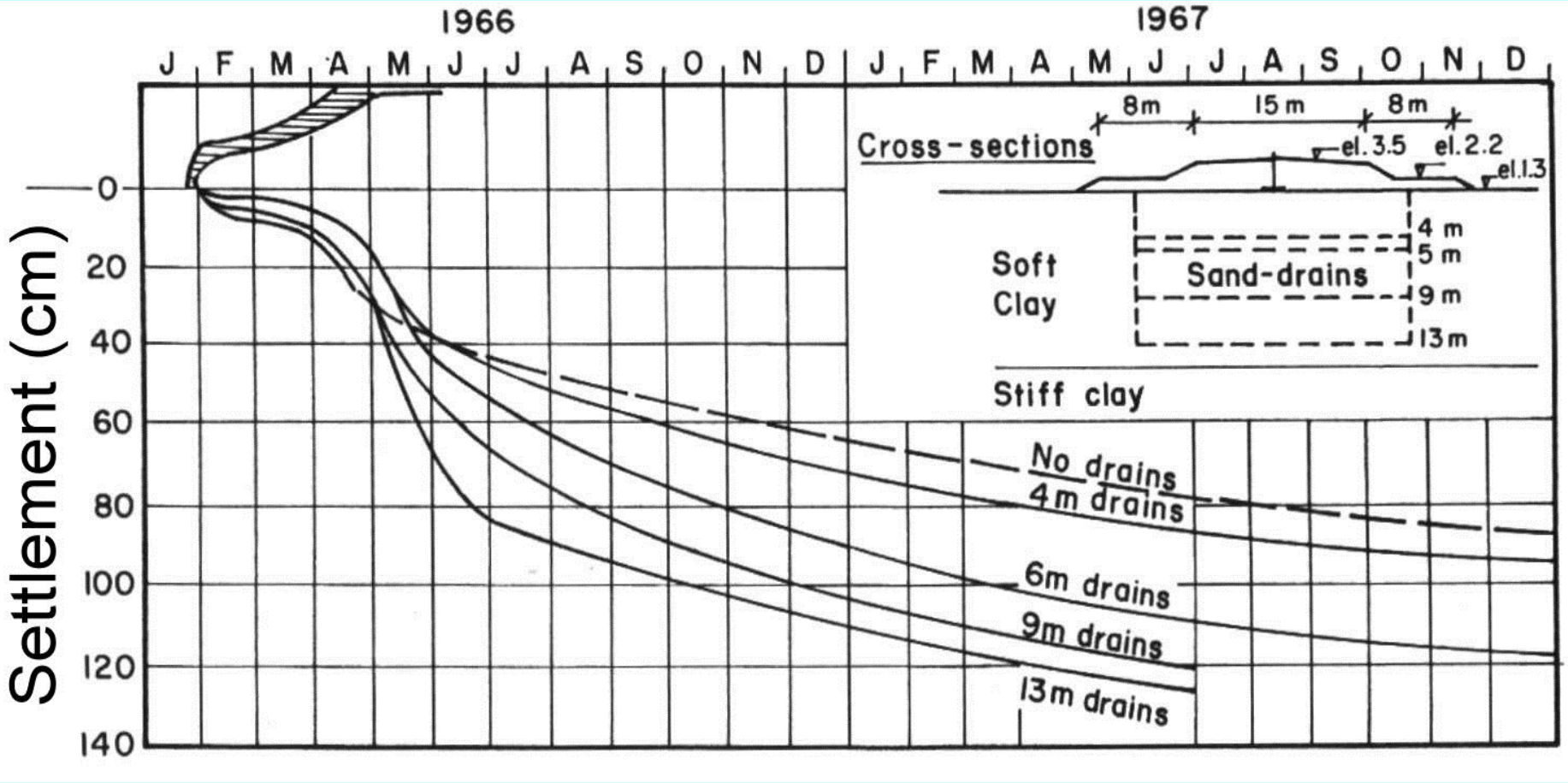

15m
25m
25m
15m
E
25 m
7m
EDGE SLOPE 1:3
PLAN
SCALE 0 5 10 m
Legend
Piezometer (P)
Settlement Point (S)
Lateral Movement Stake (L)
Failure Stake
7m
25m
4m
1m
1:2
1:2.5
16 m
Soft clay
SECTION E – E

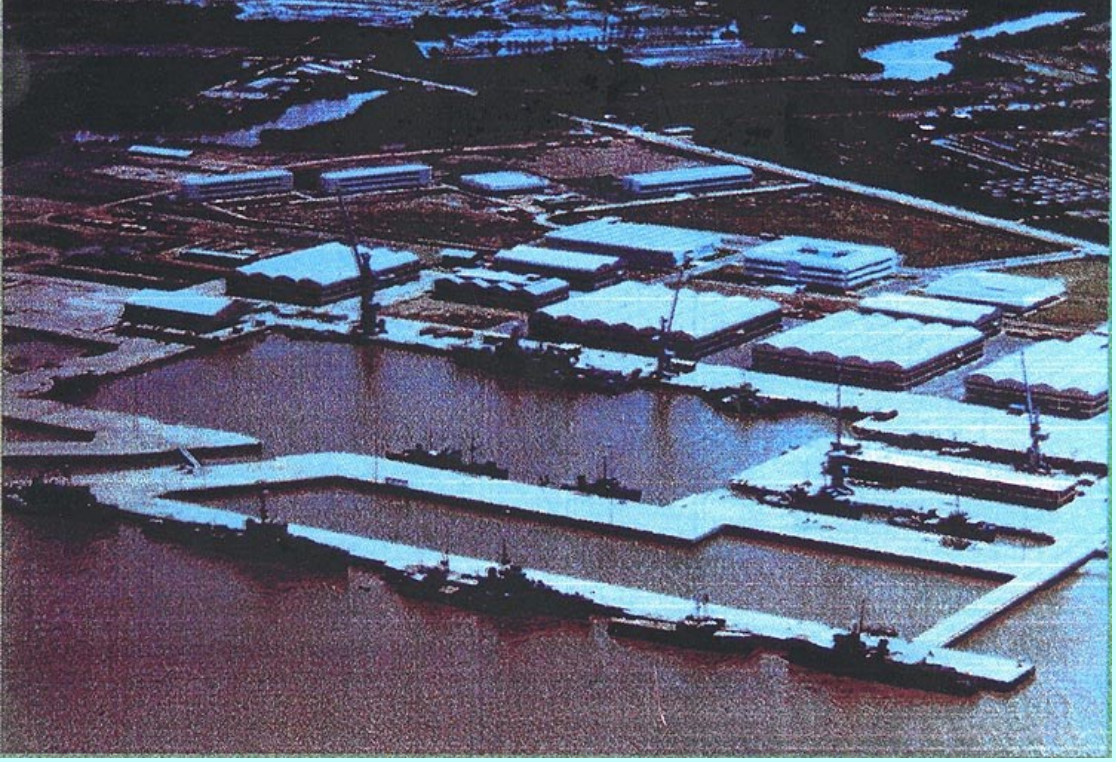

Local Failure (Day 65 : 17 September)
Cracks observed on Day 66
Cracks observed on Day 68
25 m.
7 m.
40 m.
40 m.

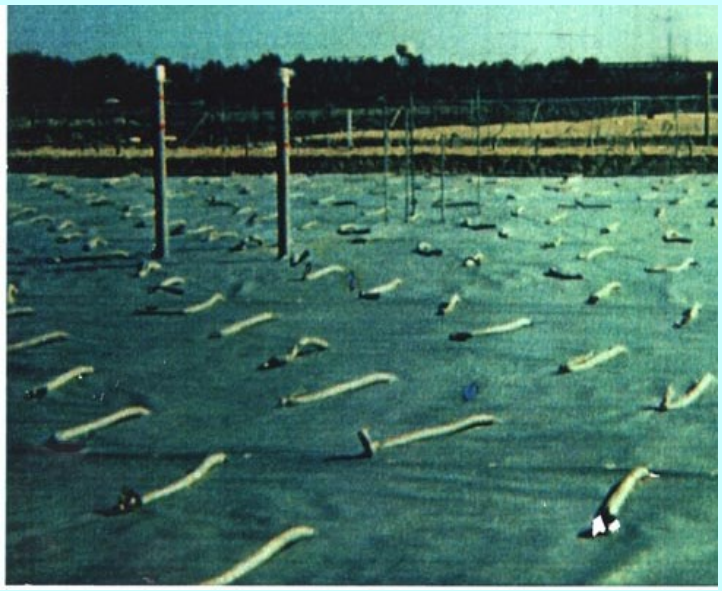

Legend
PIEZOMETER (P)
SETTLEMENT POINT (S)
LATERAL MOVEMENT STAKE(L)
FAILURE STAKE
25m
25m
25 m
25m
Berm
Edge of Slope 1:3
L301
L 302
L 303
L304
L305
L 306
PLAN
SCALE 0 5 10 m
10m
10m
10m
3.4m
Berm
1m
≈18m
Soft Clay
SECTION (M) - (M)


TEST II
TEST I
100 M
50 M
50 M
50 M
50 M
D
C
B
A
10 M
40 M
PLAN
40 M
2.1 M
18 M
SECTION C-C
2.9 M
0.5 M
100 M
LONGITUDINAL SECTION M-M
40 M
SECTION A-A
SECTION B-B
SECTION D-D
LEGEND
PIEZOMETER
SETTLEMENT POINT
LATERAL MOVEMENT STAKE
SURFACE SLAB
FAILURE STAKE
ACCESS TUBE

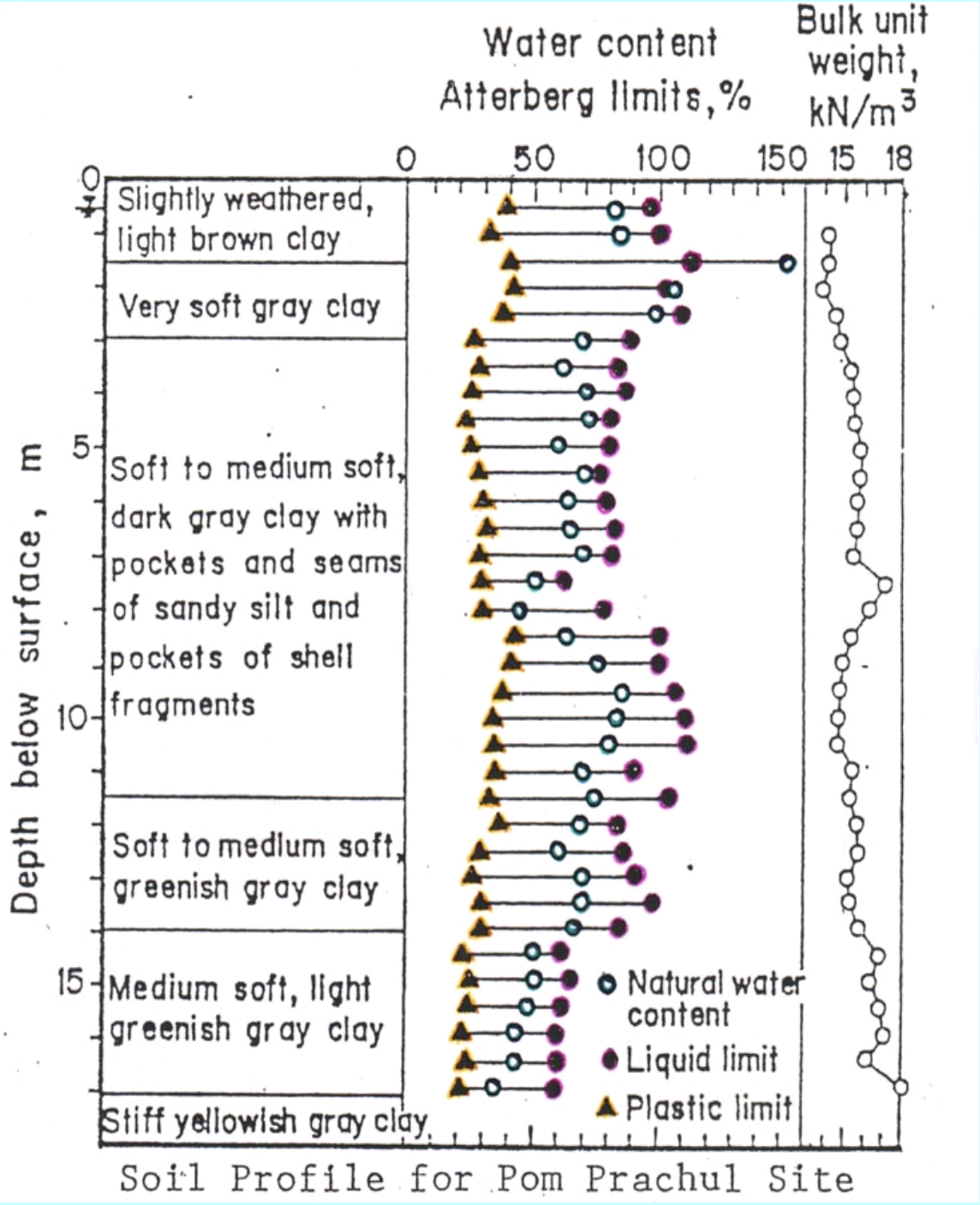
Sand Drain


1966
1967
J F M A M J J A S O N D J F M A M J J A S O N D
Settlement (cm)
0
20
40
60
80
100
120
140
Cross-sections
8m
15 m
8m
el. 3.5
el. 2.2
el. 1.3
Soft Clay
Sand-drains
4 m
5 m
9 m
13m
Stiff clay
No drains
4m drains
6m drains
9m drains
13m drains

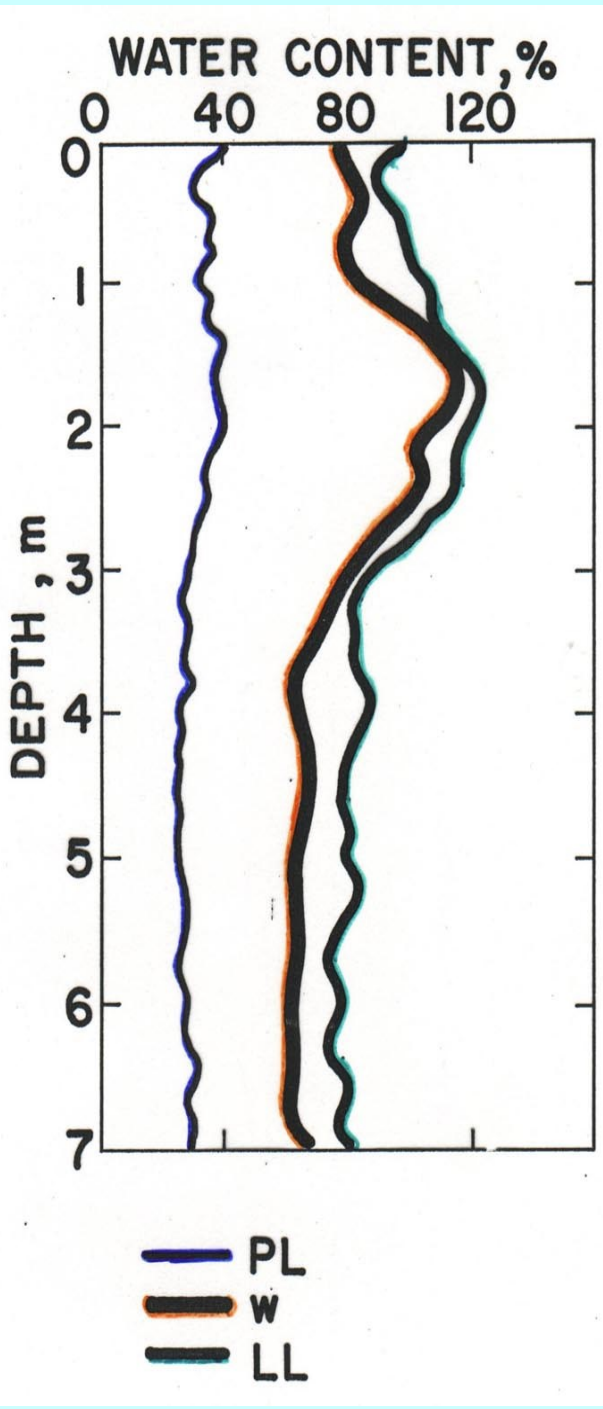


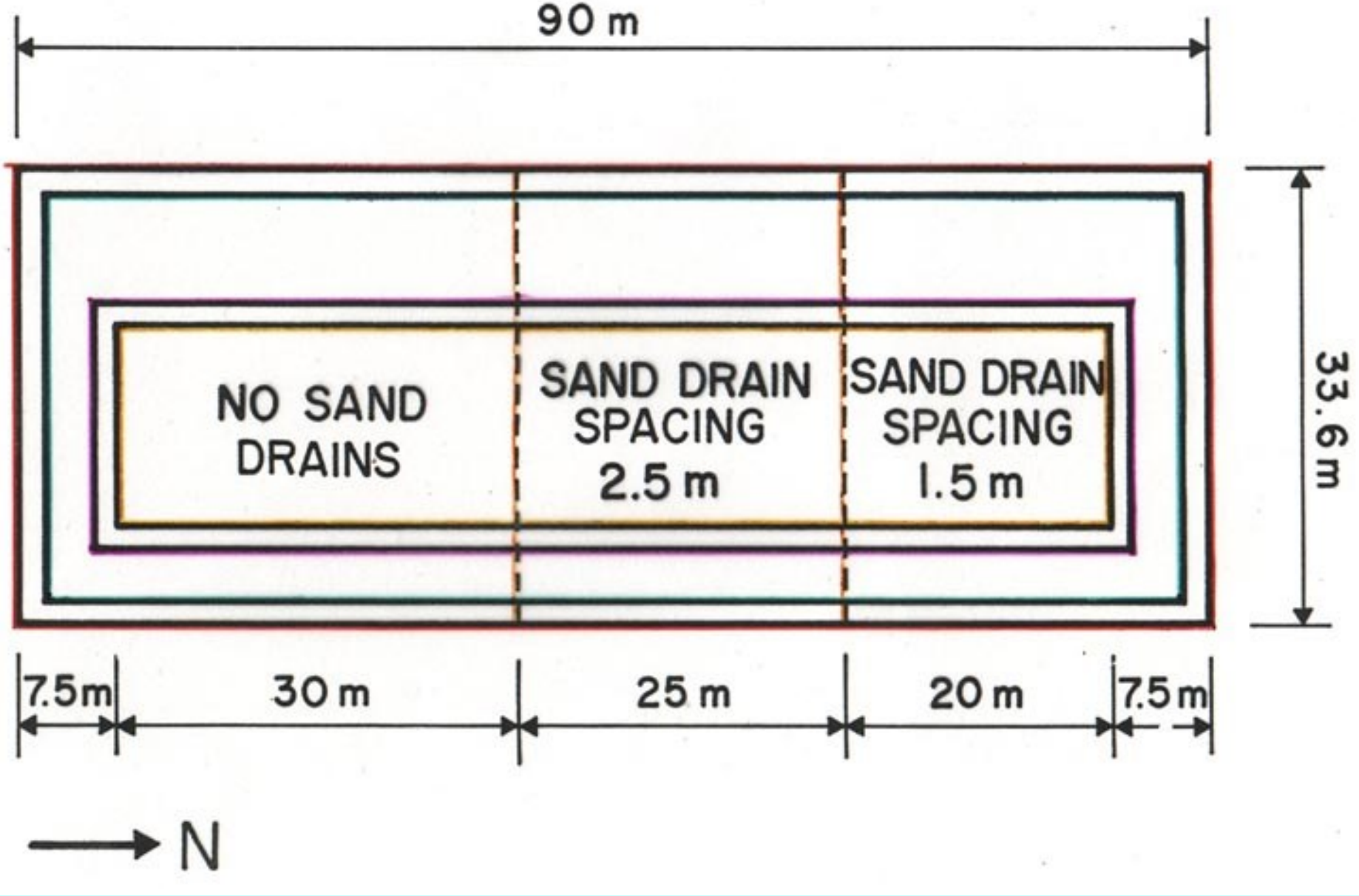


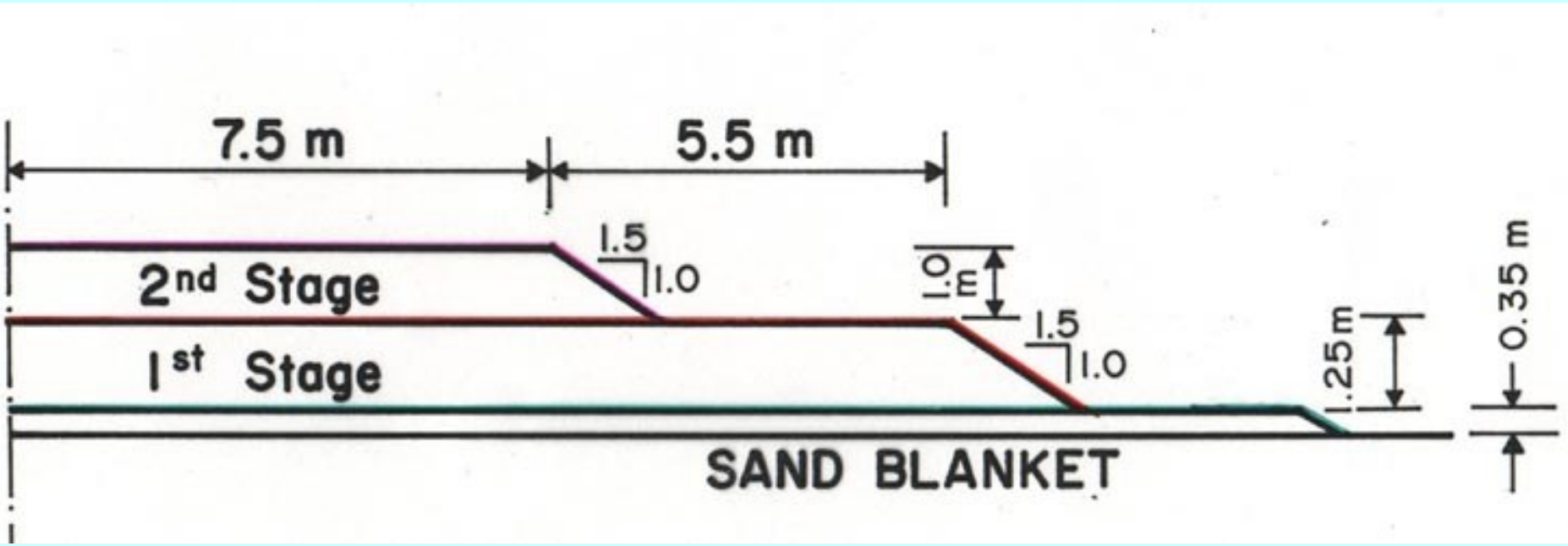
Royal Thai Navy Dockyard

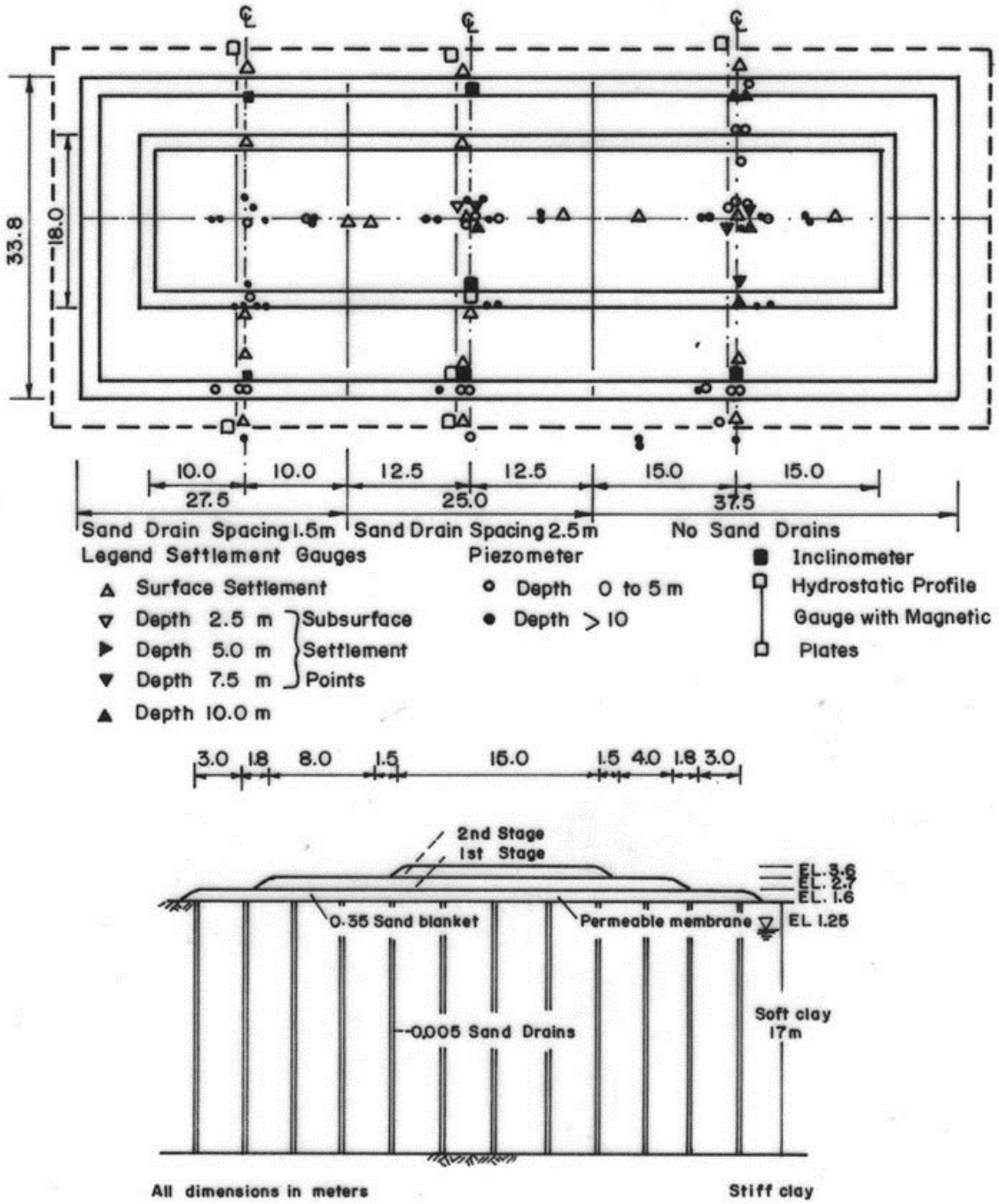


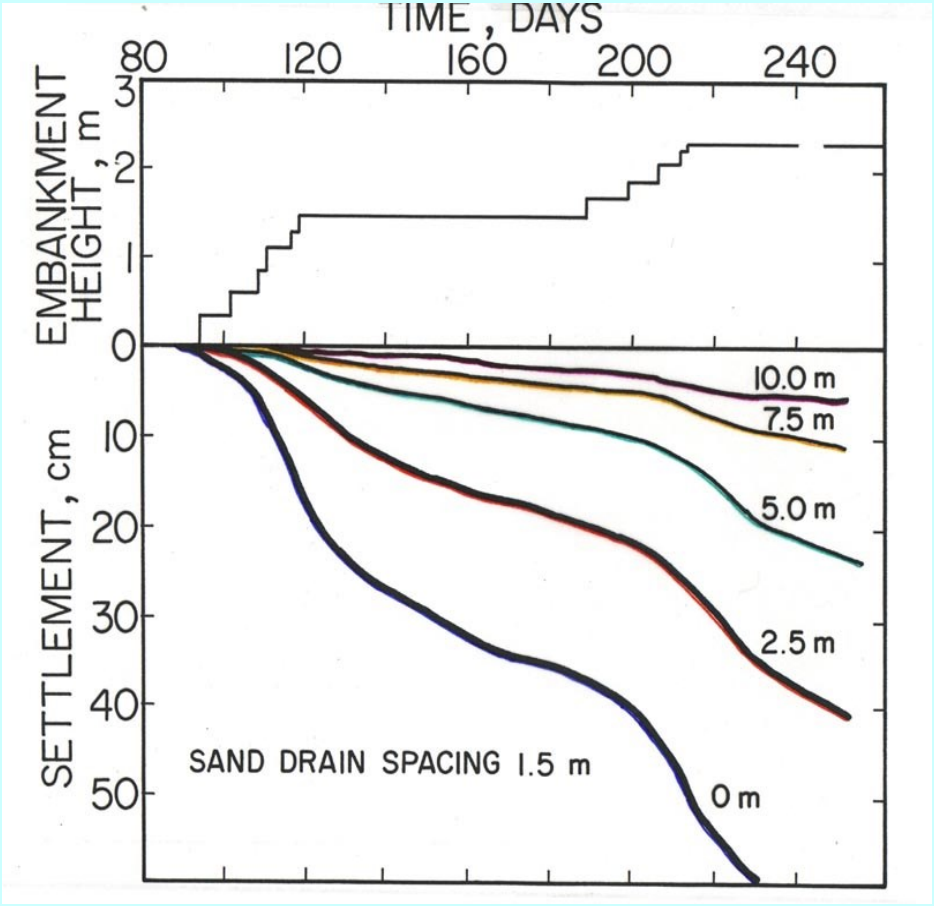
Soil Profile for Pom Prachul Site

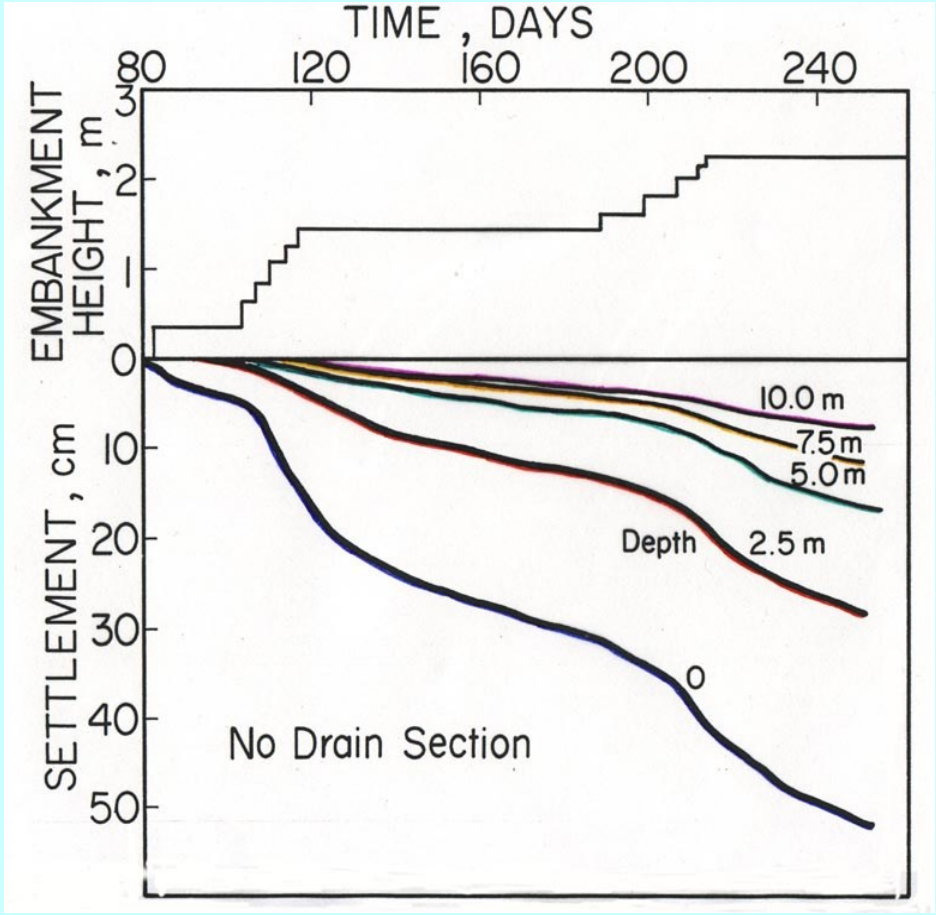


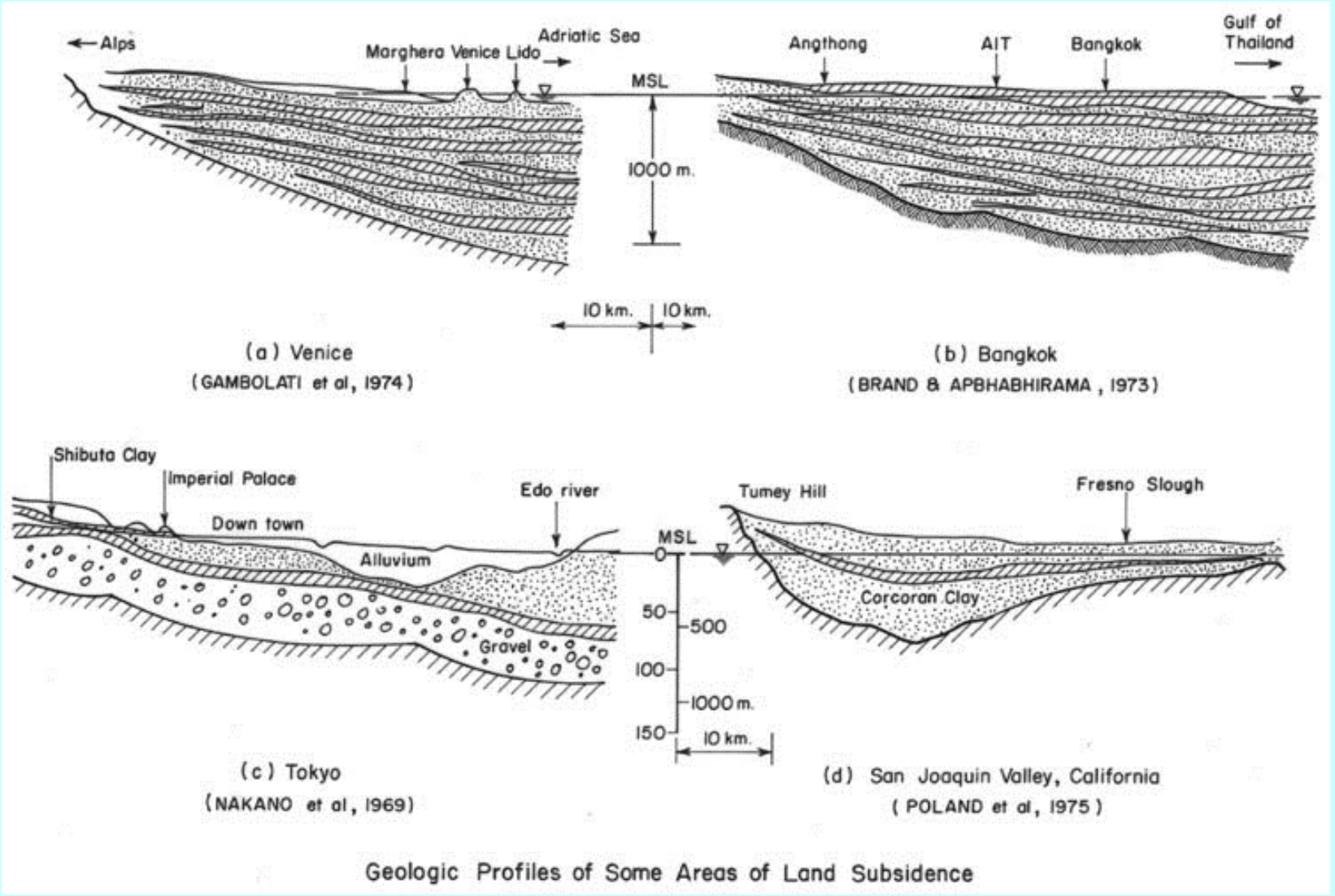

90 m
NO SAND DRAINS
SAND DRAIN SPACING 2.5 m
SAND DRAIN SPACING 1.5 m
33.6 m
7.5m
30 m
25 m
20 m
7.5m
N


7.5 m
5.5 m
1.5
1.0
2nd Stage
1.0 m
1.5
1.0
1st Stage
1.25 m
0.35 m
SAND BLANKET

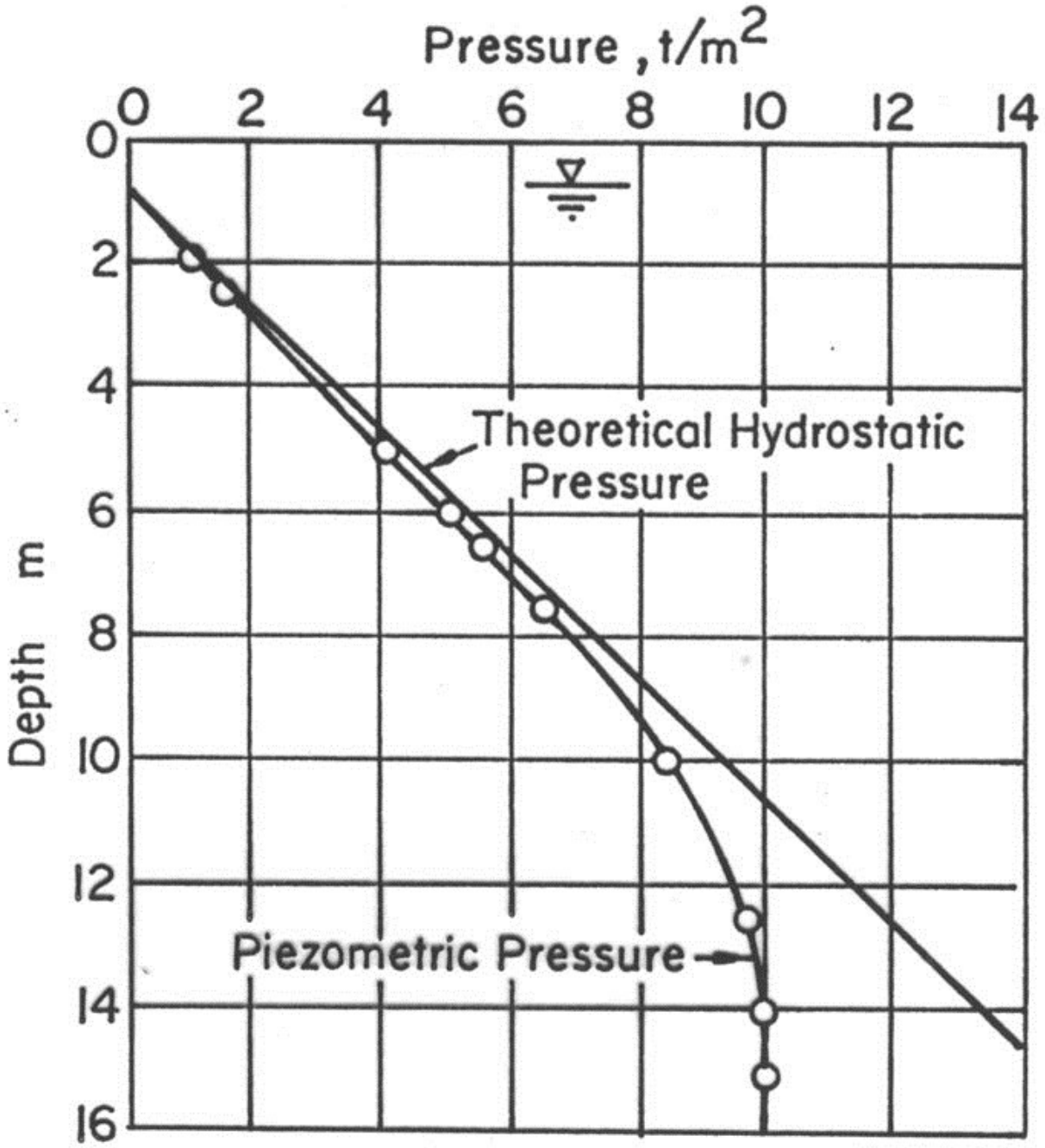

33.8
18.0
10.0
10.0
12.5
12.5
15.0
15.0
27.5
25.0
37.5
Sand Drain Spacing 1.5m
Sand Drain Spacing 2.5m
No Sand Drains
Legend Settlement Gauges
Surface Settlement
Depth 2.5 m
Depth 5.0 m
Depth 7.5 m
Subsurface Settlement Points
Depth 10.0 m
Piezometer
Depth 0 to 5 m
Depth > 10
Inclinometer
Hydrostatic Profile
Gauge with Magnetic Plates
3.0
1.8
8.0
1.5
15.0
1.5
4.0
1.8
3.0
2nd Stage
1st Stage
EL. 3.6
EL. 2.7
EL. 1.6
0.35 Sand blanket
Permeable membrane
EL. 1.25
0.005 Sand Drains
Soft clay
17m
All dimensions in meters
Stiff clay

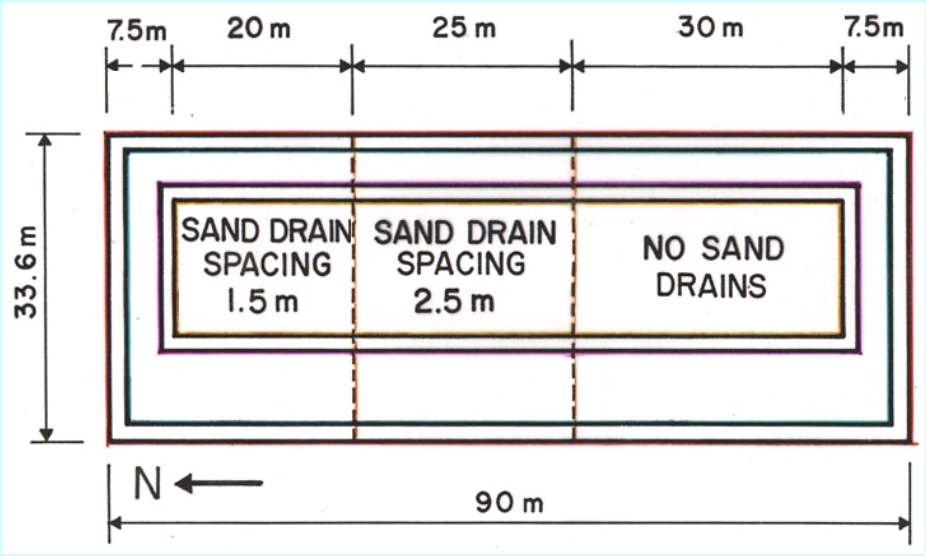

TIME , DAYS
80
120
160
200
240
EMBANKMENT HEIGHT, m
0
1
2
3
SETTLEMENT, cm
10
20
30
40
50
10.0 m
7.5 m
5.0 m
2.5 m
0 m
SAND DRAIN SPACING 1.5 m

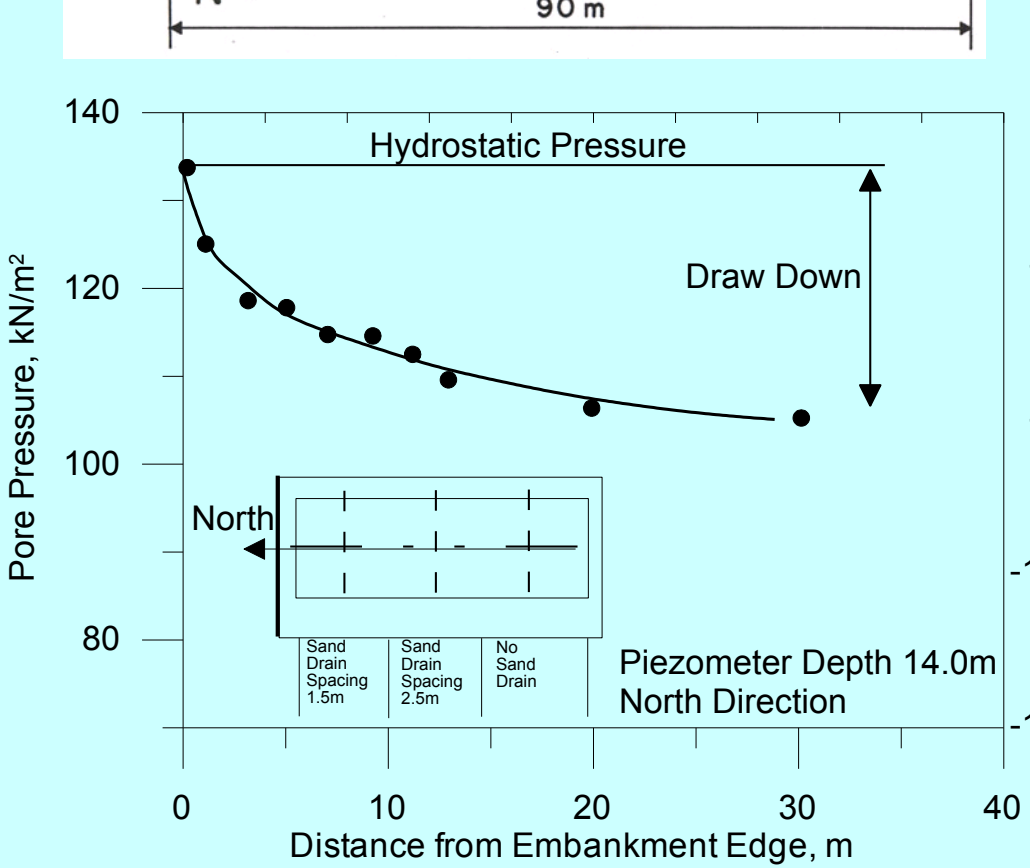

TIME , DAYS
80
120
160
200
240
EMBANKMENT HEIGHT, m
0
1
2
3
SETTLEMENT, cm
10
20
30
40
50
10.0 m
7.5 m
5.0 m
Depth
2.5 m
0
No Drain Section

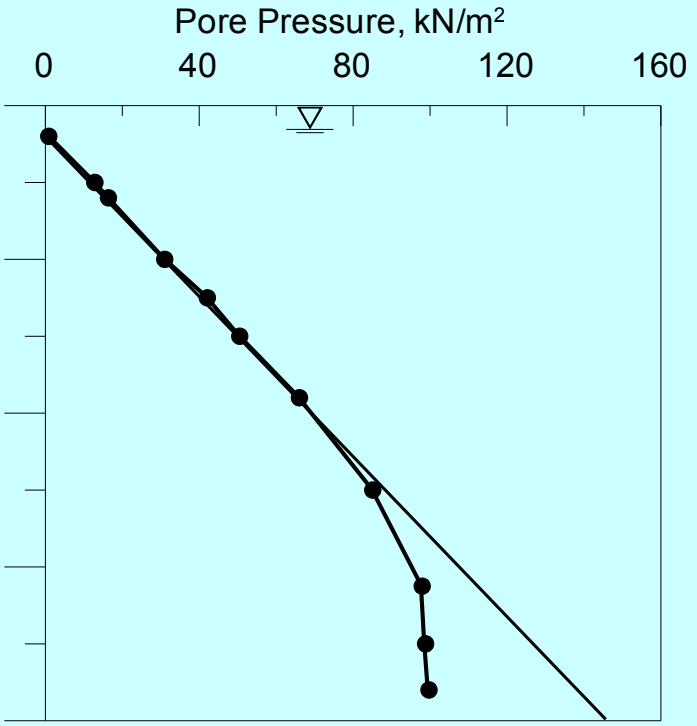


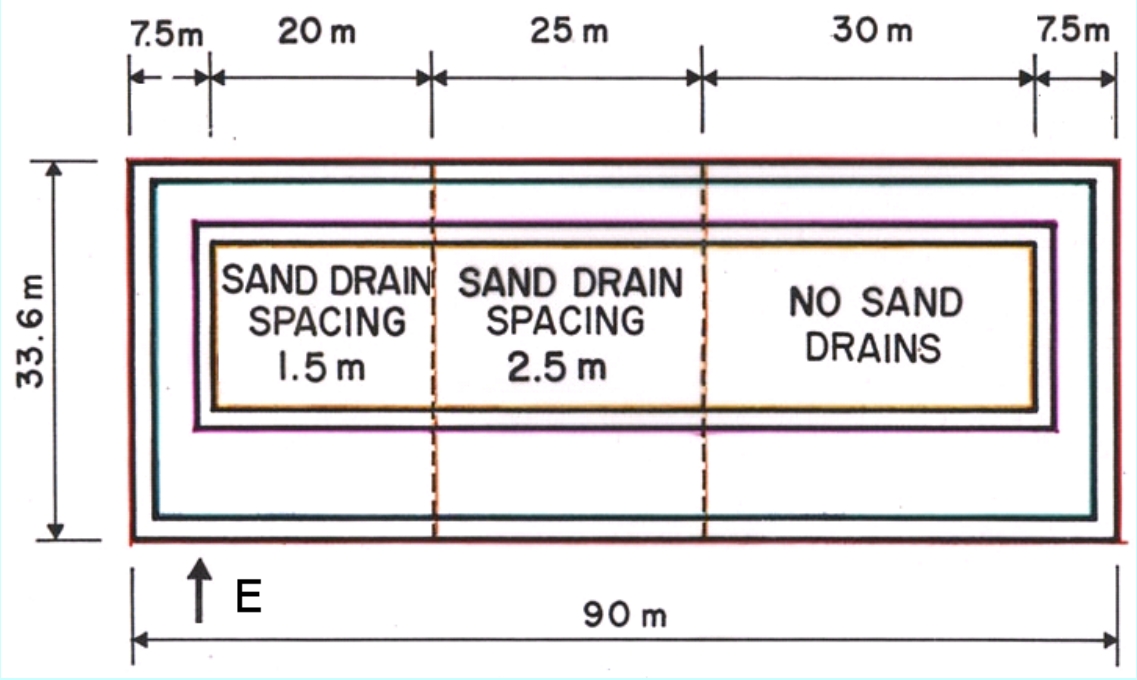
Geologic Profiles of Some Areas of Land Subsidence

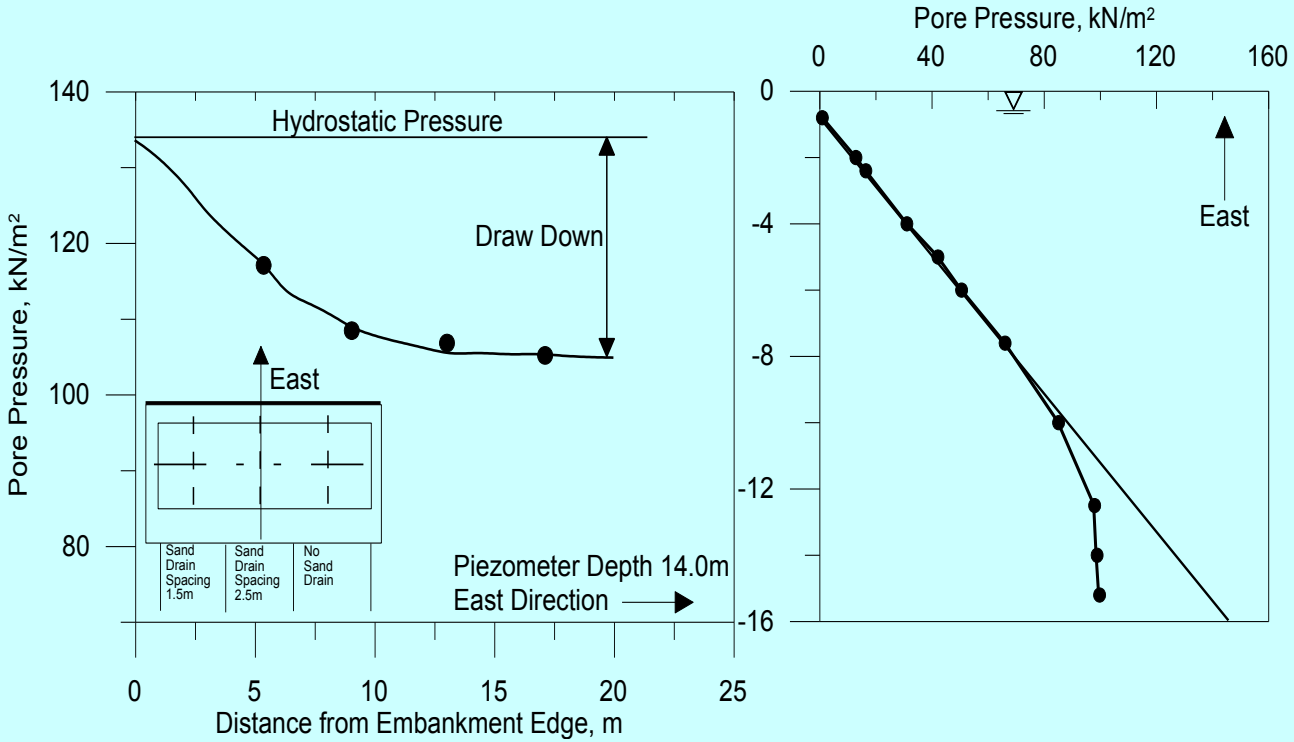

Pressure , t/m2
0
2
4
6
8
10
12
14
Depth m
0
2
4
6
8
10
12
14
16
Theoretical Hydrostatic Pressure
Piezometric Pressure

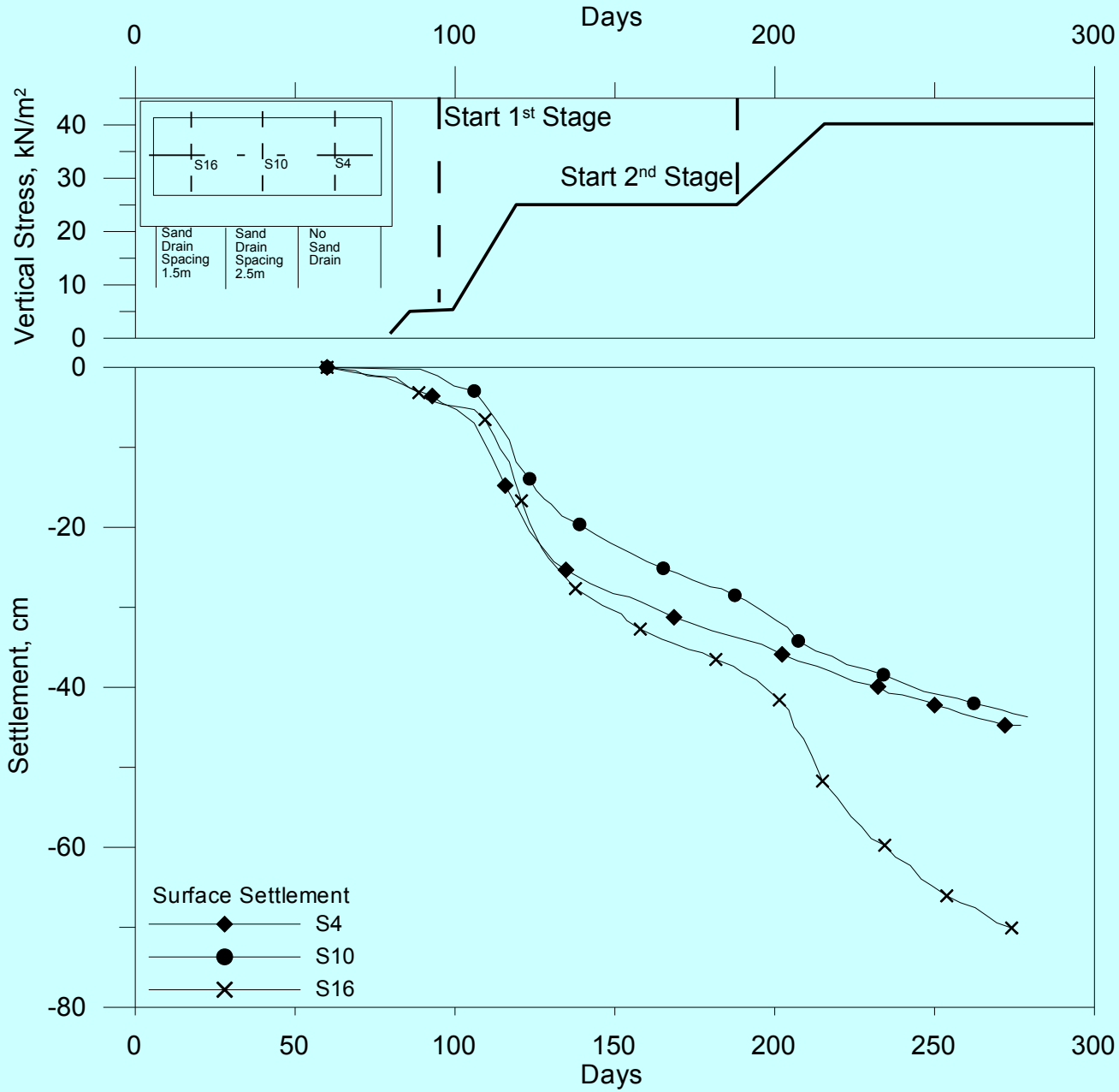

7.5m
20 m
25 m
30 m
7.5m
33.6 m
SAND DRAIN SPACING 1.5 m
SAND DRAIN SPACING 2.5 m
NO SAND DRAINS
N
90 m


Hydrostatic Pressure
Draw Down
North
Sand Drain Spacing 1.5m
Sand Drain Spacing 2.5m
No Sand Drain
Piezometer Depth 14.0m
North Direction
Pore Pressure, kN/m²
Distance from Embankment Edge, m
140
120
100
80
0
10
20
30
40


Pore Pressure, kN/m²
0
40
80
120
160
0
-4
-8
-12
-16


7.5m
20 m
25 m
30 m
7.5m
33.6 m
SAND DRAIN SPACING 1.5 m
SAND DRAIN SPACING 2.5 m
NO SAND DRAINS
E
90 m


Pore Pressure, kN/m²
140
120
100
80
Hydrostatic Pressure
Draw Down
East
Sand Drain Spacing 1.5m
Sand Drain Spacing 2.5m
No Sand Drain
Piezometer Depth 14.0m
East Direction
0
5
10
15
20
25
Distance from Embankment Edge, m
Pore Pressure, kN/m²
0
40
80
120
160
0
-4
-8
-12
-16
East


Days
0
100
200
300
Vertical Stress, kN/m²
40
30
20
10
0
Start 1st Stage
Start 2nd Stage
S16
S10
S4
Sand Drain Spacing 1.5m
Sand Drain Spacing 2.5m
No Sand Drain
Settlement, cm
0
-20
-40
-60
-80
Surface Settlement
S4
S10
S16
0
50
100
150
200
250
300
Days

1. Gold Coast Highway
   (stone columns)
2. Sunshine Coast Motorway
   (Vertical drains-PVD)
3. Port of Brisbane Motorway
   (Vertical drains-PVD)

Surprises and ill-conceived ideas

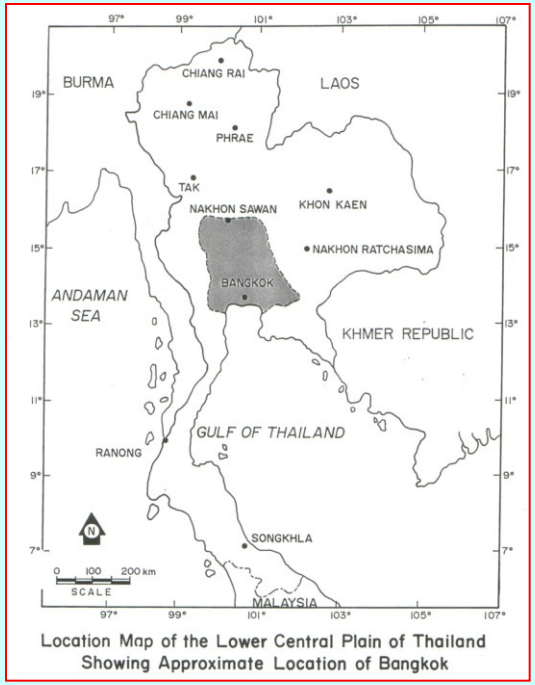
1. **Insufficient surcharge**

2. **No-drain and drained sections in one embankment**

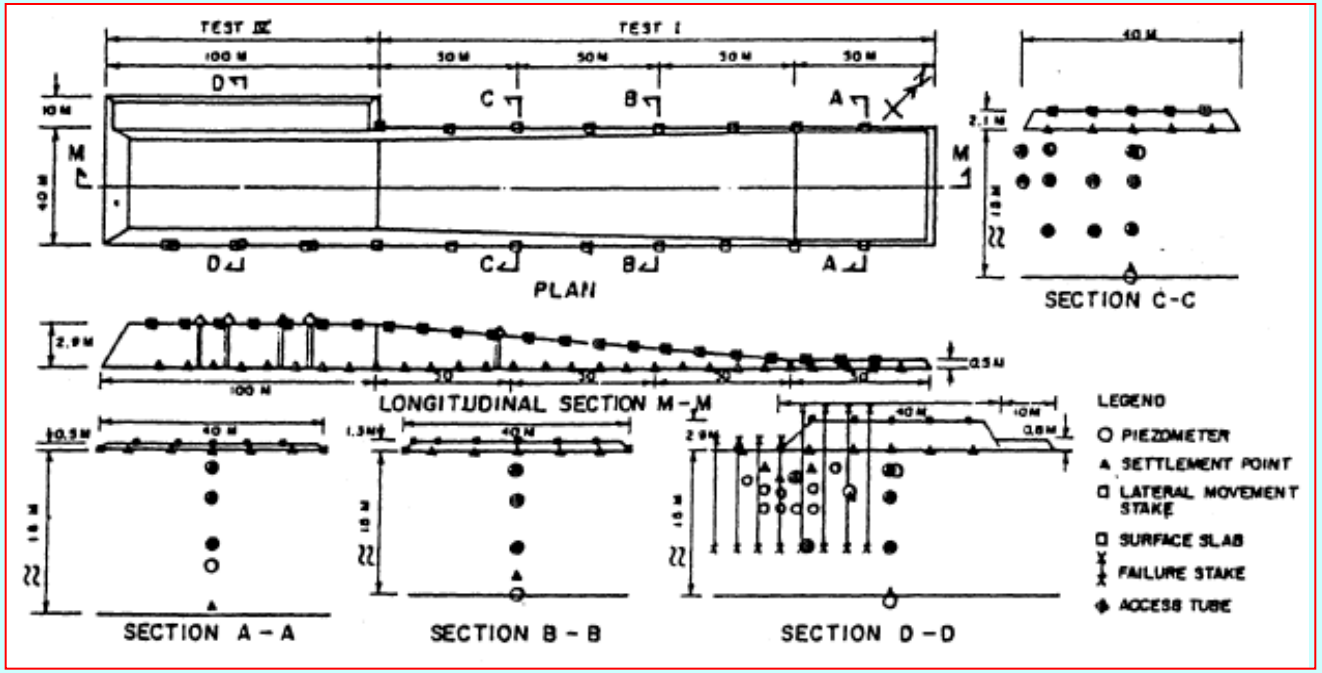
3. **Recharge and possible hydraulic connections**

Geotechnical Investigation at Nong Ngo Hao Airport Site

Phase	Year	Title
I	1972 - 1974	Geotechnical Investiga-tions by Asian Institute of Technology and N.D. Lea and Associates, Kampsax
II	1983 - 1984	Pre-loading with Sand Drains, and , Vacuum-Drains; Moh and Associates and NACO
III	1992	An Independent Soil Engineering Study; Norwegian Geotechnical Institute in cooperation the STS Engineering Consultant Co. Ltd.
IV	1993 - 1995	Full scale Field test of Prefabricated vertical drains by the Asian Institute of Technology



Location Map of the Lower Central Plain of Thailand Showing Approximate Location of Bangkok



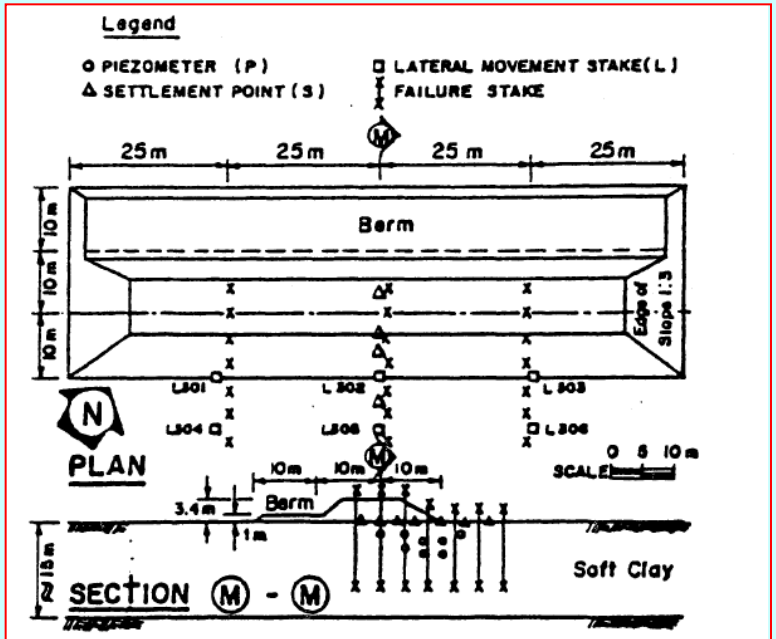
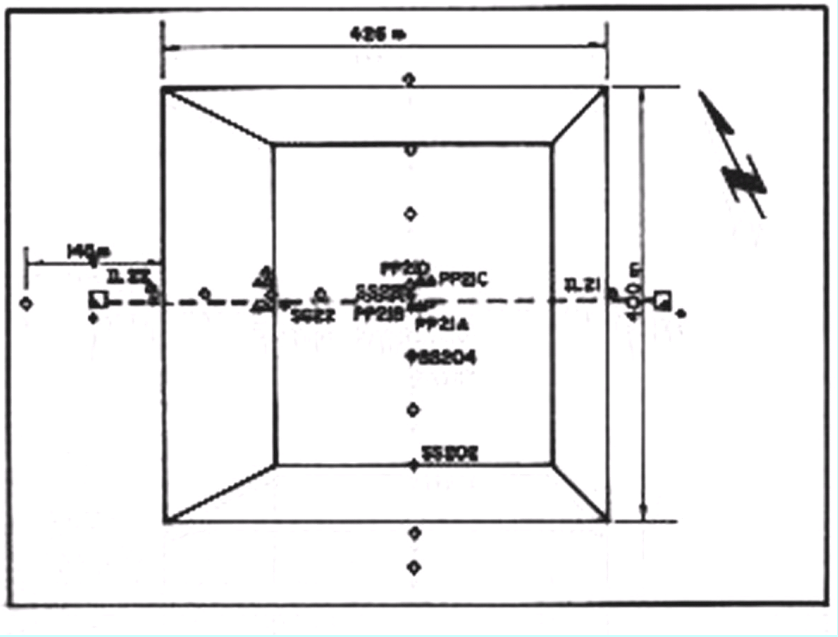
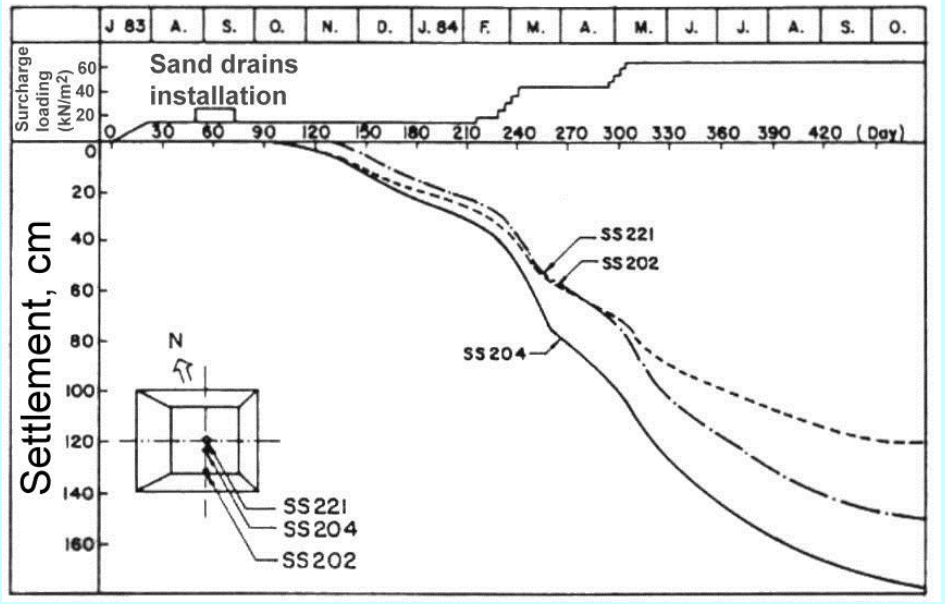
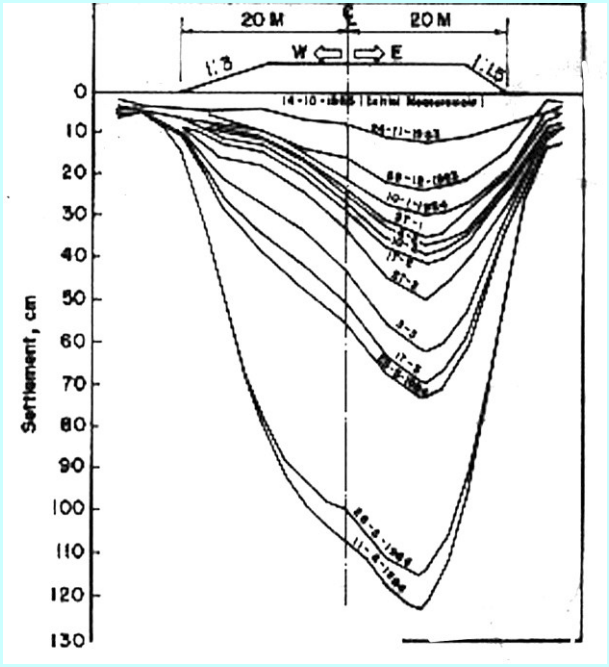
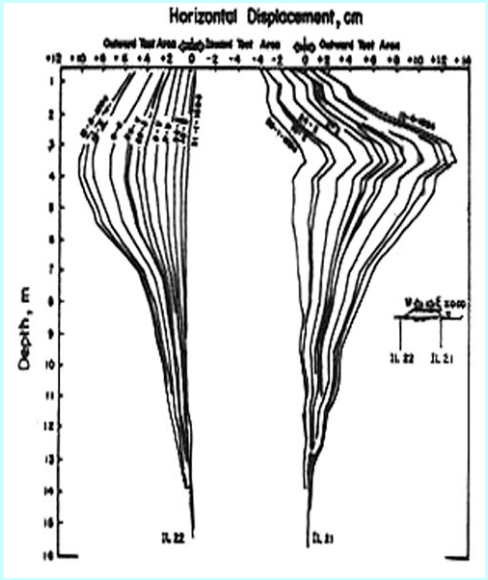


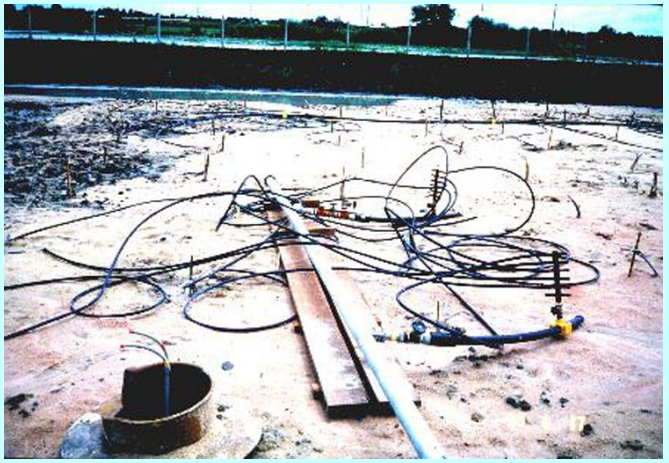
Fig. 1. Embankment I Taken to Failure.


425 m
145 m
100 m
PP22D
PP22C
PP22B
PP21A
SS204
SS205
3622

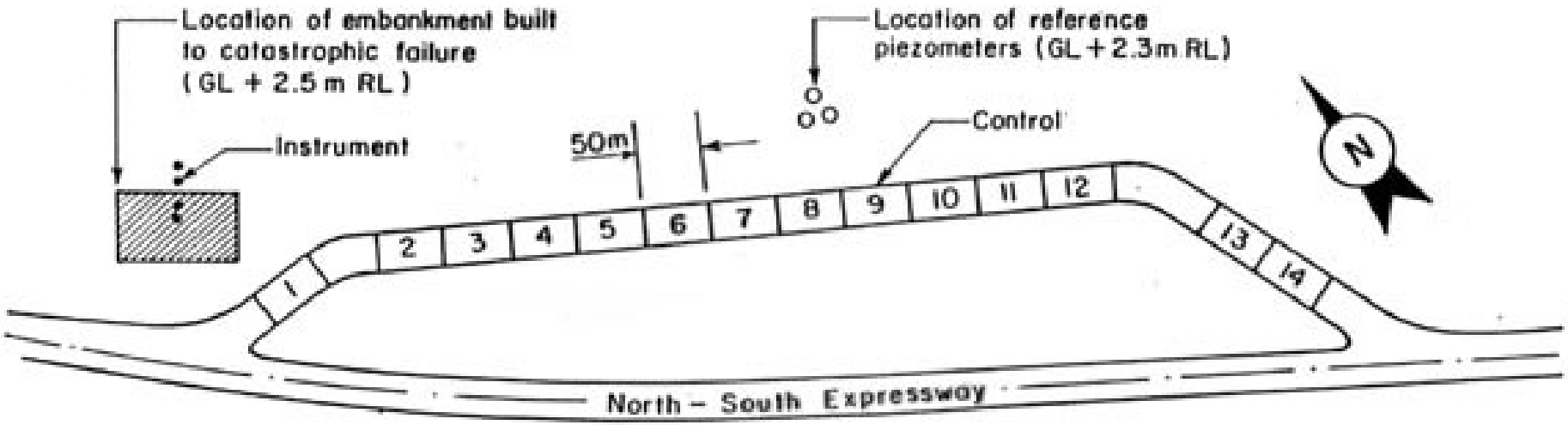

J 83
A.
S.
O.
N.
D.
J. 84
F.
M.
A.
M.
J.
J.
A.
S.
O.
Surcharge loading (kN/m²)
Sand drains installation
0 30 60 90 120 150 180 210 240 270 300 330 360 390 420 (Day)
Settlement, cm
SS221
SS202
SS204
N
SS221
SS204
SS202

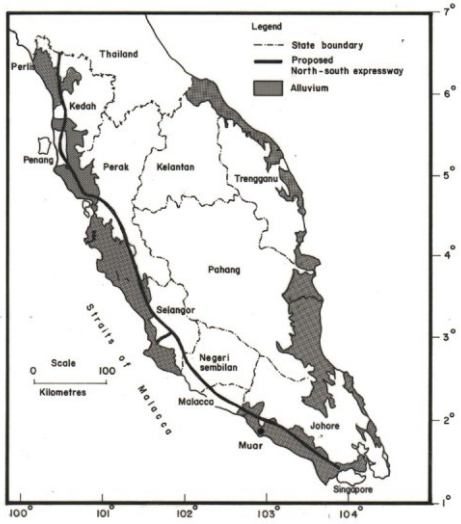

20 M
20 M
W
E
1:3
1:1.5
Settlement, cm


Horizontal Displacement, cm
Depth, m



LEMBAGA LEBUHRAYA MALAYSIA
TRIAL EMBANKMENTS





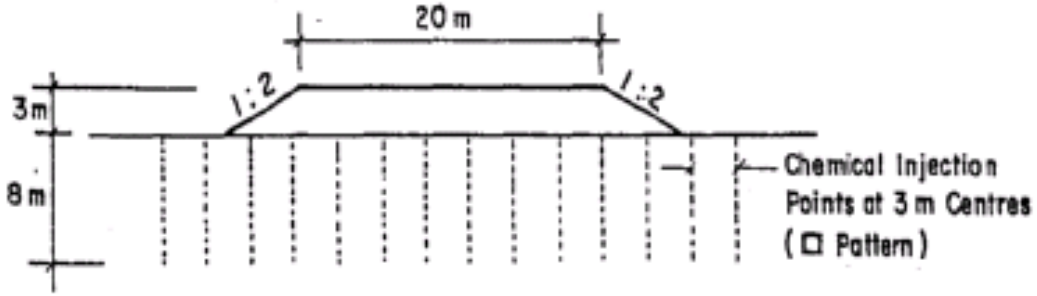
River and Coastal Alluvium of Peninsular Malaysia

Method of Ground Improvement :

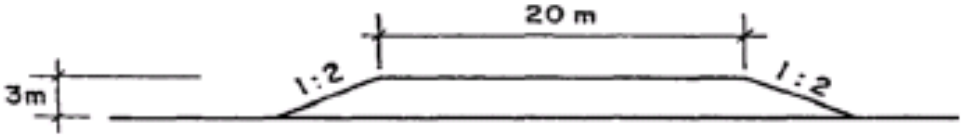
- Electro - osmosis (6)
- Chemical Injection (1 & 4)
- Sand Sandwich (13)
- Preloading & Drains (11, 12 & 14)
- Micro Piles (3)
- Vacuum Preloading (10)
- Sand Compaction Piles (8)
- Well-point Preloading (5)

Prestressed Spun Piles (7)

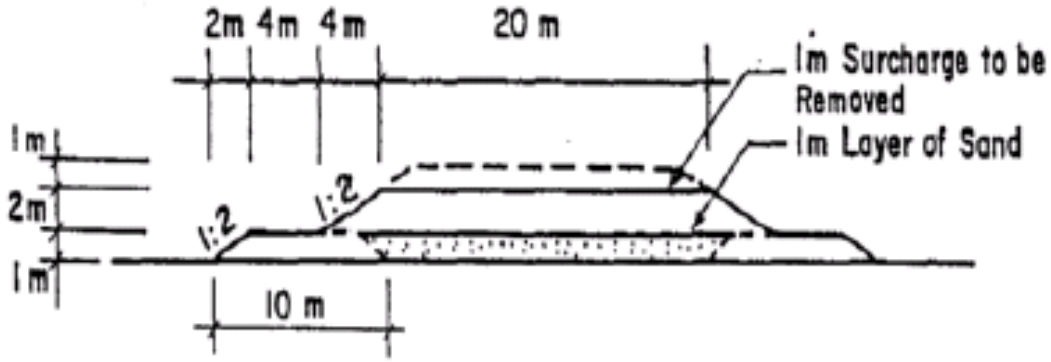
Layout of Trial Embankments



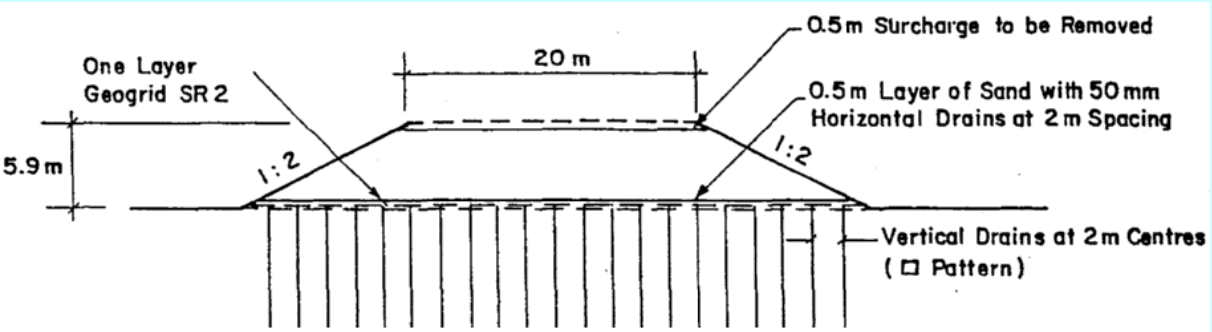
Scheme 3/1



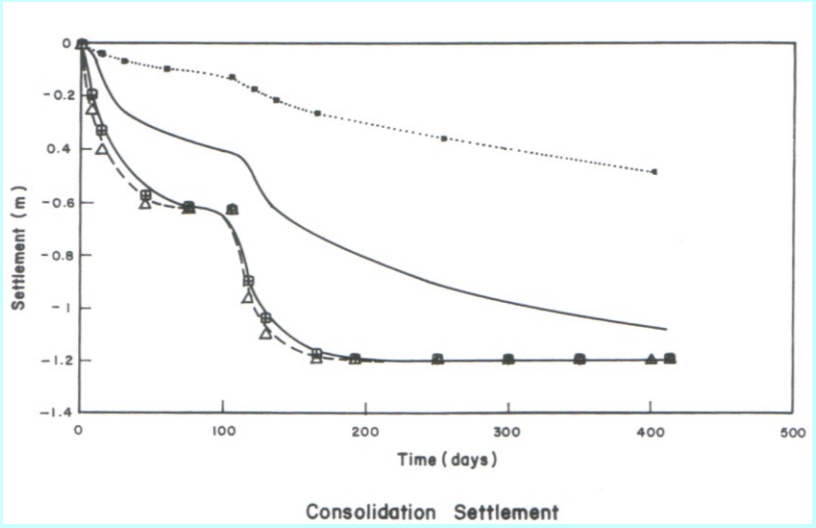
Scheme 3/2



Scheme 3/3

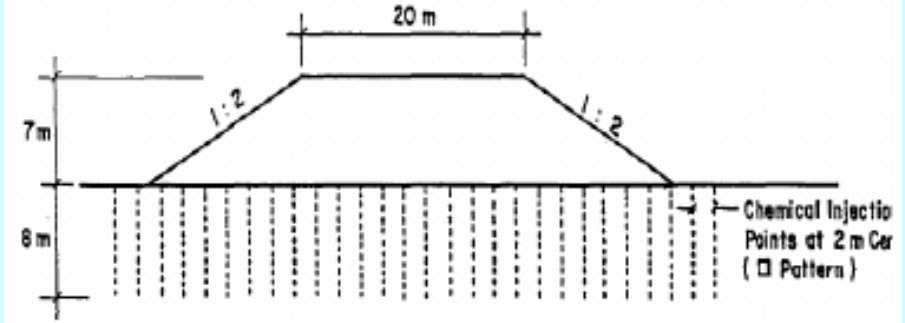


Scheme 3/4

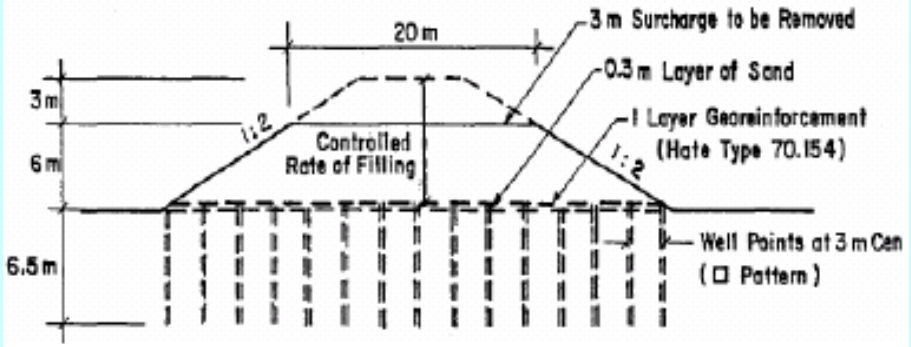


Consolidation Settlement

SCHEME	GROUND IMPROVEMENT
3/1	Electro Chemical Injection
3/2	Control
3/3	Sand Sandwick Method
3/4	Preloading, Geogrid & Prefabricated Vertical Drain

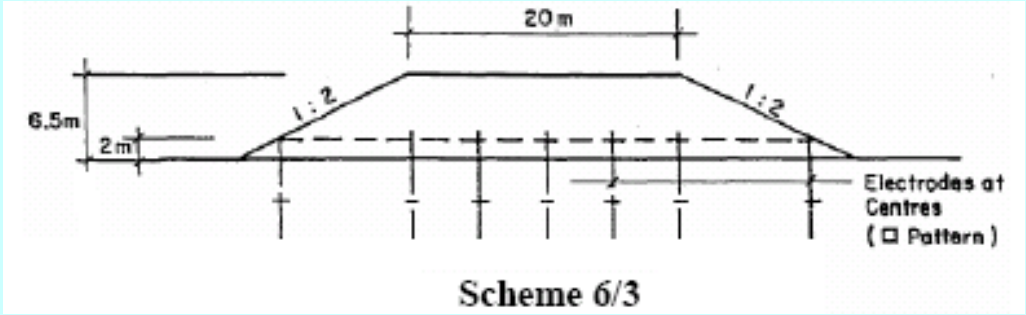


Scheme 6/1

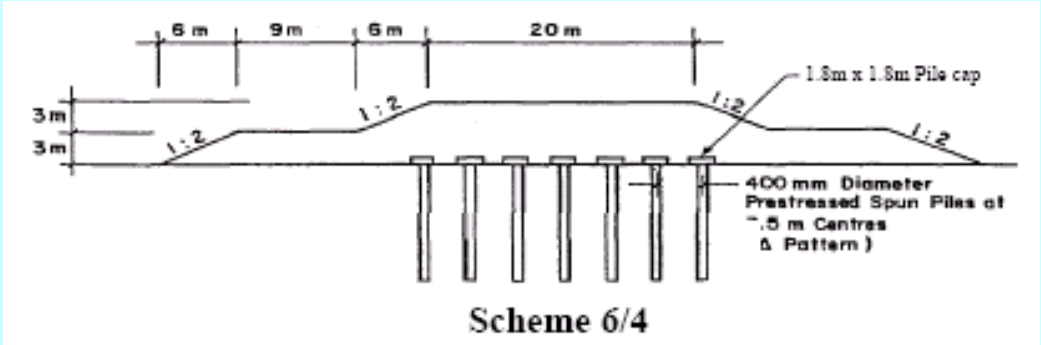


Scheme 6/2

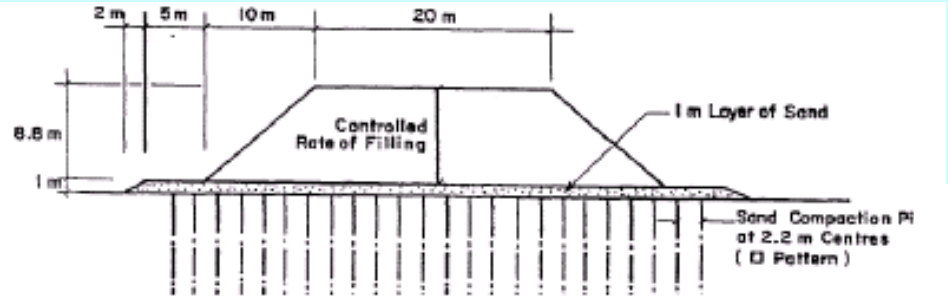
SCHEME	GROUND IMPROVEMENT
6/1	Electro Chemical Injection
6/2	Well Point Preloading
6/3	Electro Osmosis
6/4	Prestressed Spun Piles



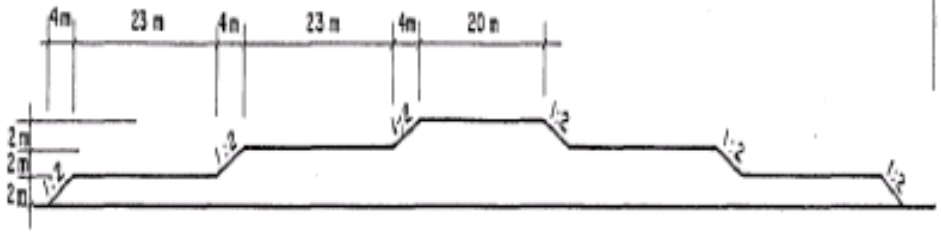
Scheme 6/3



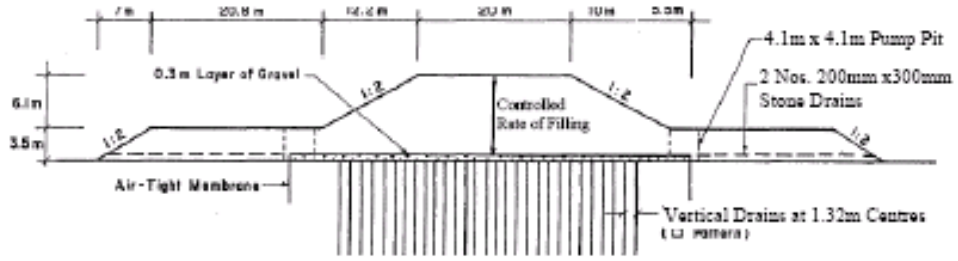
Scheme 6/4



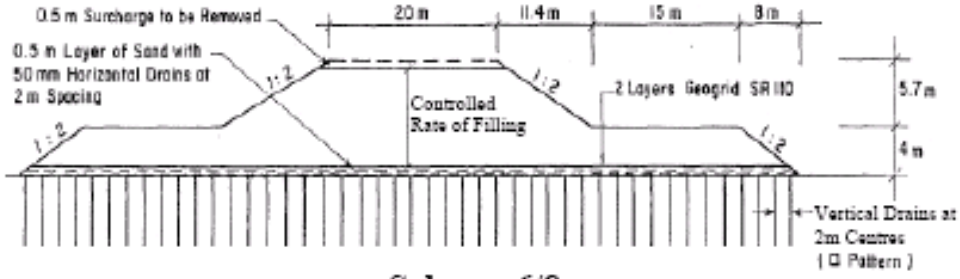
Scheme 6/5



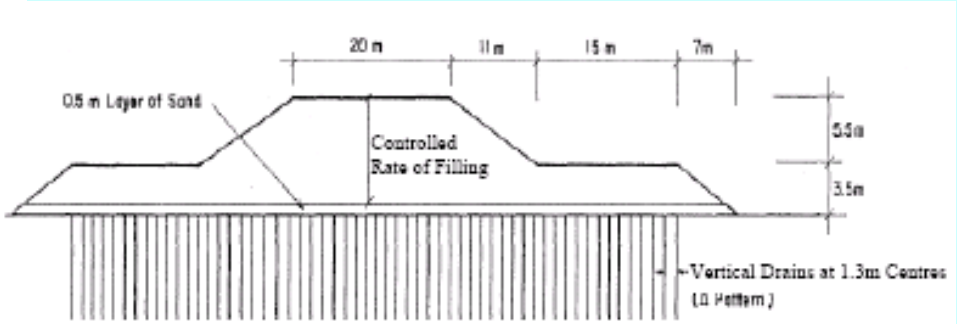
Scheme 6/6



Scheme 6/7

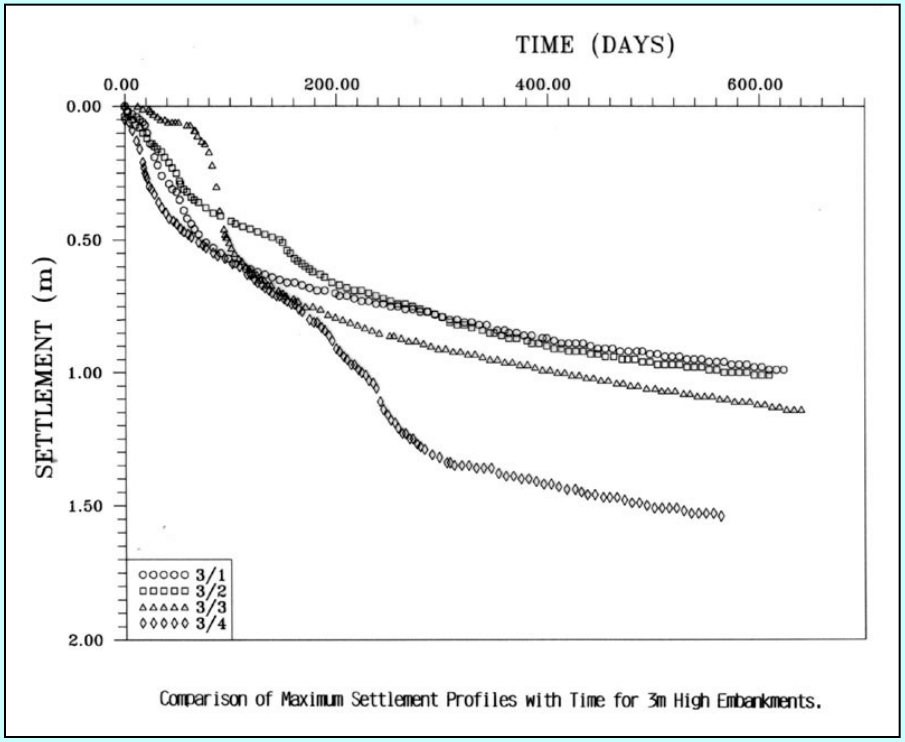


Scheme 6/8

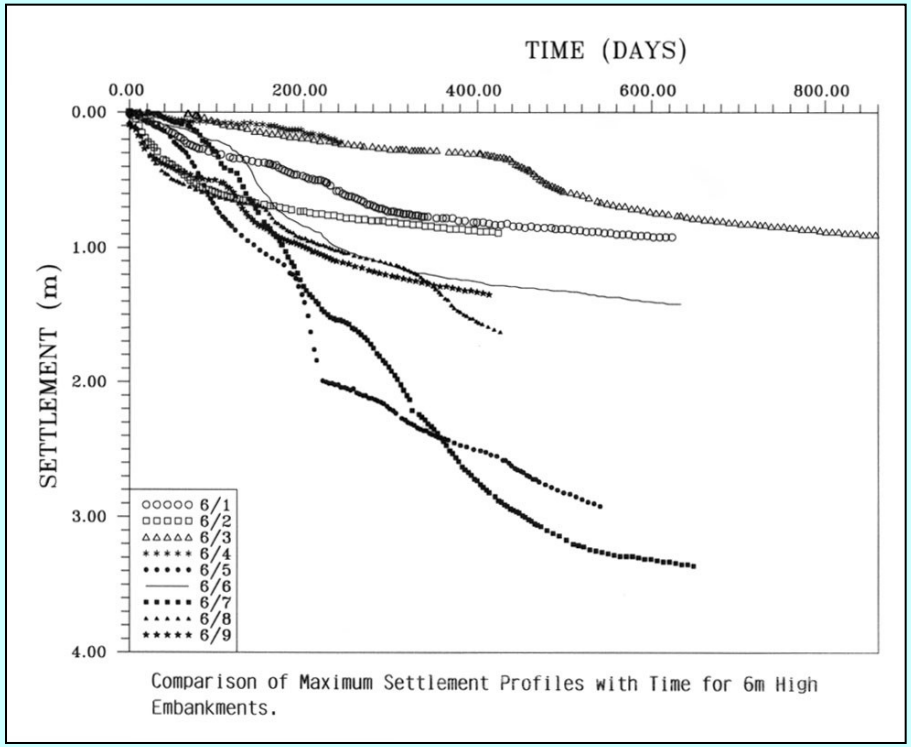


Scheme 6/9

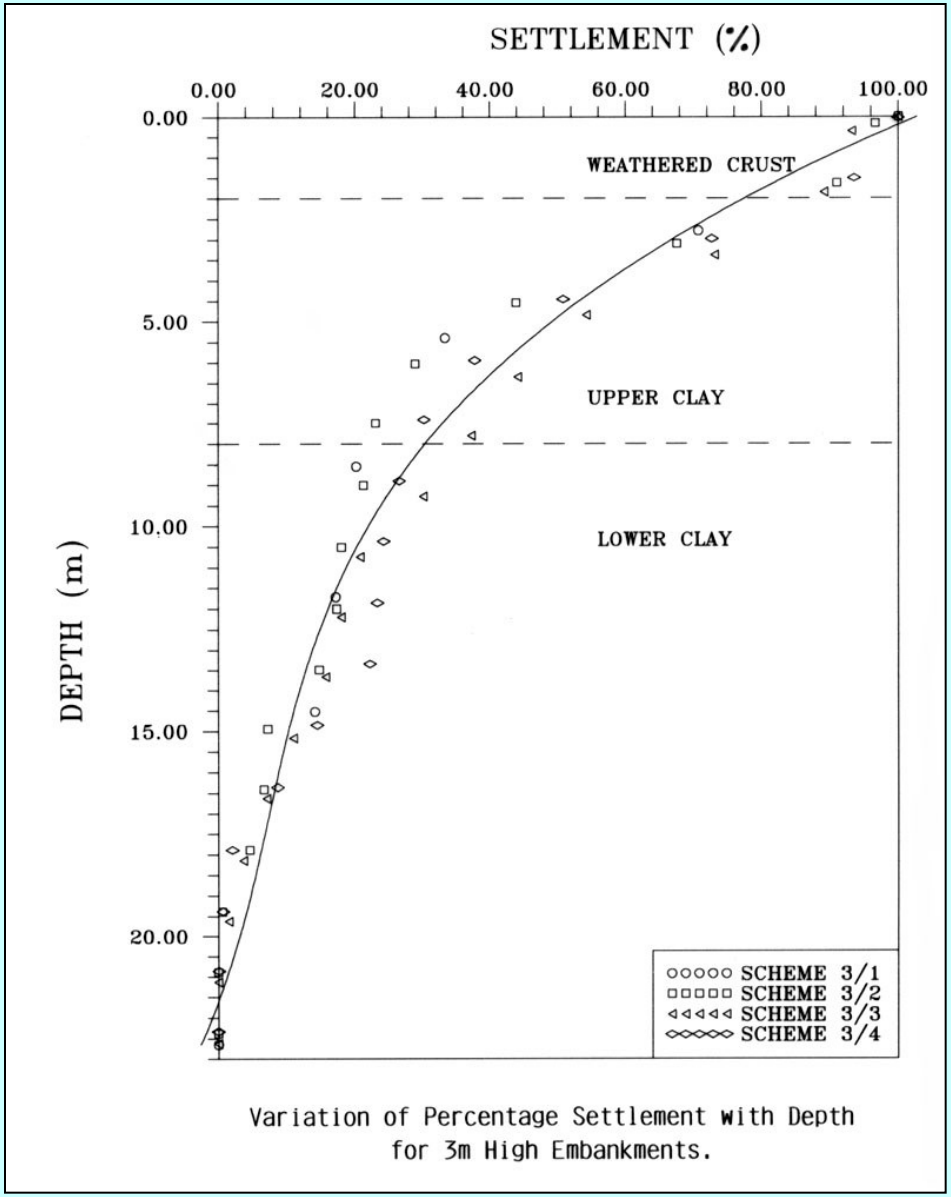
SCHEME	GROUND IMPROVEMENT
6/5	Sand Compaction Piles
6/6	Control
6/7	Vacuum Preloading & Prefabricated Vertical drain
6/8	Preloading, Geogrid & Prefabricated Vertical Drain
6/9	Preloading & Vertical Drain



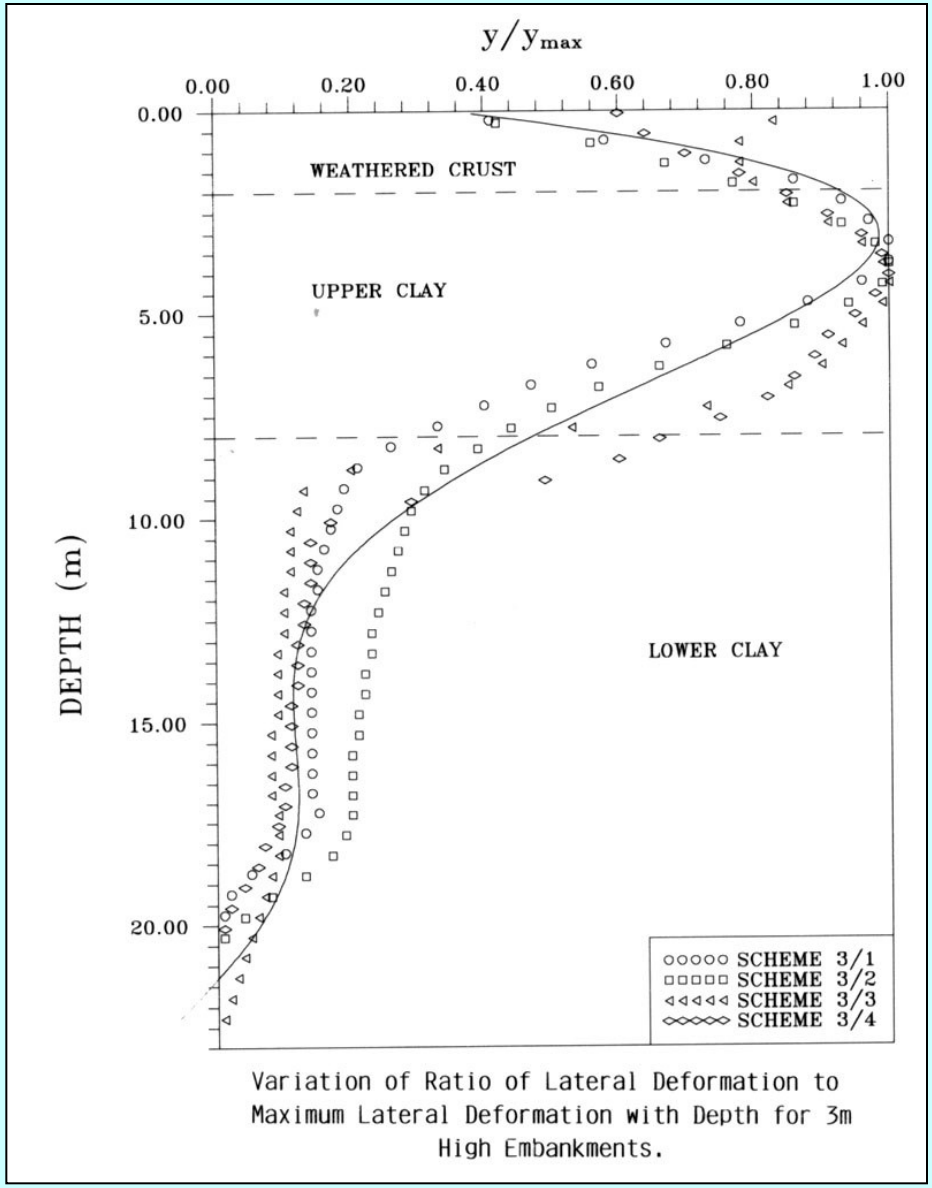
Comparison of Maximum Settlement Profiles with Time for 3m High Embankments.



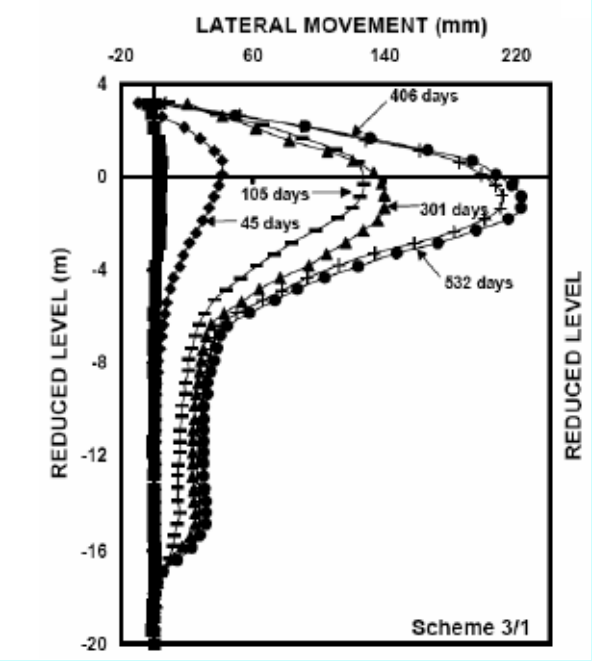
Comparison of Maximum Settlement Profiles with Time for 6m High Embankments.

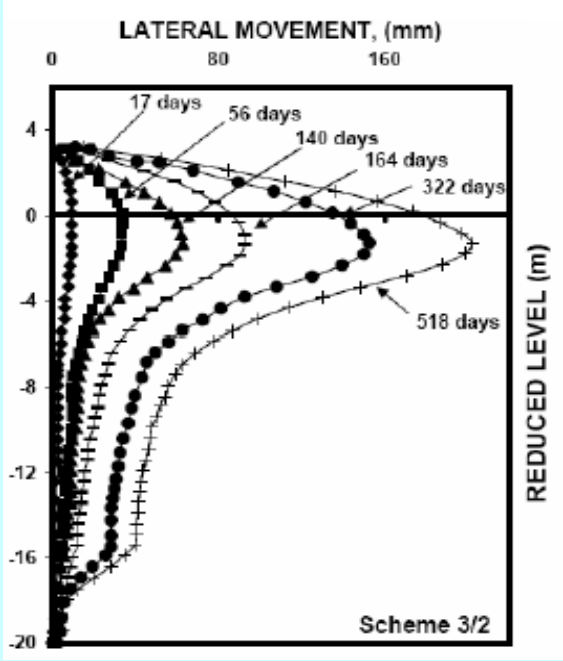


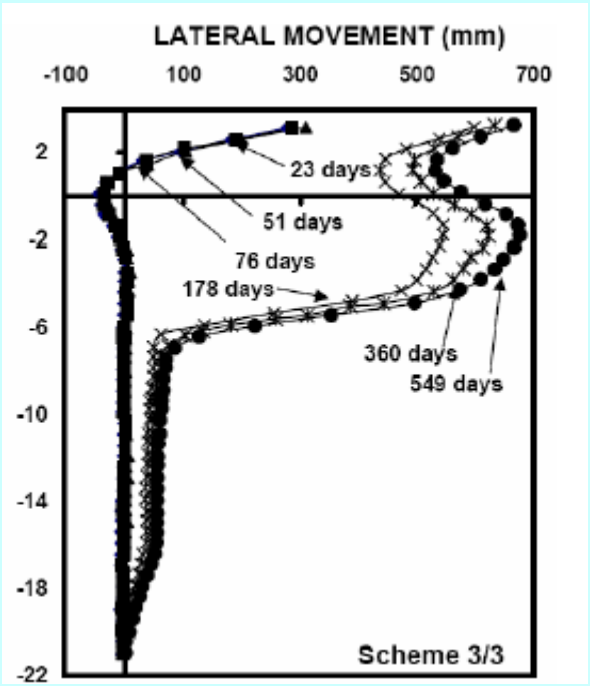
Variation of Percentage Settlement with Depth for 3m High Embankments.

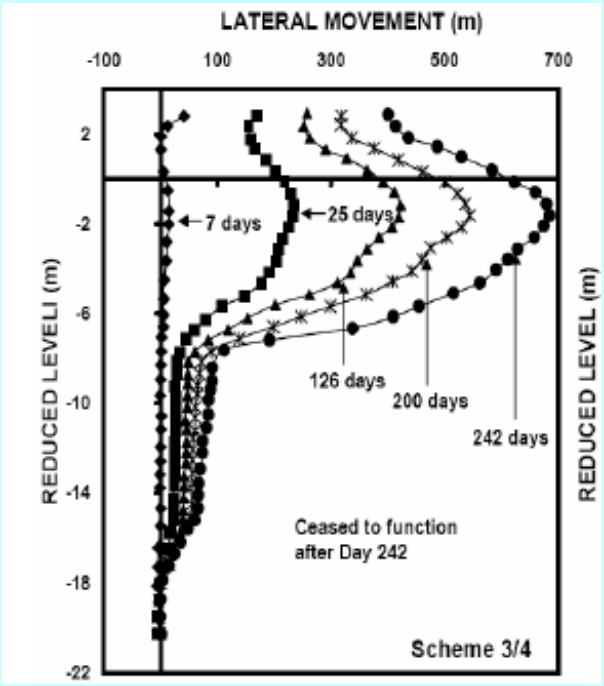


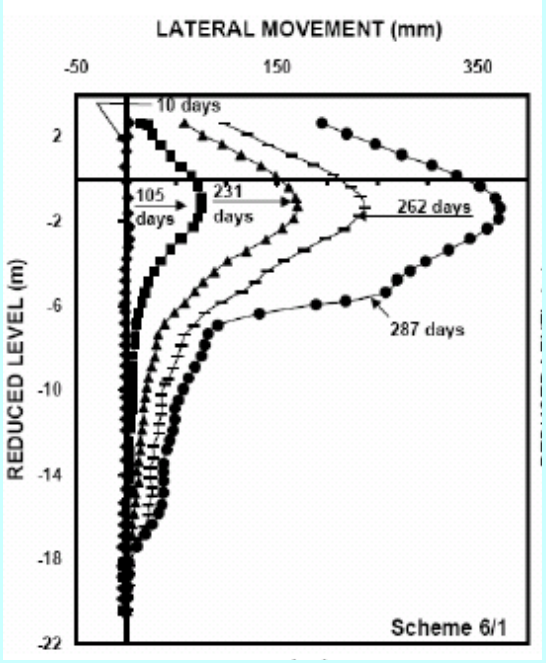
Variation of Ratio of Lateral Deformation to Maximum Lateral Deformation with Depth for 3m High Embankments.

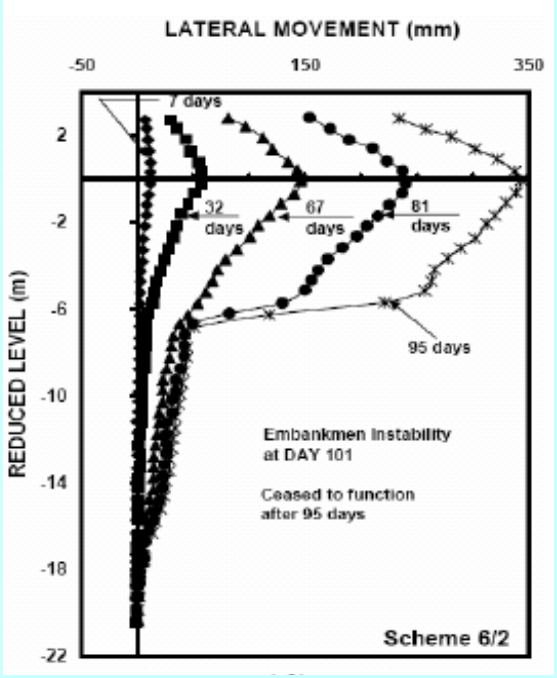

LATERAL MOVEMENT (mm)
-20
60
140
220
4
0
-4
-8
-12
-16
-20
406 days
105 days
45 days
301 days
532 days
REDUCED LEVEL (m)
REDUCED LEVEL
Scheme 3/1

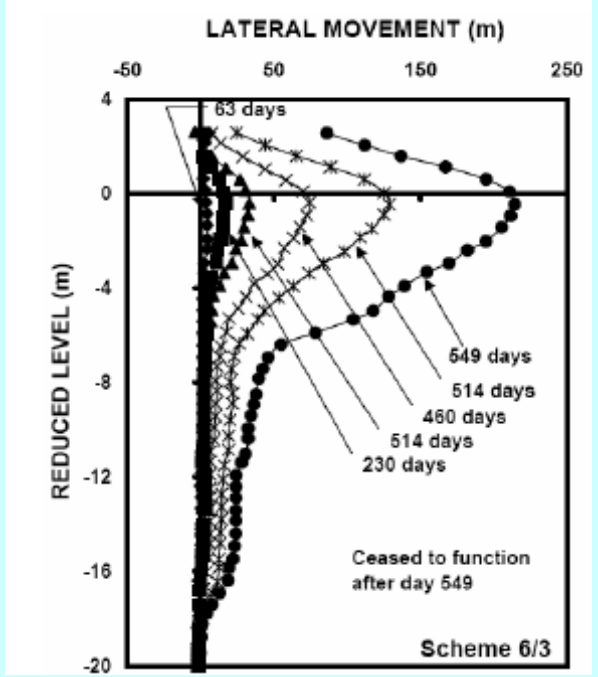

LATERAL MOVEMENT, (mm)
0
80
160
4
0
-4
-8
-12
-16
-20
17 days
56 days
140 days
164 days
322 days
518 days
REDUCED LEVEL (m)
Scheme 3/2

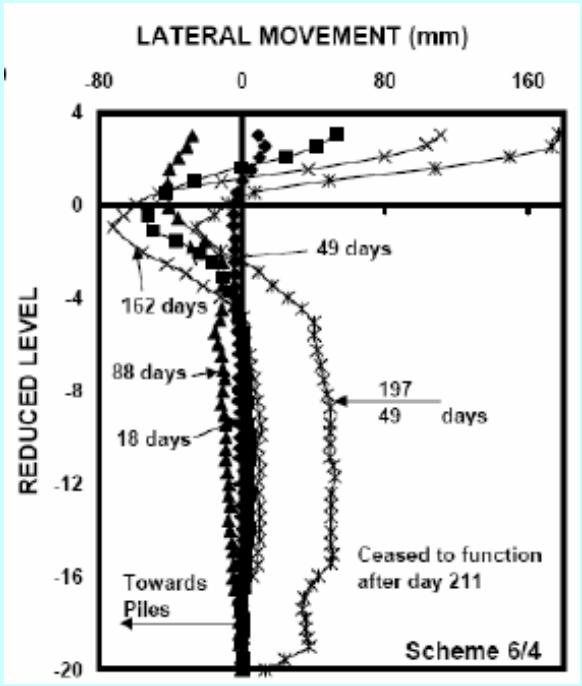

LATERAL MOVEMENT (mm)
-100
100
300
500
700
2
-2
-6
-10
-14
-18
-22
23 days
51 days
76 days
178 days
360 days
549 days
Scheme 3/3

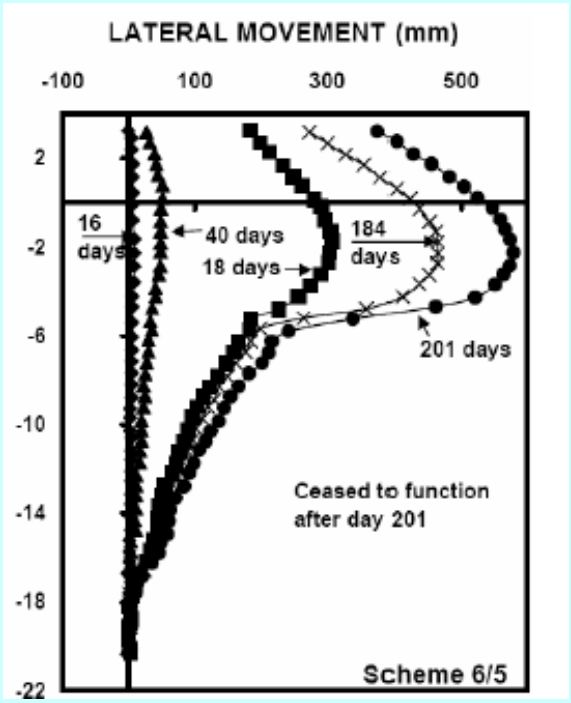

LATERAL MOVEMENT (m)
-100
100
300
500
700
2
-2
-6
-10
-14
-18
-22
7 days
25 days
126 days
200 days
242 days
Ceased to function
after Day 242
REDUCED LEVELI (m)
REDUCED LEVEL (m)
Scheme 3/4

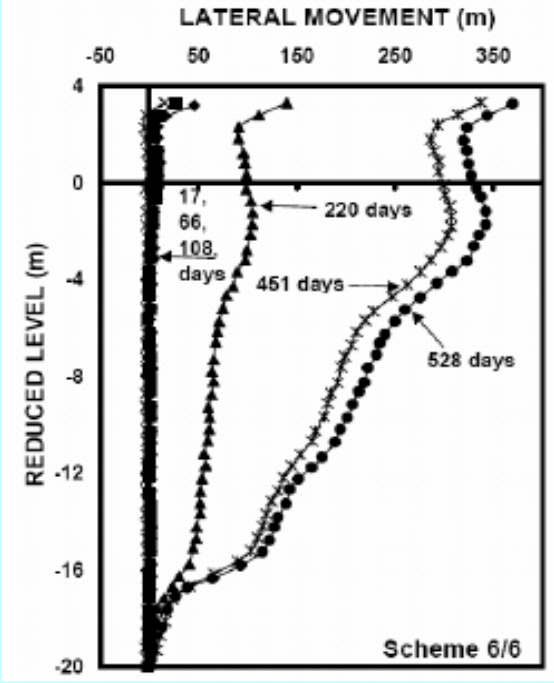

LATERAL MOVEMENT (mm)
-50
150
350
10 days
105 days
231 days
262 days
287 days
REDUCED LEVEL (m)
2
-2
-6
-10
-14
-18
-22
Scheme 6/1

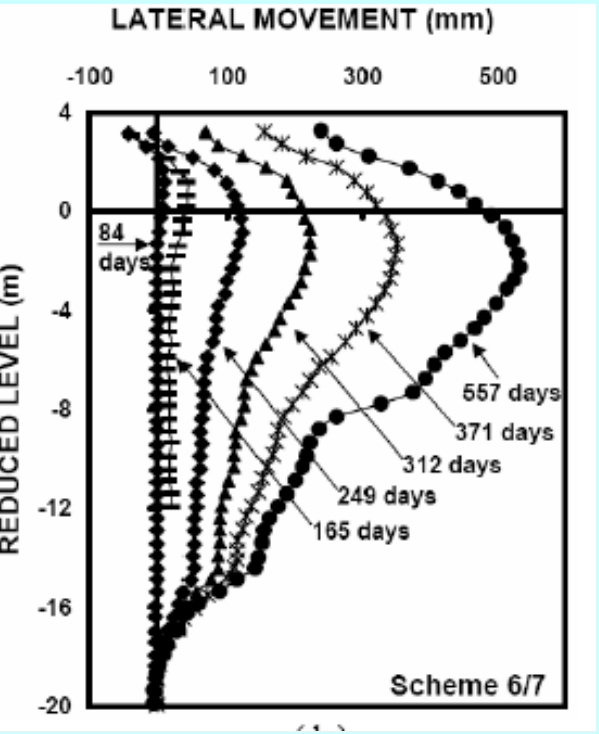

LATERAL MOVEMENT (mm)
-50
150
350
7 days
32 days
67 days
81 days
95 days
Embankmen Instability at DAY 101
Ceased to function after 95 days
REDUCED LEVEL (m)
2
-2
-6
-10
-14
-18
-22
Scheme 6/2

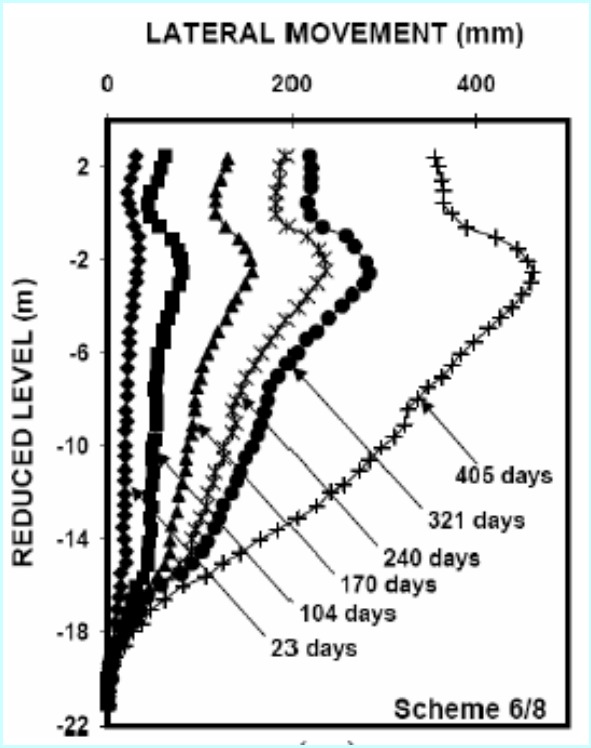

LATERAL MOVEMENT (m)
-50
50
150
250
63 days
549 days
514 days
460 days
514 days
230 days
Ceased to function after day 549
REDUCED LEVEL (m)
4
0
-4
-8
-12
-16
-20
Scheme 6/3

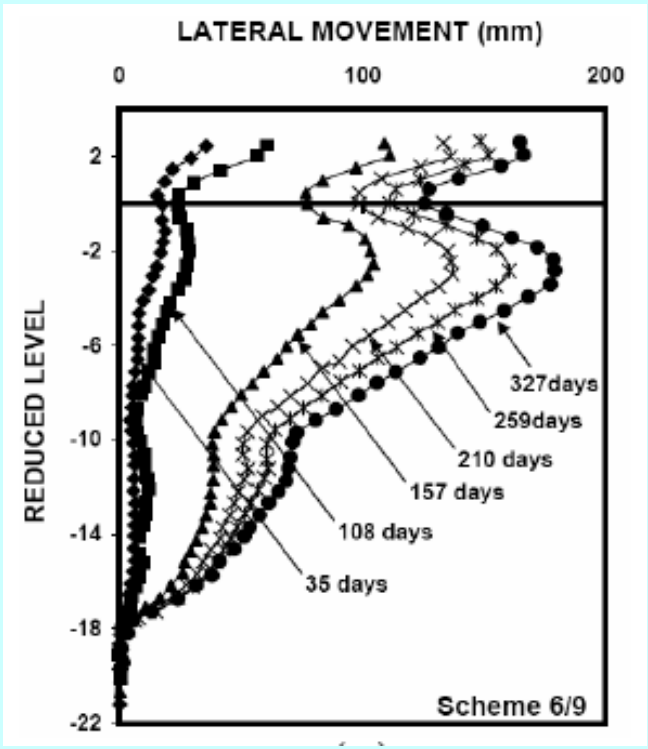

LATERAL MOVEMENT (mm)
-80
0
80
160
49 days
162 days
88 days
197 days
49 days
18 days
Towards Piles
Ceased to function after day 211
REDUCED LEVEL
4
0
-4
-8
-12
-16
-20
Scheme 6/4

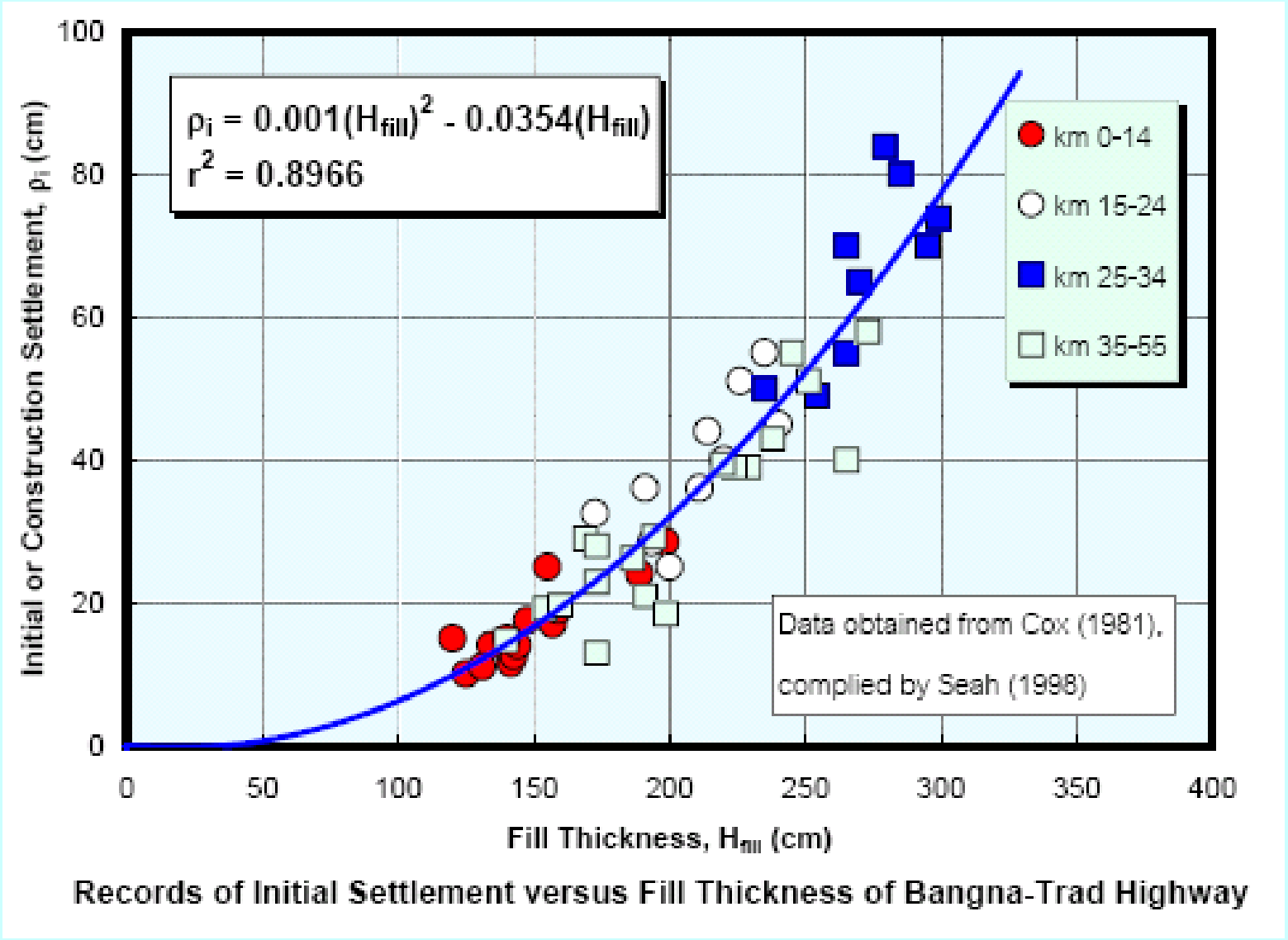

LATERAL MOVEMENT (mm)
-100
100
300
500
16 days
40 days
18 days
184 days
201 days
Ceased to function after day 201
2
-2
-6
-10
-14
-18
-22
Scheme 6/5


LATERAL MOVEMENT (m)
-50
50
150
250
350
17, 66, 108 days
220 days
451 days
528 days
REDUCED LEVEL (m)
4
0
-4
-8
-12
-16
-20
Scheme 6/6

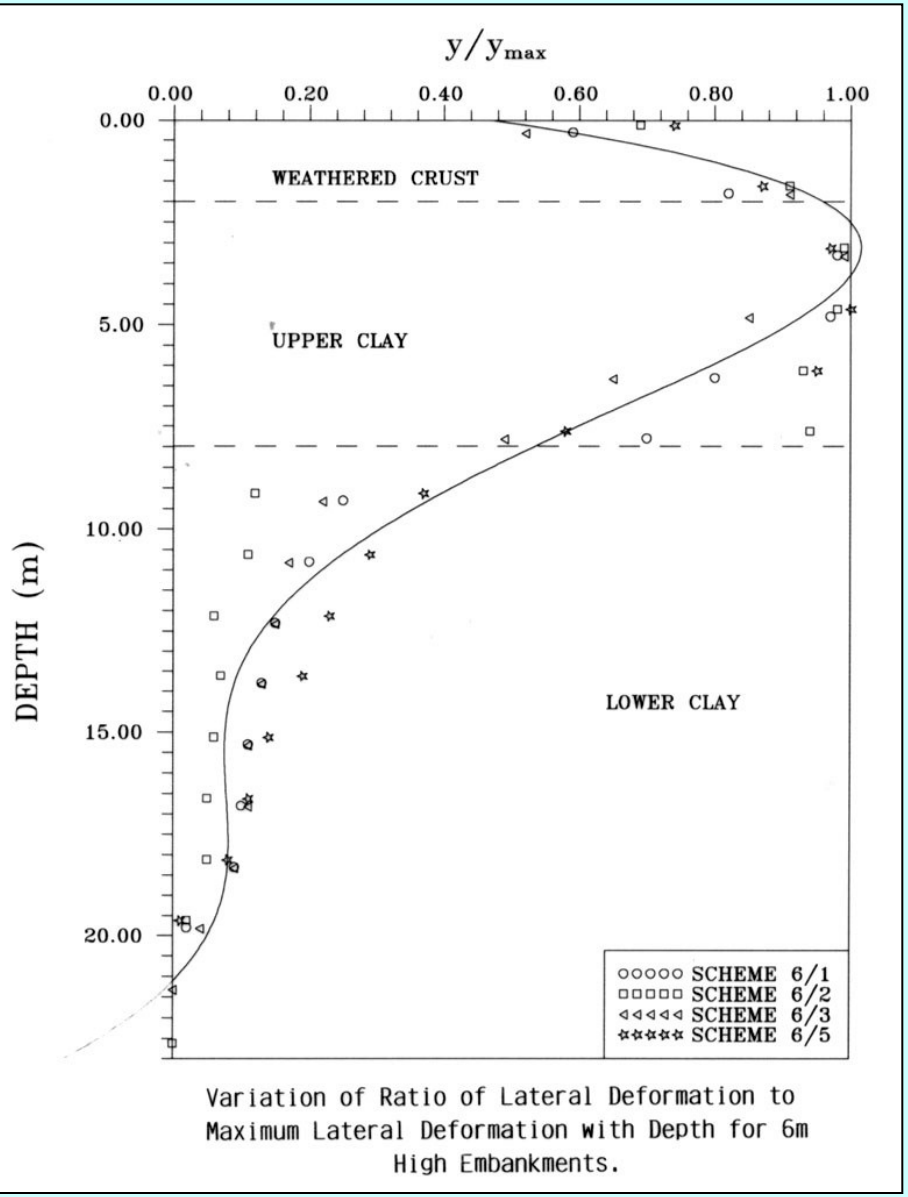

LATERAL MOVEMENT (mm)
-100
100
300
500
4
0
-4
-8
-12
-16
-20
REDUCED LEVEL (m)
84 days
557 days
371 days
312 days
249 days
165 days
Scheme 6/7


LATERAL MOVEMENT (mm)
0
200
400
2
-2
-6
-10
-14
-18
-22
REDUCED LEVEL (m)
405 days
321 days
240 days
170 days
104 days
23 days
Scheme 6/8

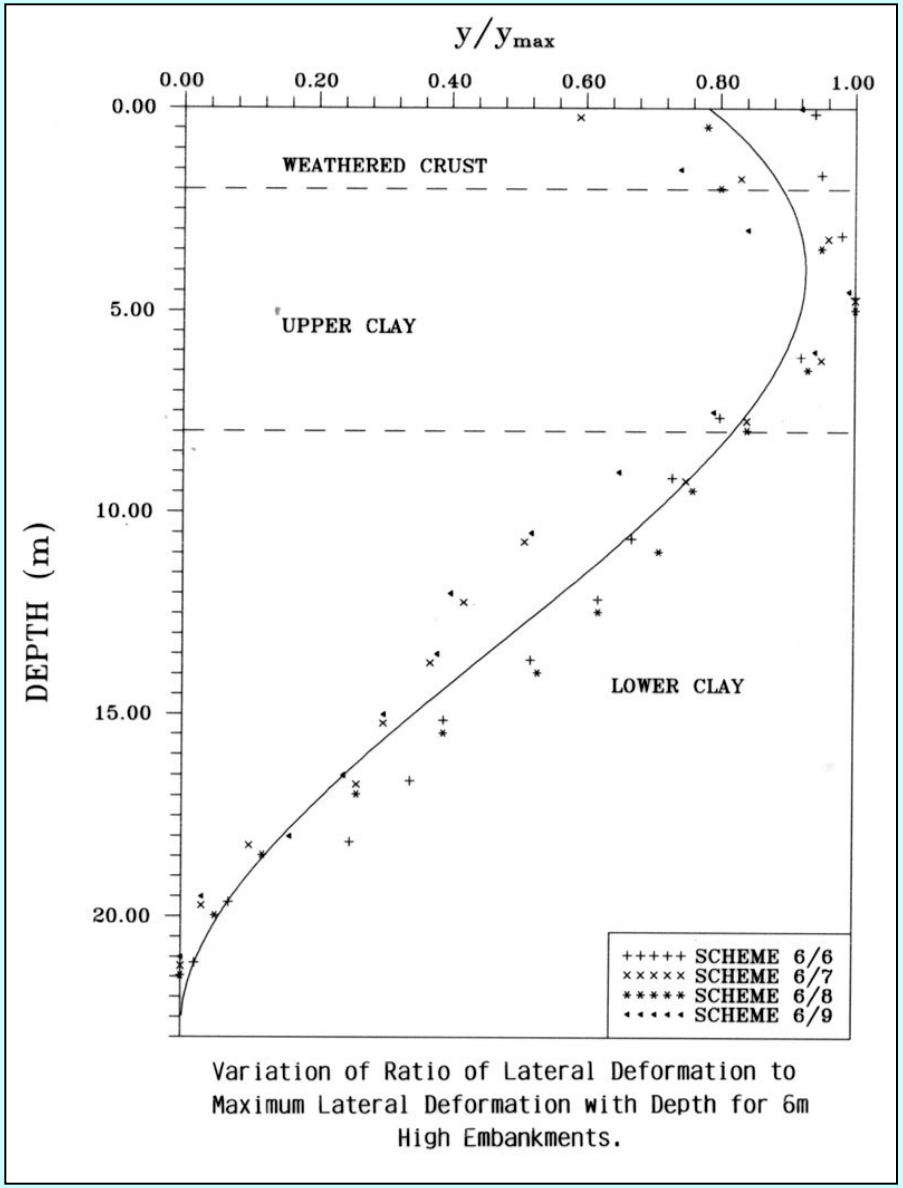

LATERAL MOVEMENT (mm)
0
100
200
2
-2
-6
-10
-14
-18
-22
REDUCED LEVEL
327days
259days
210 days
157 days
108 days
35 days
Scheme 6/9



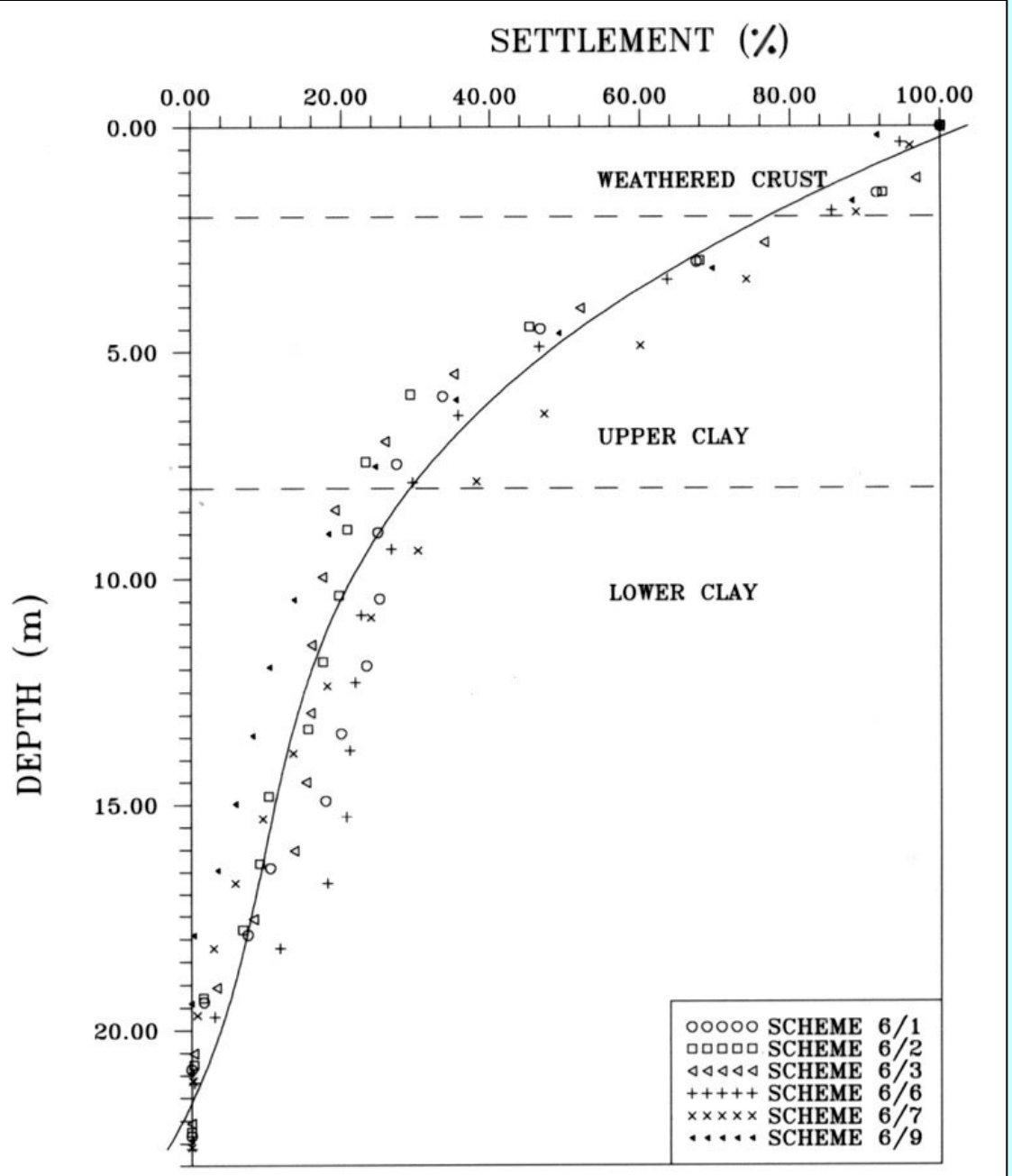
Records of Initial Settlement versus Fill Thickness of Bangna-Trad Highway



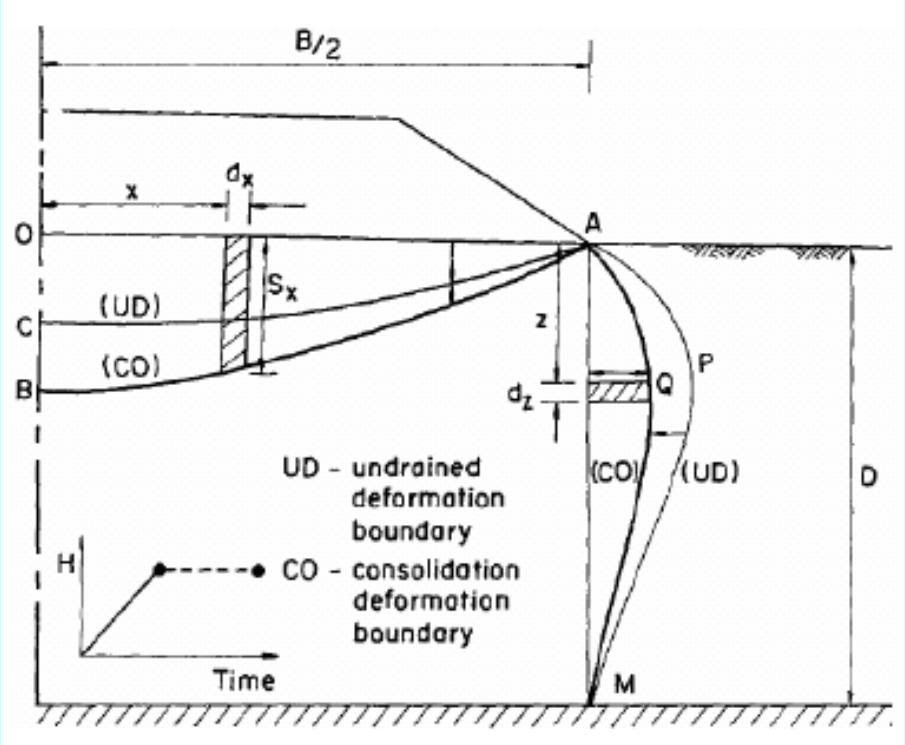
Variation of Ratio of Lateral Deformation to Maximum Lateral Deformation with Depth for 6m High Embankments.

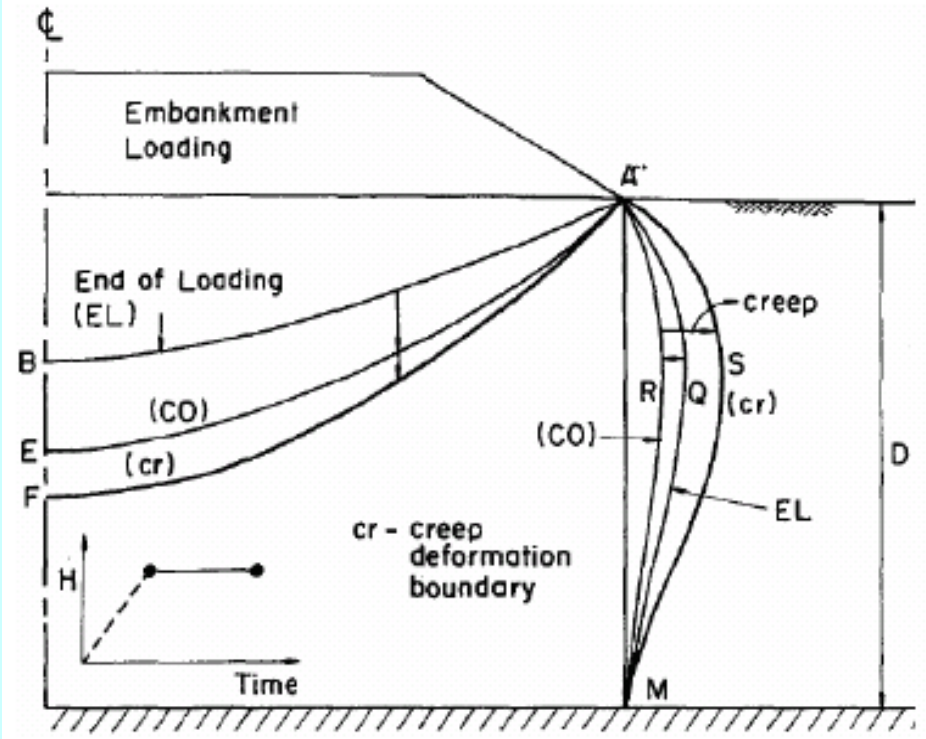


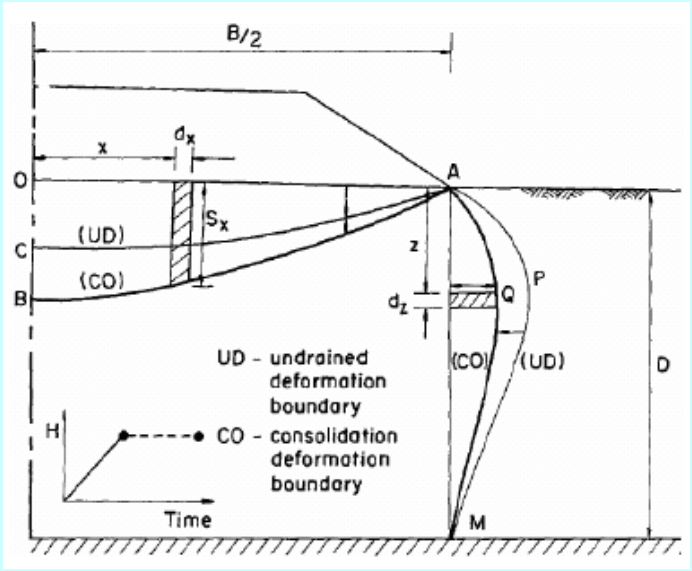
Variation of Ratio of Lateral Deformation to Maximum Lateral Deformation with Depth for 6m High Embankments.

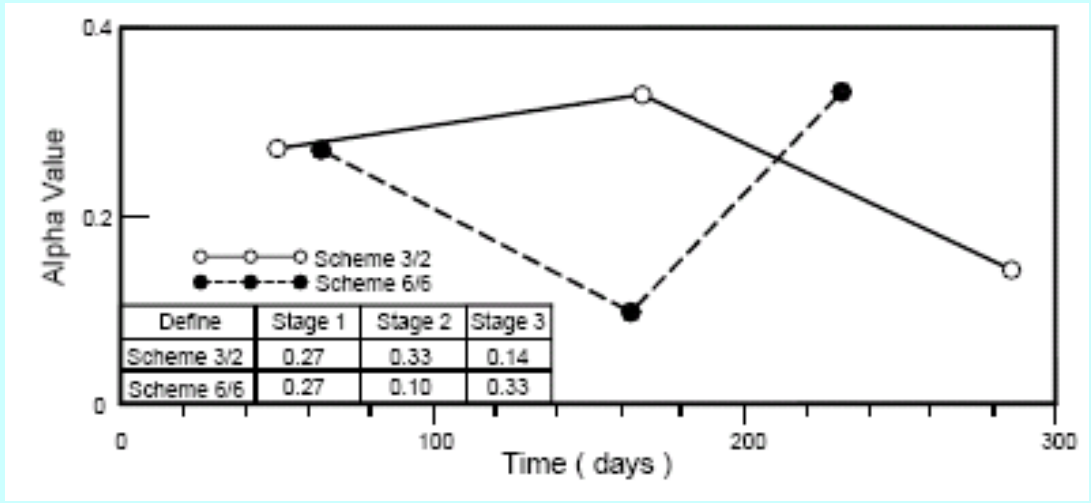


Variation of Percentage Settlement with Depth
for 6m High Embankments.

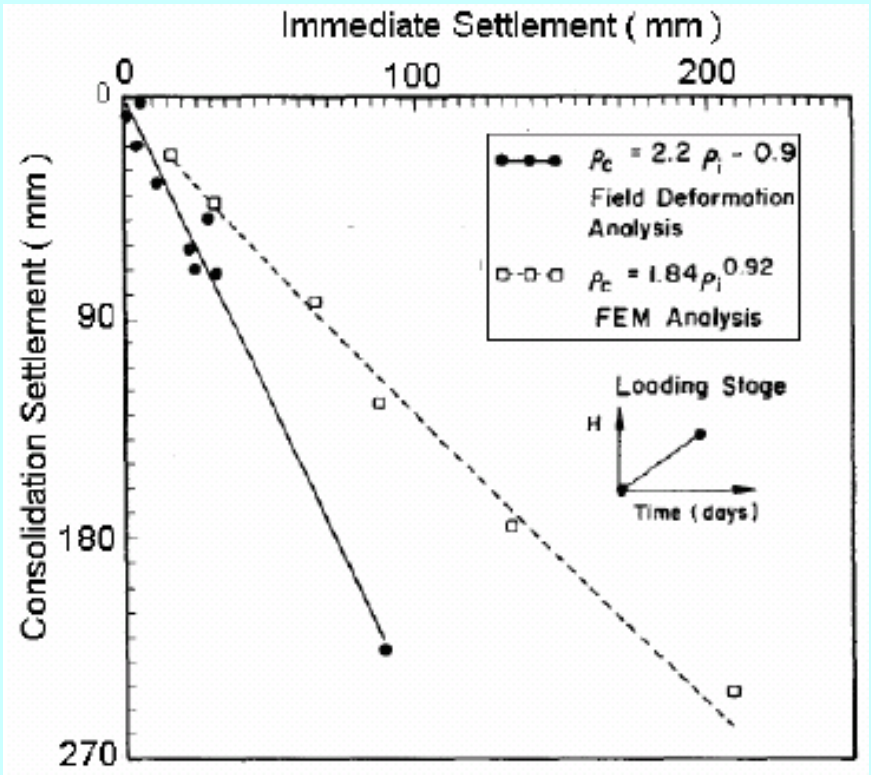

B/2
x
d_x
O
A
S_x
(UD)
C
(CO)
B
z
d_z
Q
P
(CO)
(UD)
D
UD - undrained deformation boundary
CO - consolidation deformation boundary
H
Time
M

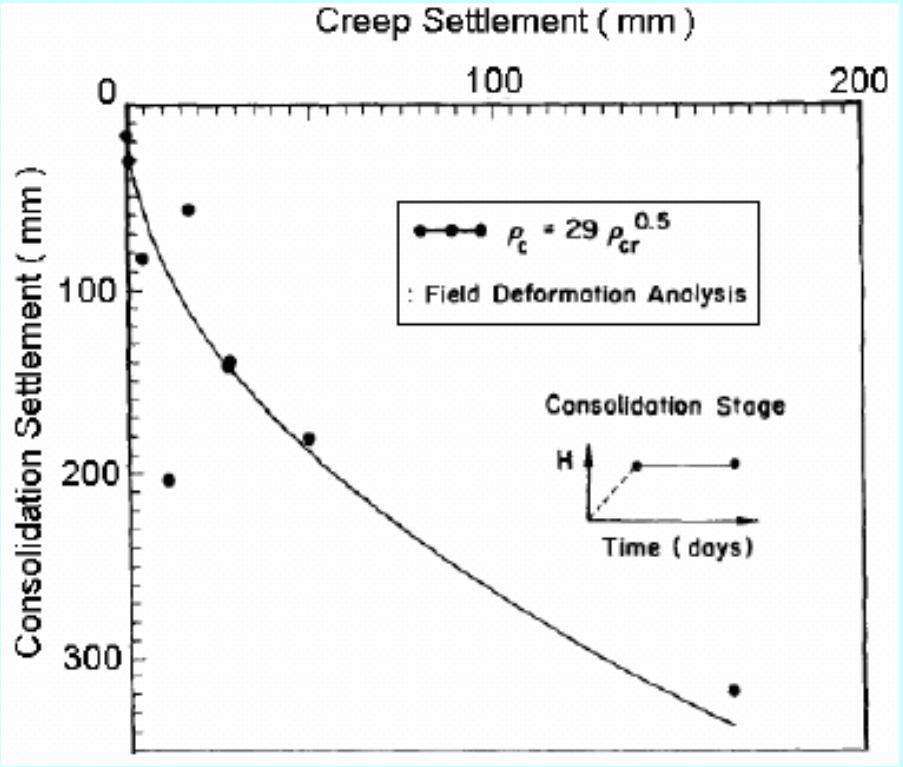

℄
Embankment Loading
A'
End of Loading (EL)
creep
B
S
(cr)
R
Q
(CO)
(CO)
E
(cr)
F
D
EL
cr - creep deformation boundary
H
Time
M

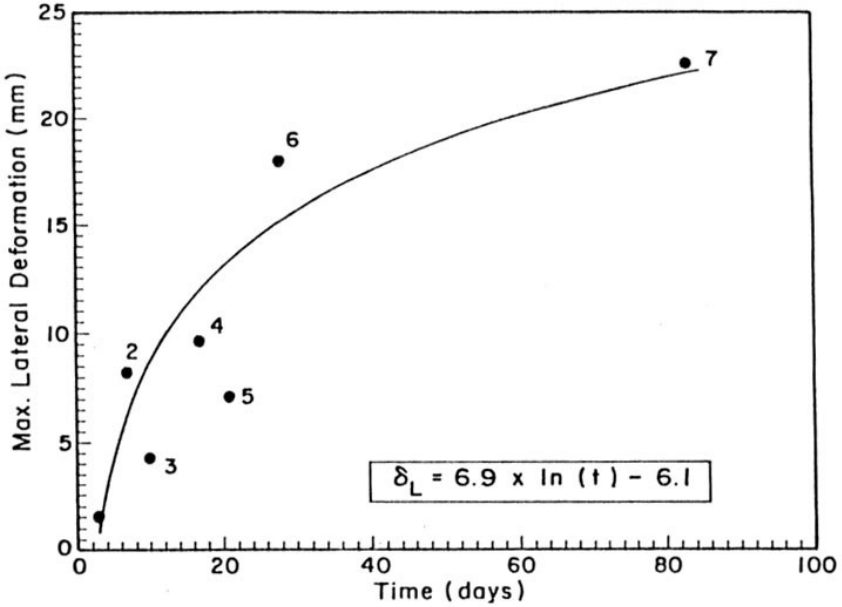




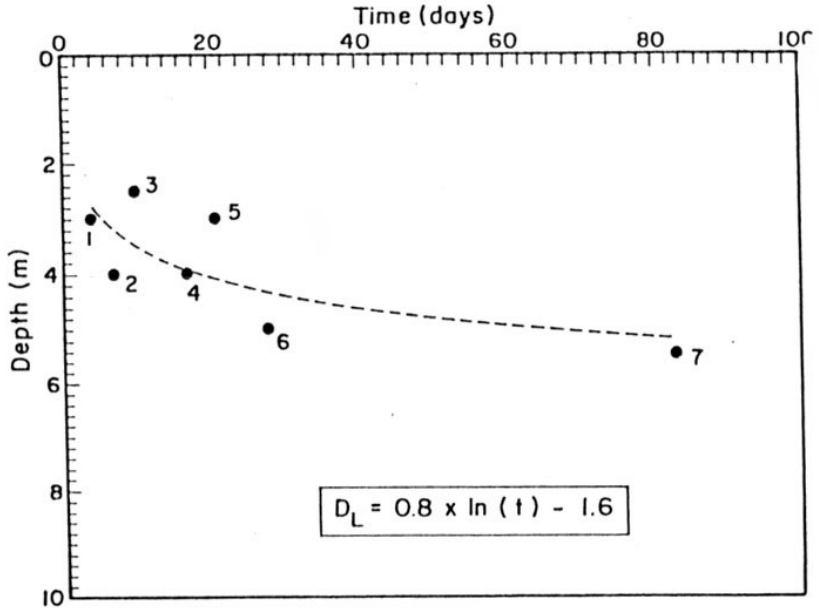
Define	Stage 1	Stage 2	Stage 3
Scheme 3/2	0.27	0.33	0.14
Scheme 6/6	0.27	0.10	0.33



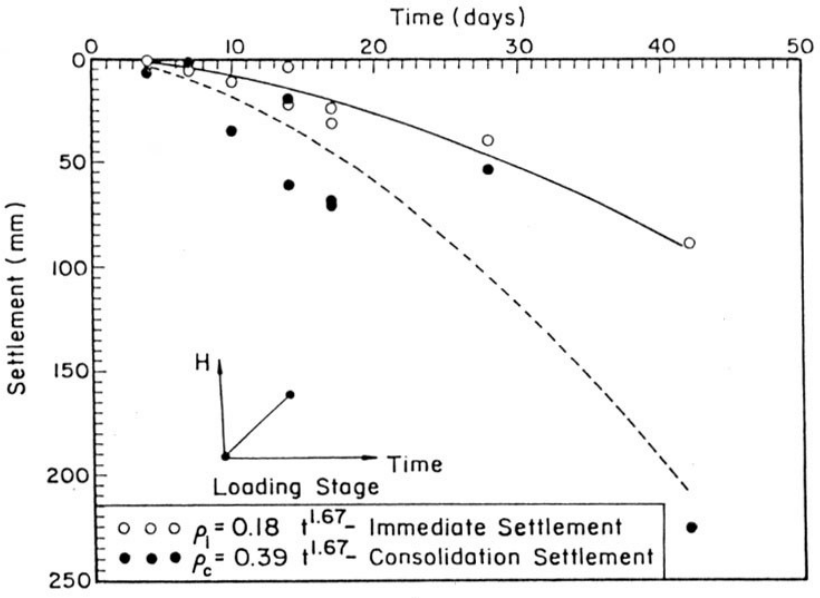




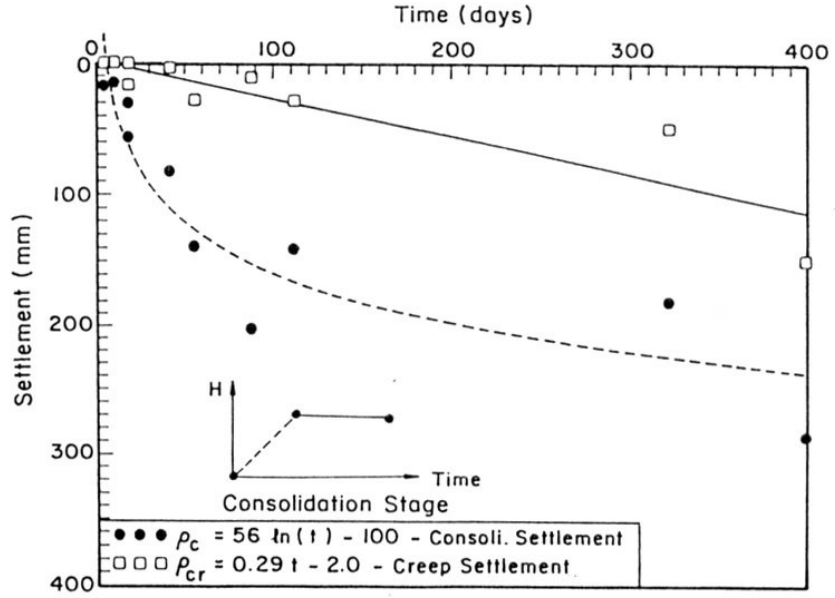
Variation of Lateral Deformation with Time.



Variation of Depth, at which Maximum Lateral Deformation Occurs, with



Plot of Immediate and Consolidation Settlement with Time.



Plot of Consolidation and creep Settlement with Time.

Geotechnical studies at the new Bangkok international airport site (1972-1996)

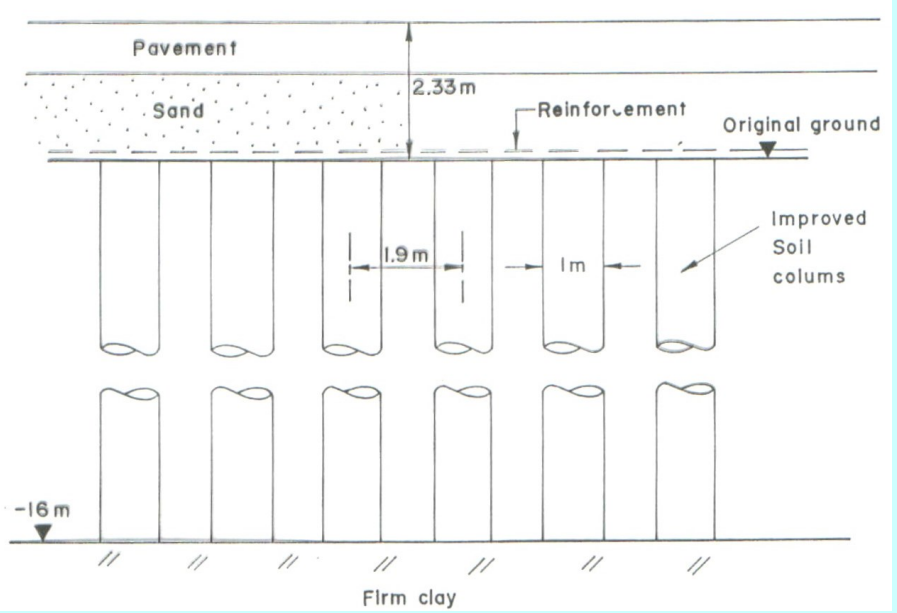
Geotechnical Investigation at Nong Ngo Hao Airport Site

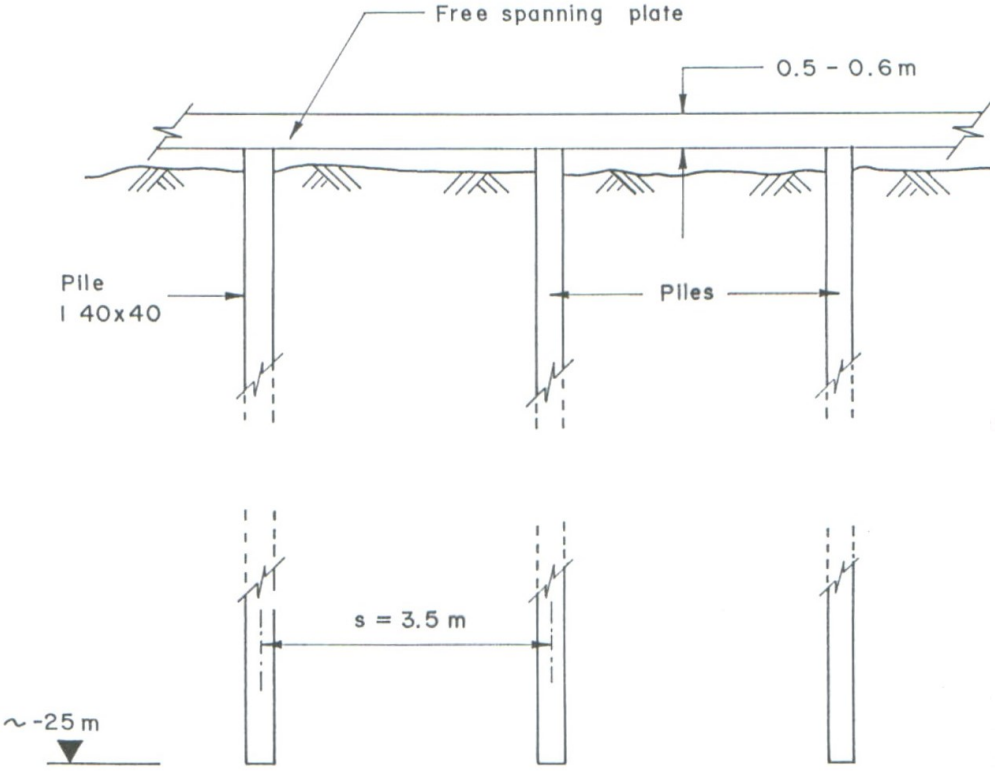
Phase	Year	Title
I	1972 - 1974	Geotechnical Investiga-tions by Asian Institute of Technology and N.D. Lea and Associates, Kampsax
II	1983 - 1984	Pre-loading with Sand Drains, and , Vacuum-Drains; Moh and Associates and NACO
III	1992	An Independent Soil Engineering Study; Norwegian Geotechnical Institute in cooperation the STS Engineering Consultant Co. Ltd.
IV	1993 - 1995	Full scale Field test of Prefabricated vertical drains by the Asian Institute of Technology

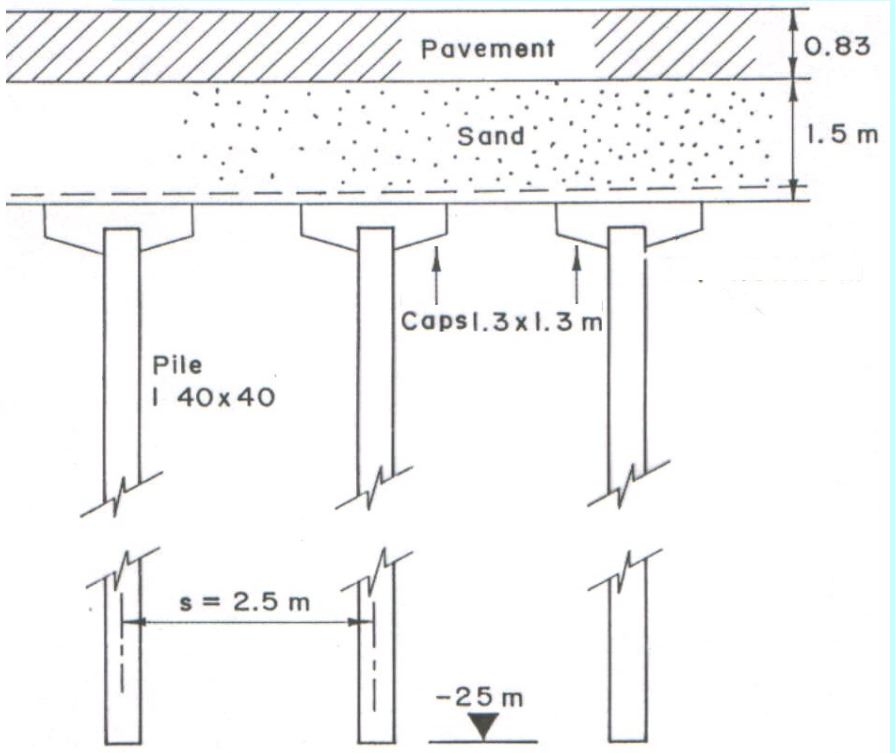
Full scale fields with prefabricated vertical drains (PVD) at the Second Bangkok International Airport

Alternative Design Concepts

- **Preconsolidation with Vertical Drain**
- **Deep Soil Improvement**
- **Piles supporting a free spanning concrete slab**
- **Relief Piles with Caps**







PVD Installation

Scope of Work

1. From published information, the types of suitable Prefabricated Vertical Drains (PVD).
2. Laboratory tests to determine the desirable PVD properties.
3. Field performance of at least three PVD types.
4. Controlling parameters, i.e., PVD properties, spacing and depth of PVD.
5. Comparative performance of PVD and sand drains (as studied in 1983).
6. Criteria for selecting PVD, design approach, installation procedures and specifications.

Background

1. Previous negative experience with large diameter sand drains by NGI in highway projects in Bangkok.

2. Previous negative experience with sandwicks at the Dockyard site in Bangkok.

3. Previous negative experience with vacuum drains at the airport site in Bangkok.

4. No clear evidence of pore pressure dissipation at the Changi reclamation project in Singapore.

5. Performance of Desol PVD at the Muar site in Malaysia.

6. Piezometric draw-down due to deep well pumping and possible fear of hydraulic connections between PVD and the underlying aquifers in Bangkok.

List of PVD Considered from Worldwide Survey

- **Alidrain (Studded on both sides)**
- **Ameridrain (408)**
- **Castle Board (CS1)**
- **Colbond (CX 1000)**
- **Flodrain (FD4-EX)**
- **Geodrain (L-Type)**
- **Mebra (MD-7007)**

(i) Drains with separate core and filter:

(a) **grooved core :Ameridrain, Mebra and Geodrain**

(b) **studded core :Flodrain**

(c) **filament core :Colbond**

(ii) Drains with filter fixed to core

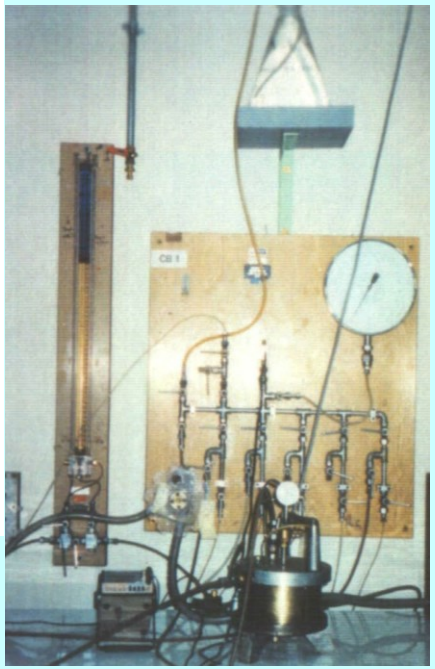
(a) **grooved core : Castle Board**

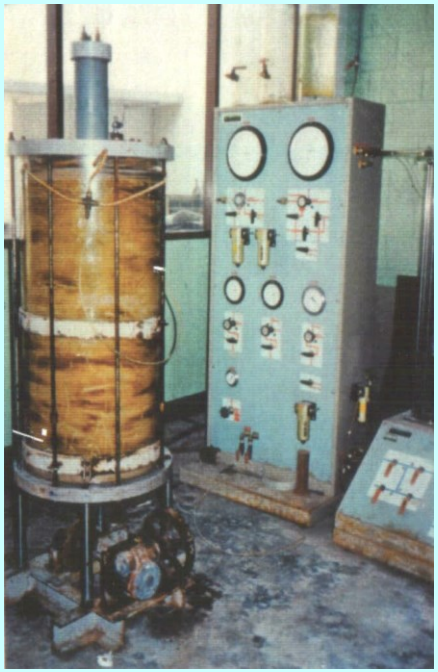
PVD Selection, Installation Procedures and Specifications

- **Criteria to ensure safe installation of the Drains**
- **Criteria to ensure the Optimal Performance of the Drains**
- **Proven record of successful use in similar Soil Conditions**
- **Cost and Availability**

Specification Criteria for PVD

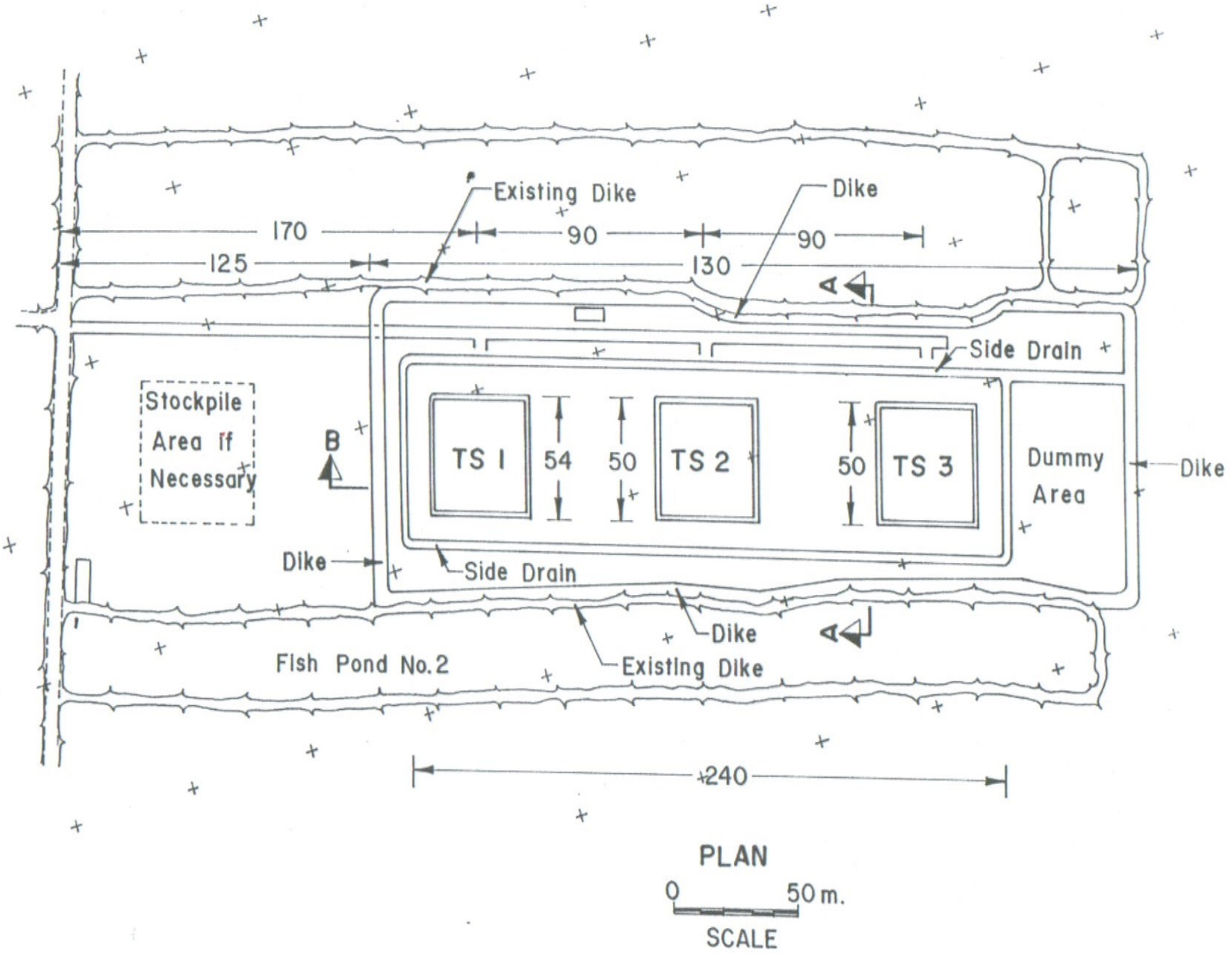
Properties	Test Designation	Proposed Values
Apparent Opening Size, μm	ASTM D4751-87	Less than 90
Grab Tensile Strength, kN	ASTM D4632-91	Greater than 0.35
Trapezoidal Tear Strength, kN	ASTM D4533-91	Greater than 0.10
Puncture Resistance, kN	ASTM D4833-88	Greater than 0.10
Burst Strength, kN	ASTM D3786-80a	Greater than 900
Discharge Capacity at 7 days, 200 kPa at Hydraulic Gradient of 1 m^3/yr	ASTM D4716-87	Greater than 500
Discharge Capacity @ 200 kPa and Hydraulic Gradient of 1 m^3/yr	Modified Triaxial (Straight)	Greater than 500
Equivalent Dia. = (Length + Width)/2		Greater than 50 mm
O_{90}/D_{85} (Opening Size of Filter / Grain Size of Clay)		Less than 3
O_{50}/D_{50} (Opening Size of Filter / Grain Size of Clay)		Less than 24

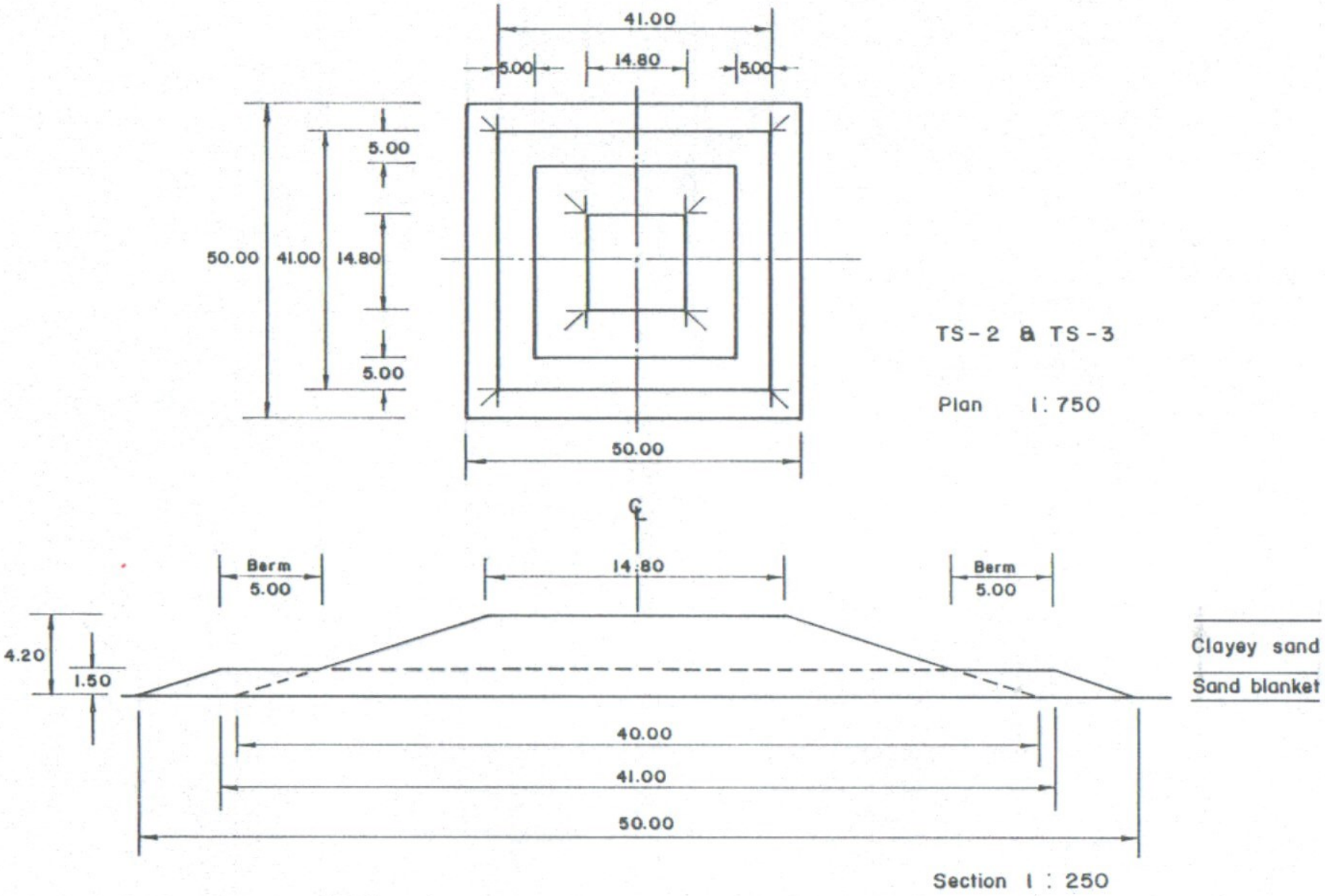




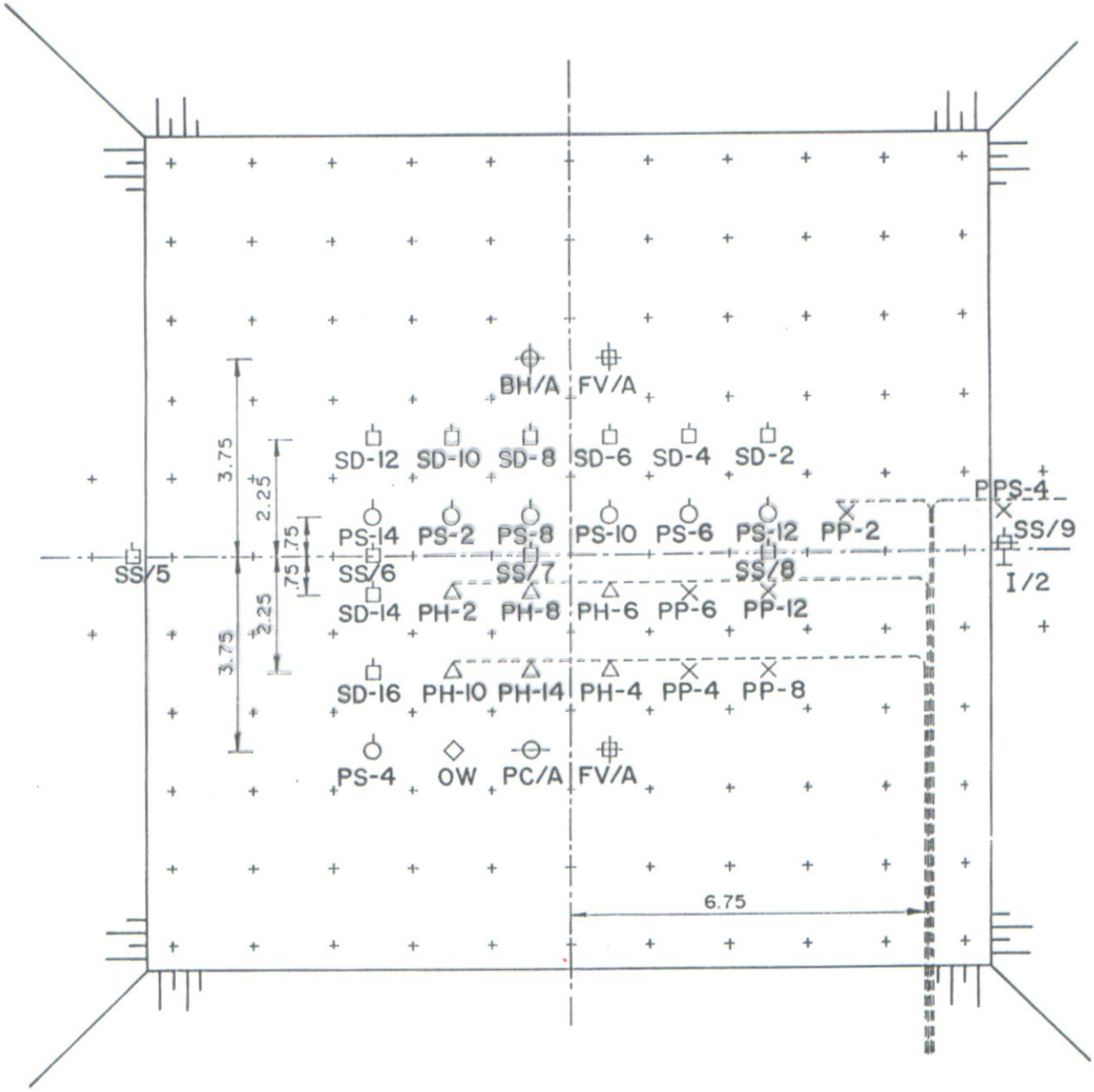
Salient Features of the Project

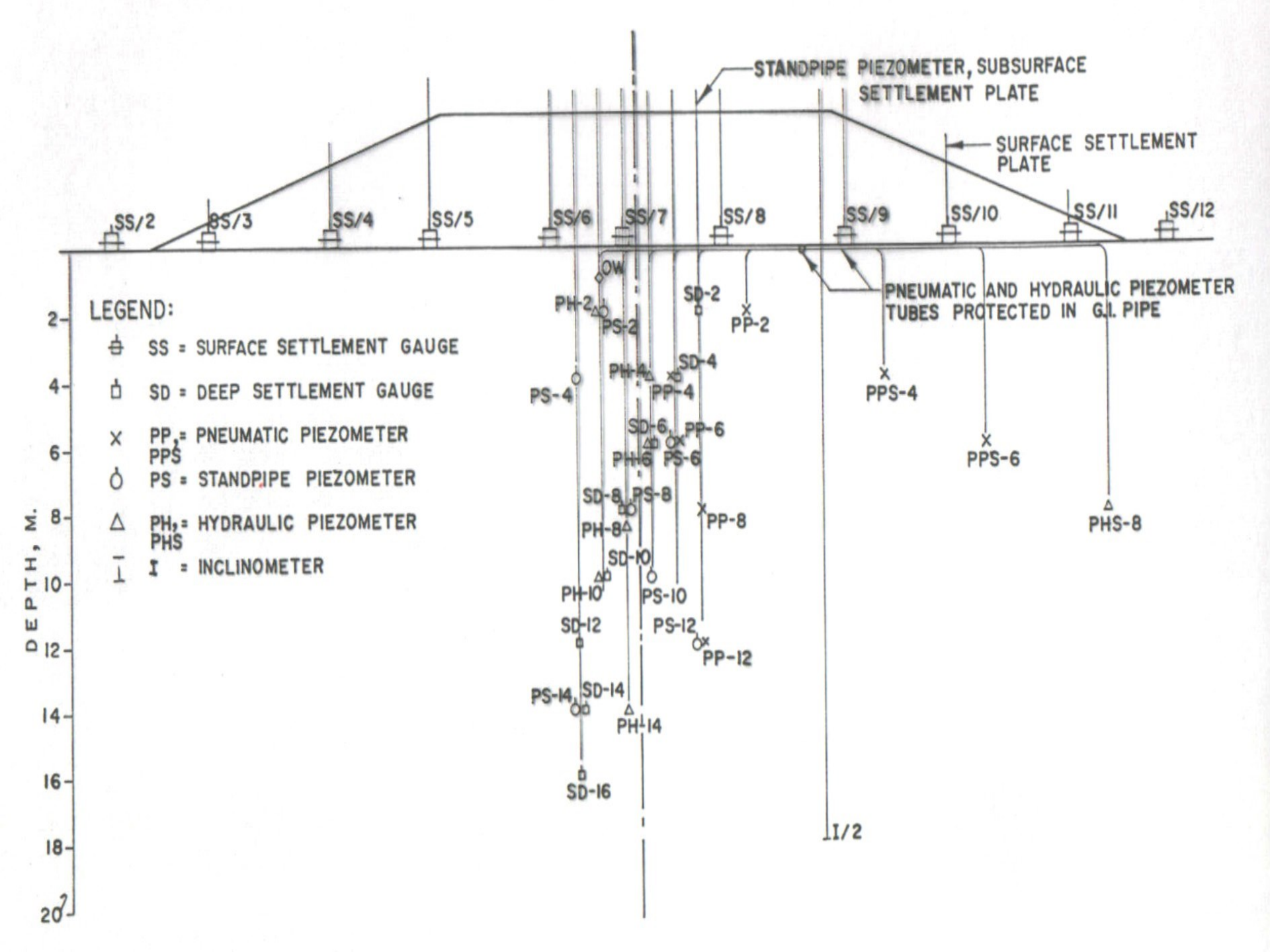
1. Selection of PVD
2. Construction in the rainy season under flooded conditions
3. Stability of the test embankments
4. Have to really prove that the settlement is due to consolidation and not from undrained yielding without any volume change
5. The piezometric draw-down due to subsidence made the computation of settlement from pore pressure dissipation difficult.
6. Computations need to convince that the degree of consolidation estimated from pore pressure dissipation and settlement measurements are comparable.
7. Undrained strength measurements should reflect the strength increase due to water content reductions.
8. Reason for continuing settlements.

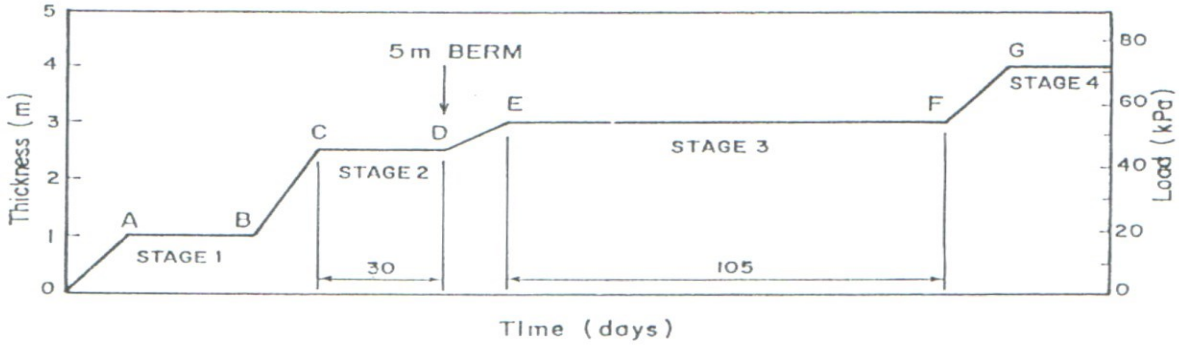

Existing Dike
Dike
170
90
90
125
130
A
Side Drain
Stockpile
Area if
Necessary
B
TS 1
54
50
TS 2
50
TS 3
Dummy
Area
Dike
Dike
Side Drain
Dike
Existing Dike
Fish Pond No.2
240
PLAN
0
50 m.
SCALE



Details of Test Embankments TS2 and TS3


BH/A
FV/A
SD-12
SD-10
SD-8
SD-6
SD-4
SD-2
PPS-4
PS-14
PS-2
PS-8
PS-10
PS-6
PS-12
PP-2
SS/9
SS/5
SS/6
SS/7
SS/8
I/2
SD-14
PH-2
PH-8
PH-6
PP-6
PP-12
SD-16
PH-10
PH-14
PH-4
PP-4
PP-8
PS-4
OW
PC/A
FV/A
3.75
2.25
.75
.75
2.25
3.75
6.75


STANDPIPE PIEZOMETER, SUBSURFACE SETTLEMENT PLATE
SURFACE SETTLEMENT PLATE
SS/2
SS/3
SS/4
SS/5
SS/6
SS/7
SS/8
SS/9
SS/10
SS/11
SS/12
PNEUMATIC AND HYDRAULIC PIEZOMETER TUBES PROTECTED IN G.I. PIPE
LEGEND:
SS = SURFACE SETTLEMENT GAUGE
SD = DEEP SETTLEMENT GAUGE
PP, PPS = PNEUMATIC PIEZOMETER
PS = STANDPIPE PIEZOMETER
PH, PHS = HYDRAULIC PIEZOMETER
I = INCLINOMETER
DEPTH, M.
2
4
6
8
10
12
14
16
18
20
OW
PH-2
PS-2
SD-2
PP-2
PS-4
PH-4
SD-4
PP-4
PPS-4
SD-6
PP-6
PH-6
PS-6
PPS-6
SD-8
PS-8
PP-8
PH-8
PHS-8
SD-10
PH-10
PS-10
SD-12
PS-12
PP-12
PS-14
SD-14
PH-14
SD-16
I/2



SUMMARY OF THE STABILITY ANALYSIS

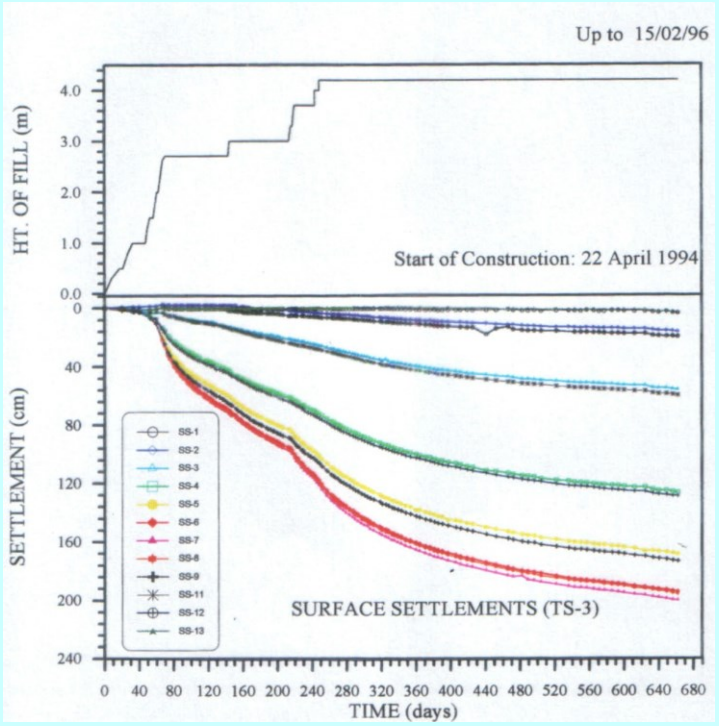
POINT	THICKNESS (m)	LOAD (kPa)	DURATION (Days)	FACTOR OF SAFETY	
				with 5 kPa Load	without Load
C	2.5	45		1.18	1.30
D	2.5	45	30	1.33	1.48
E	3.0	54		1.23	1.34
F	3.0	54	105	1.54	1.65
G	4.0	72		1.26	1.34

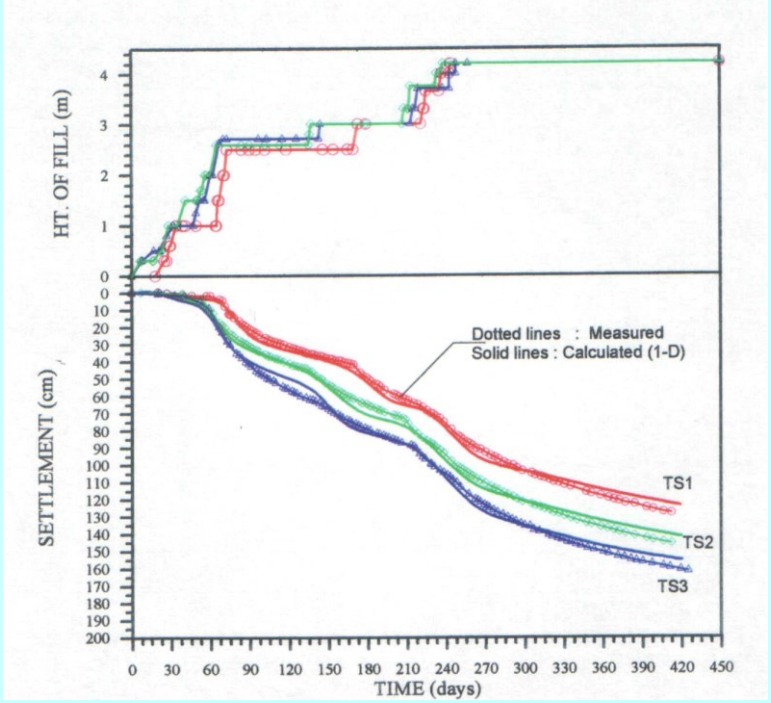
Calculated Strength Gain and Settlement at the End of Each Loading Stage

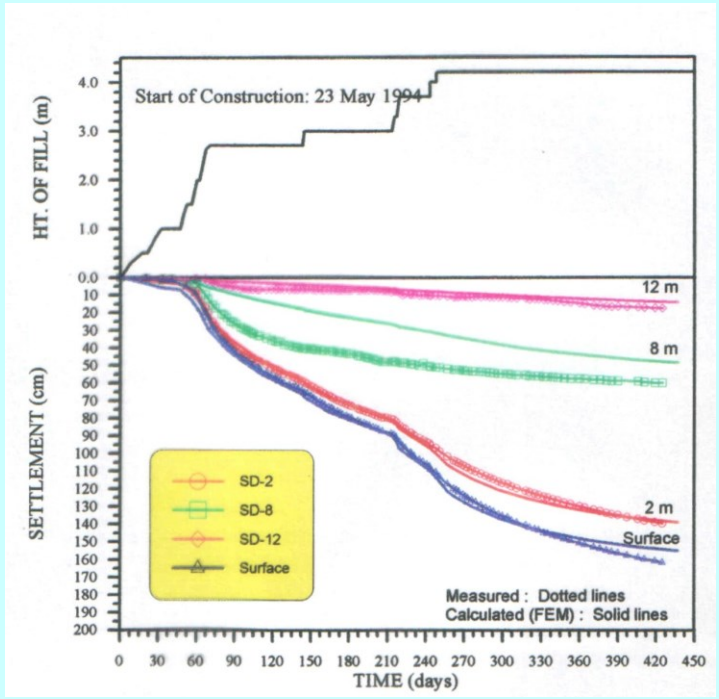
POINT	$\Delta\sigma_v'$ (kPa)	$RS_u = S_u/S_{uo}$	$\Delta\sigma_v'/\Delta q_c$	$\Delta\sigma_v'/\Delta u_p$	S_c (cm)	S_c/S_{cf}
D	11.4	1.07	0.25	0.25	22	0.17
F	35.1	1.42	0.65	0.83	65	0.50

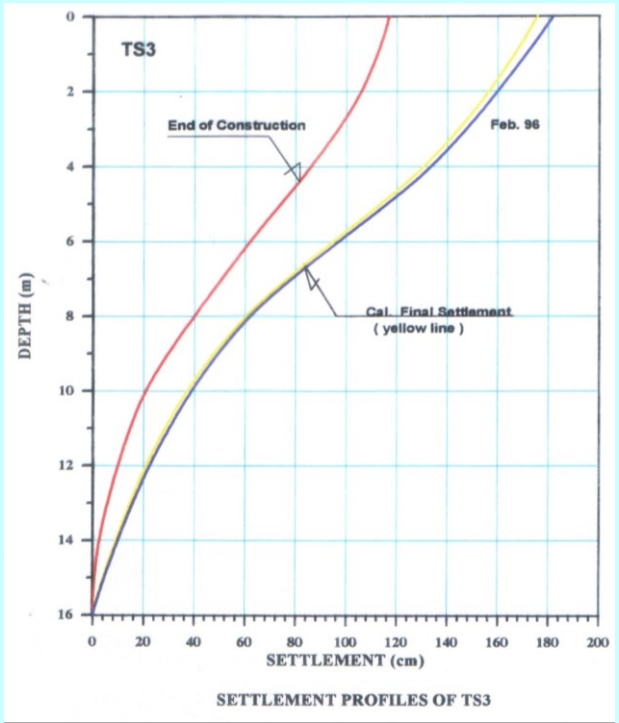
$\Delta\sigma_v'$: Increase of effective stress at calculated time
σ_c : Embankment load at calculated time
Δu_p : Excess pore pressure just after adding the additional load including the remaining pore pressure from the previous stage
S_c : Consolidation settlement at calculated time
S_{cf} : Final consolidation settlement at 72 kPa load=130 cm

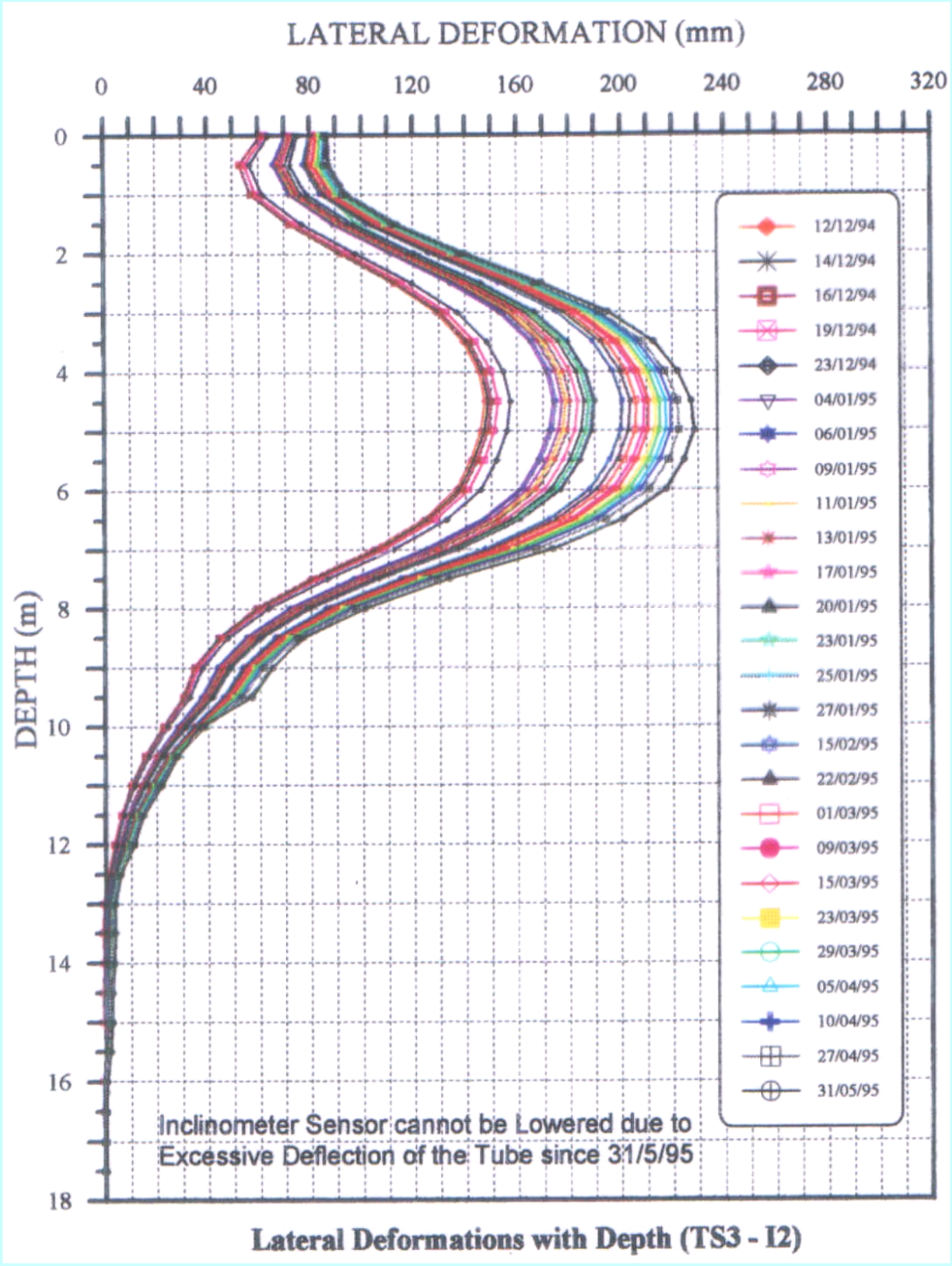
Fig. 4.18 Summary of Stability and Settlement Analyses for Embankment TS3 (with 1.0 m Drain Spacing)


Up to 15/02/96
HT. OF FILL (m)
Start of Construction: 22 April 1994
SETTLEMENT (cm)
SS-1
SS-2
SS-3
SS-4
SS-5
SS-6
SS-7
SS-8
SS-9
SS-11
SS-12
SS-13
SURFACE SETTLEMENTS (TS-3)
TIME (days)

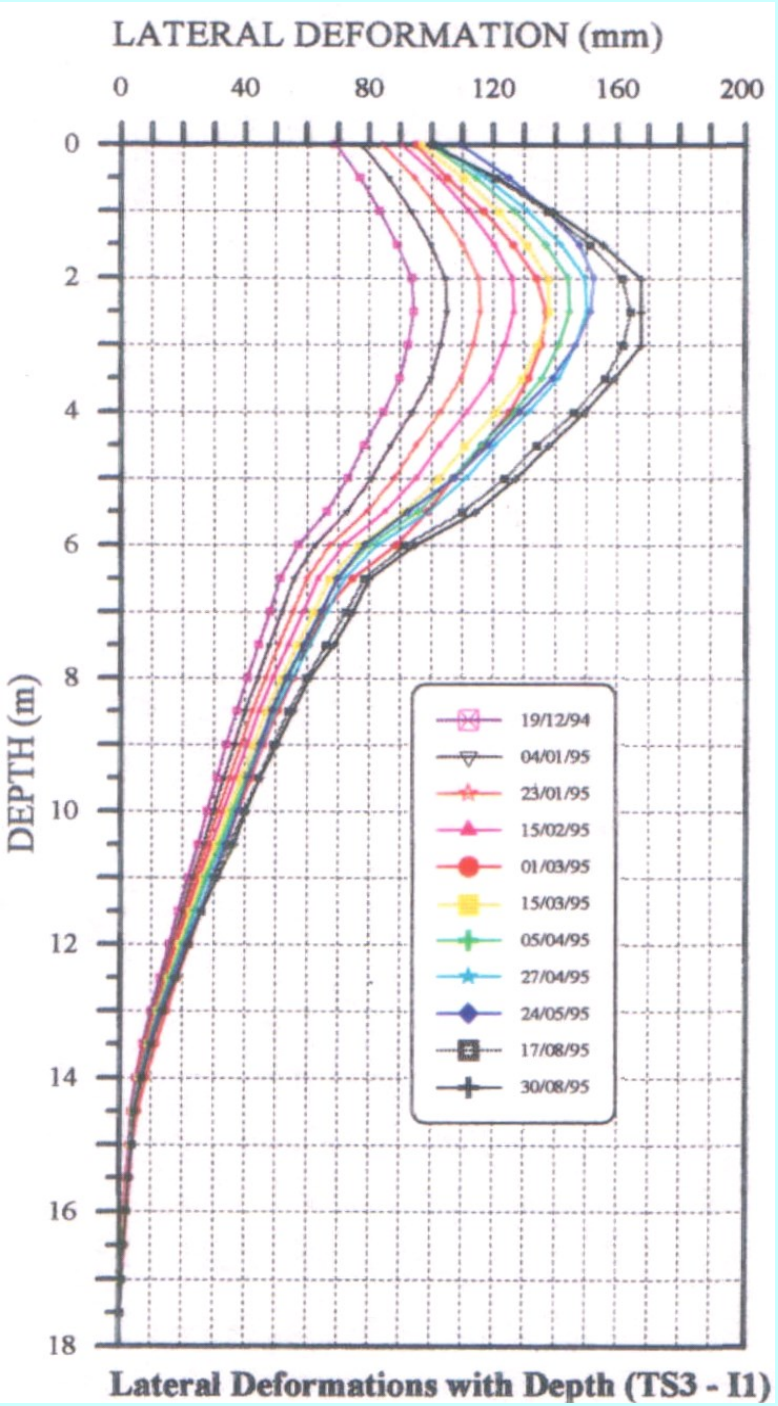

HT. OF FILL (m)
SETTLEMENT (cm)
Dotted lines : Measured
Solid lines : Calculated (1-D)
TS1
TS2
TS3
TIME (days)


Start of Construction: 23 May 1994
HT. OF FILL (m)
SETTLEMENT (cm)
12 m
8 m
2 m
Surface
SD-2
SD-8
SD-12
Surface
Measured : Dotted lines
Calculated (FEM) : Solid lines
TIME (days)

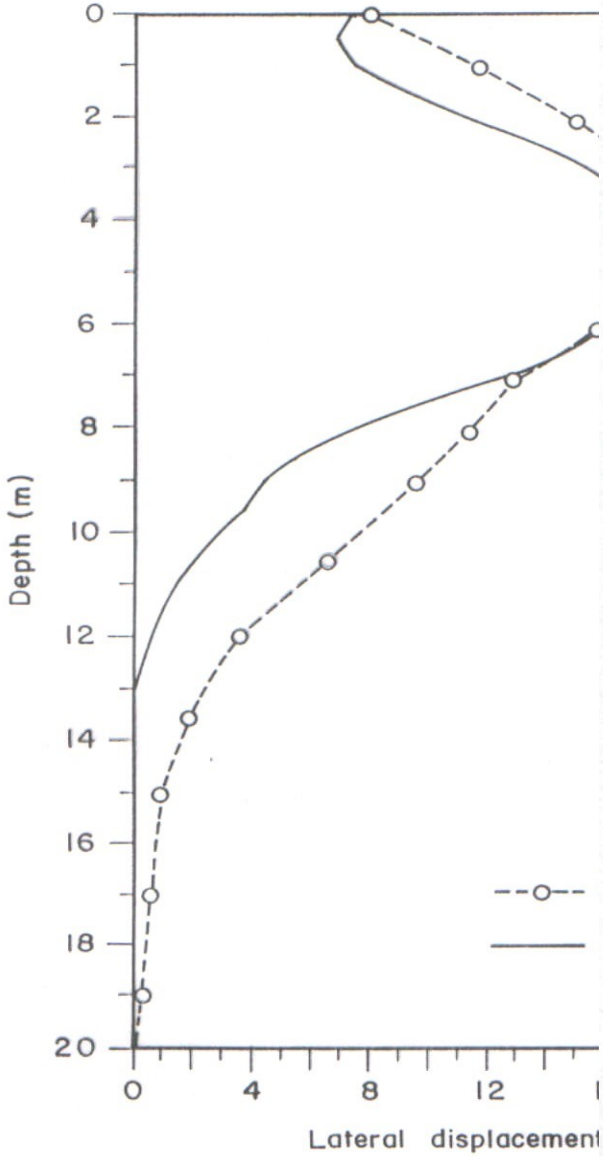

TS3
End of Construction
Feb. 96
Cal. Final Settlement
(yellow line)
DEPTH (m)
SETTLEMENT (cm)
SETTLEMENT PROFILES OF TS3



Lateral Deformations with Depth (TS3 - I2)



Lateral Deformations with Depth (TS3 - I1)



Comparison of Computed (FE
Lateral Deformations at the
for Embankment TS3

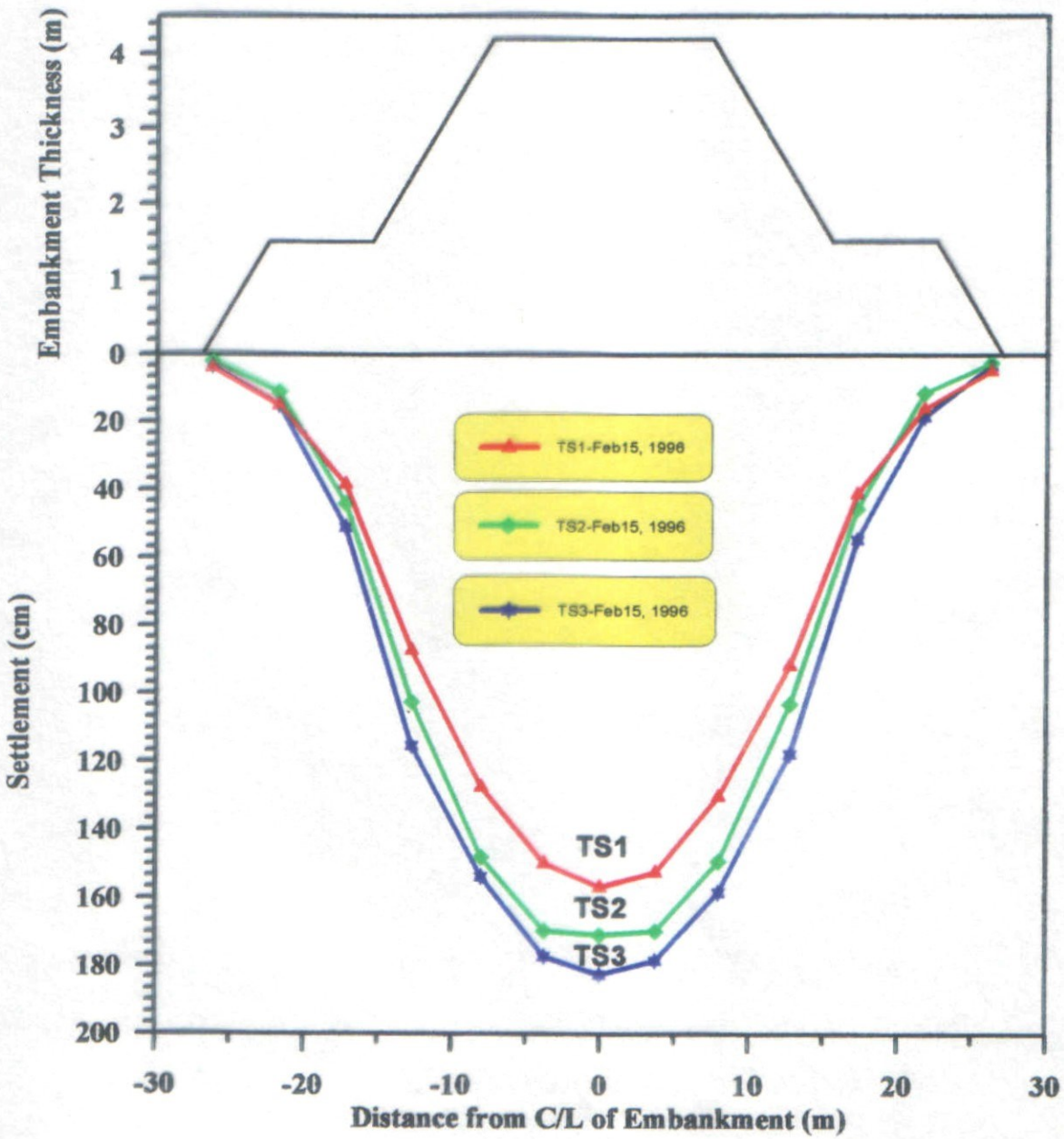
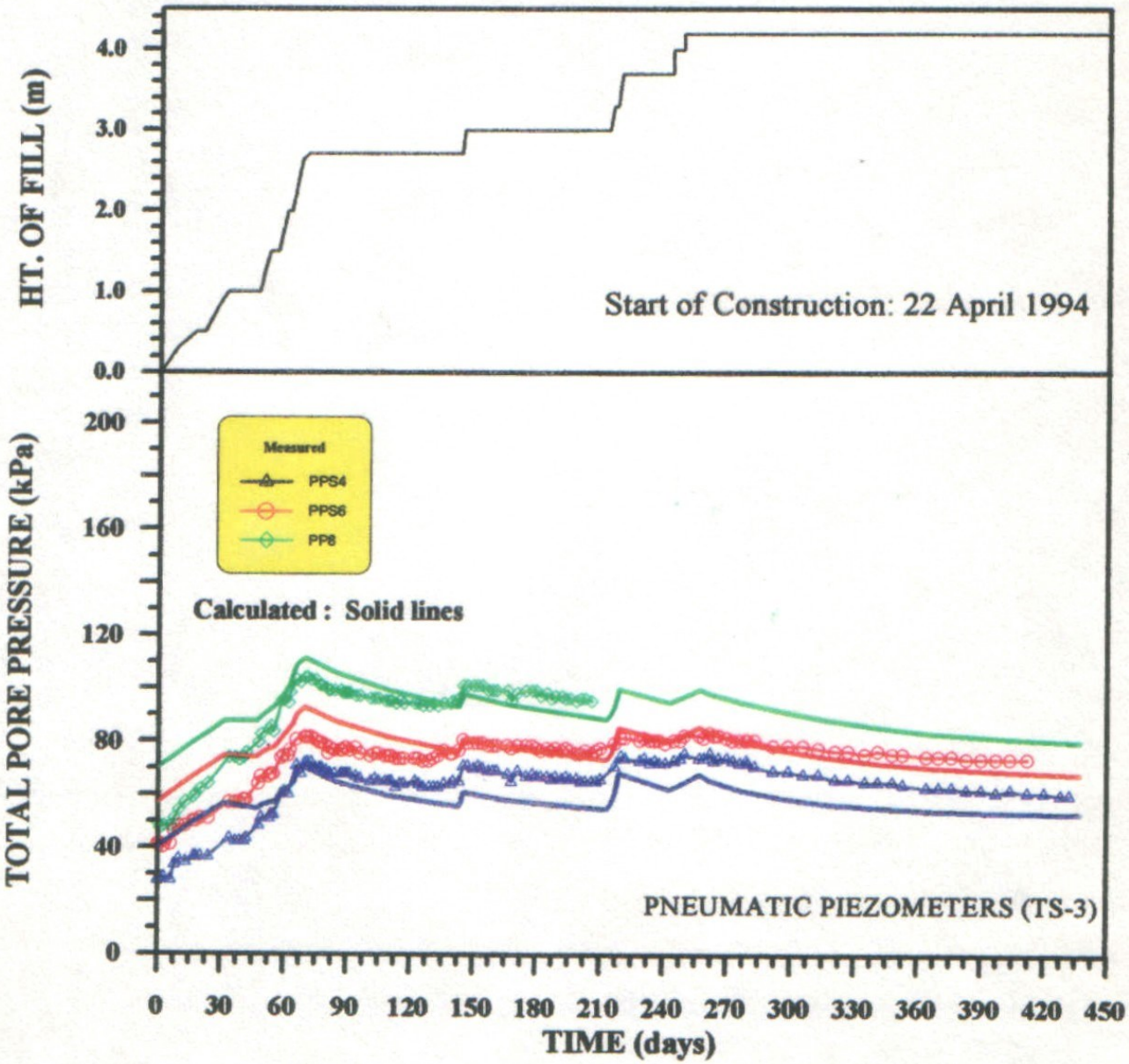


Fig. 6.7 Surface Settlement Profile across the Embankments Cross Sections (TS1, TS2 and TS3)



Comparison of FEM Calculated and Measured Total Pore Pressures at Different Depths 4 m, 6 m, and 8 m for TS3 Embankment

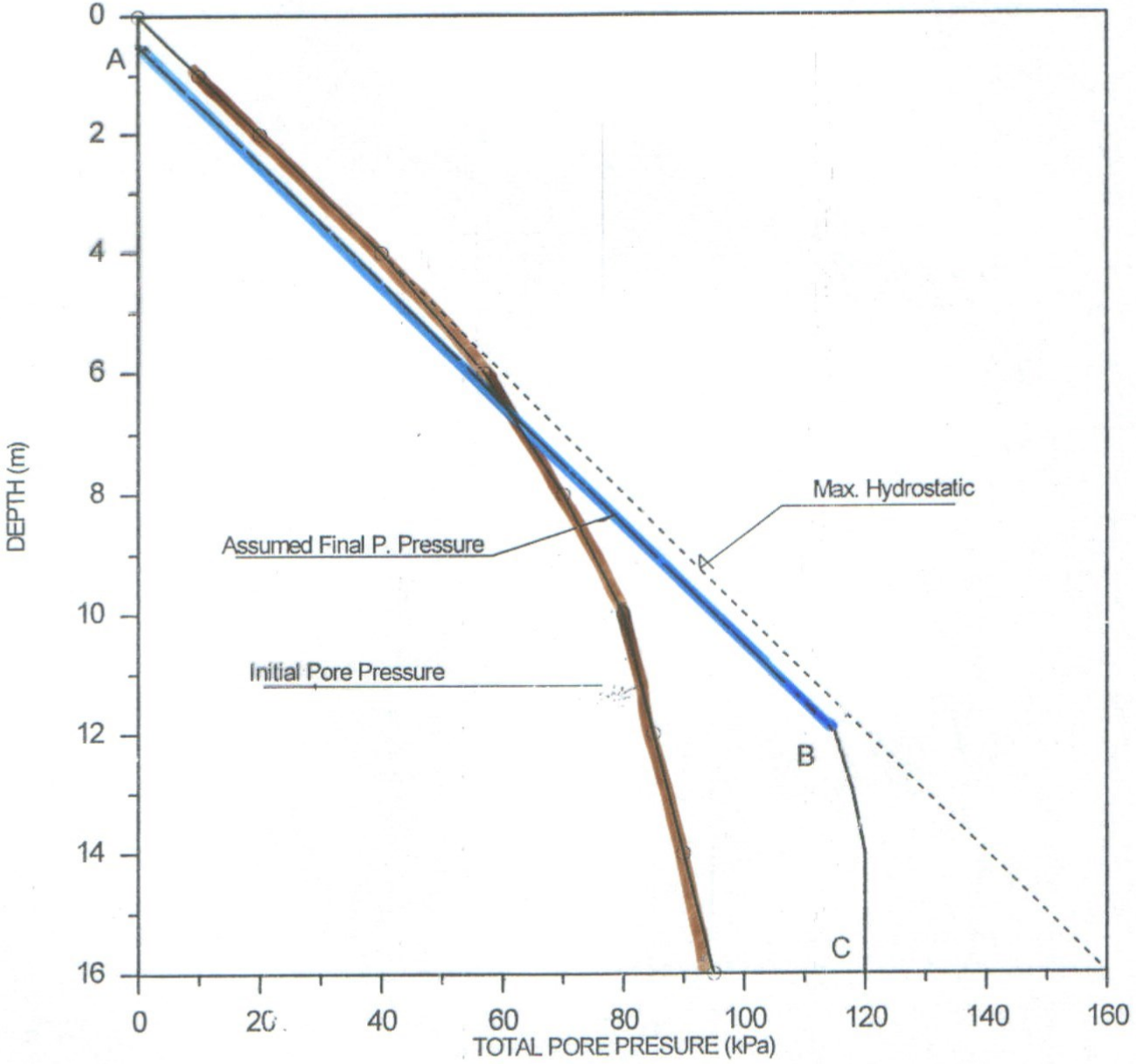
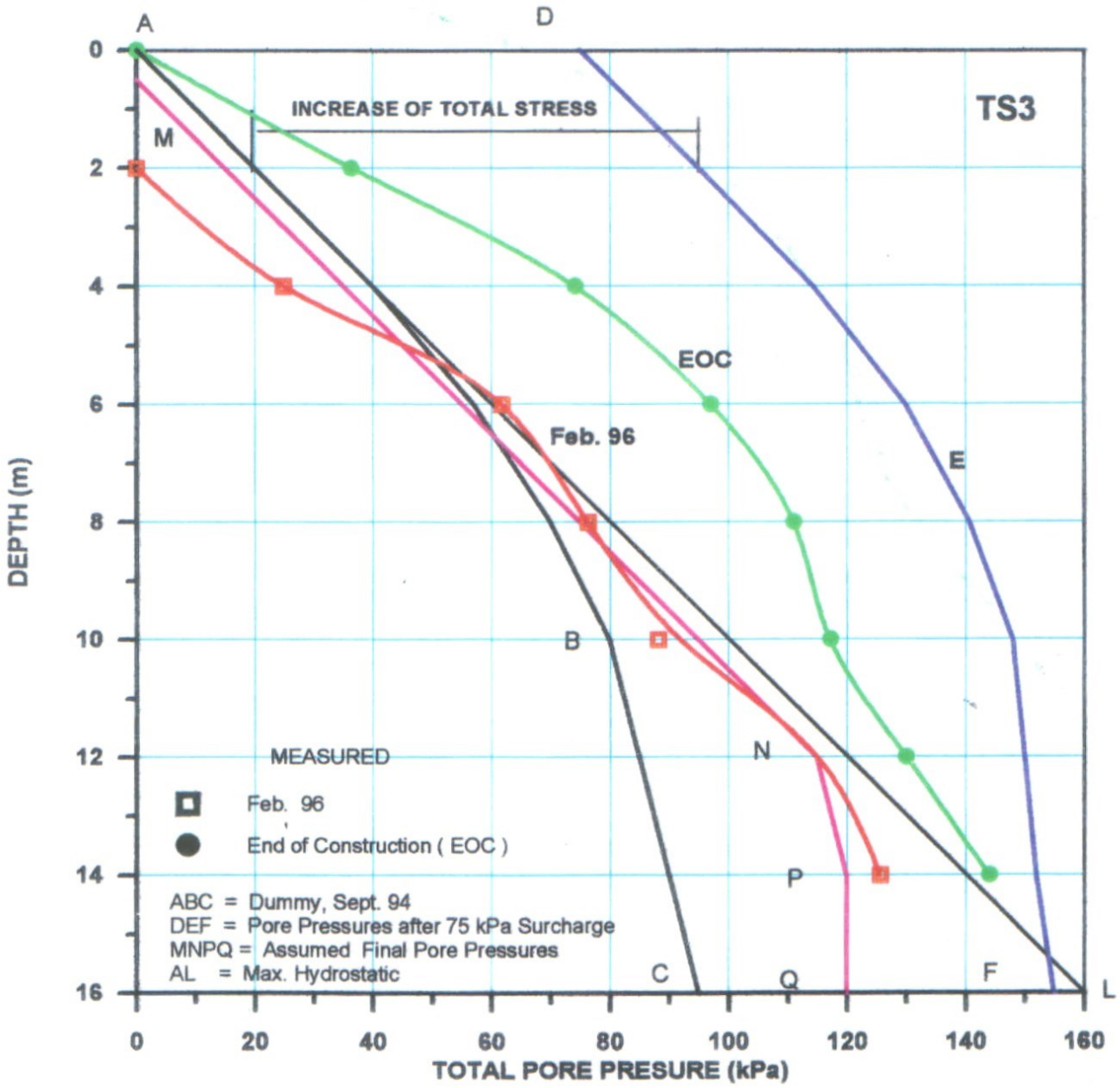
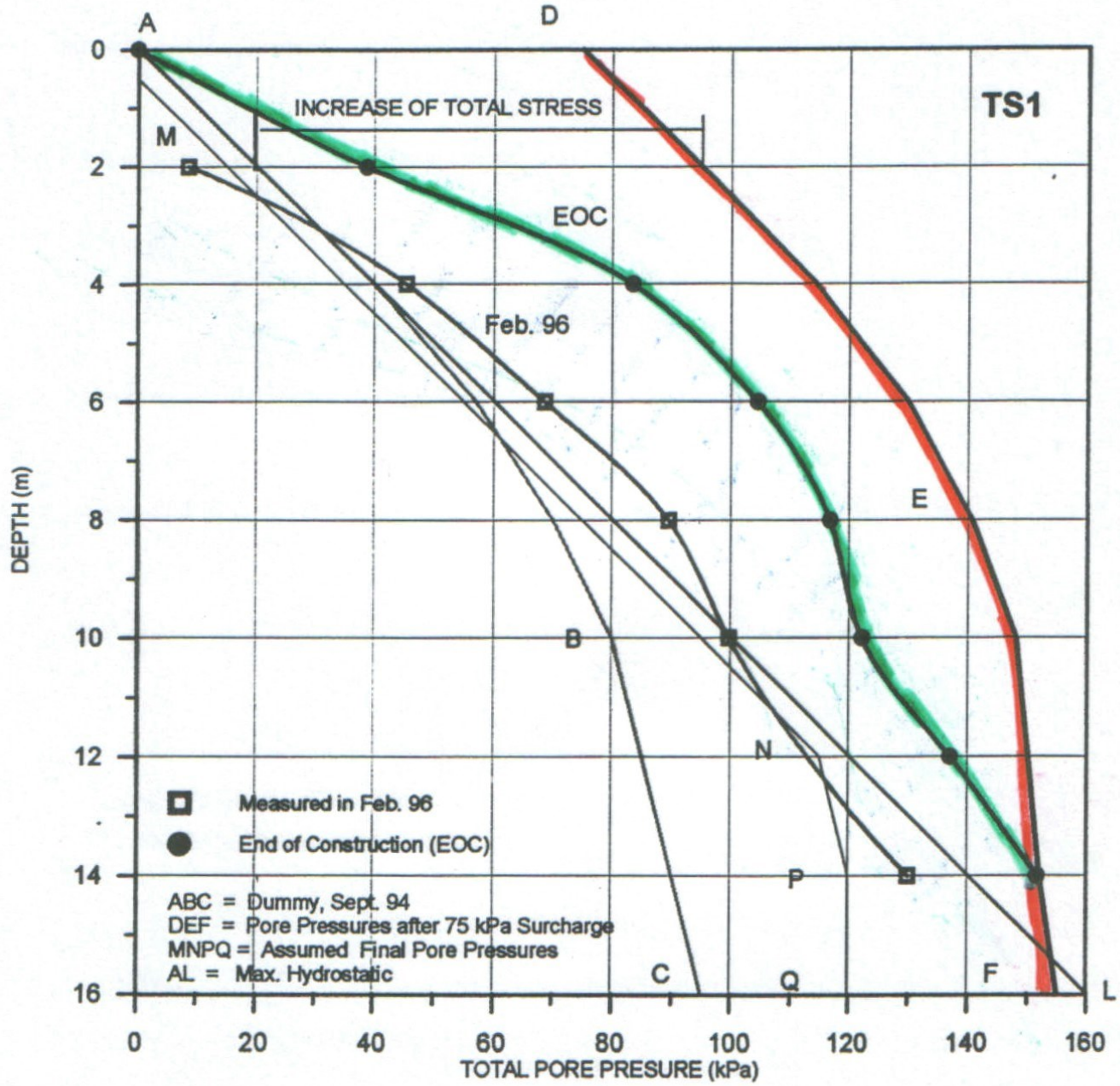


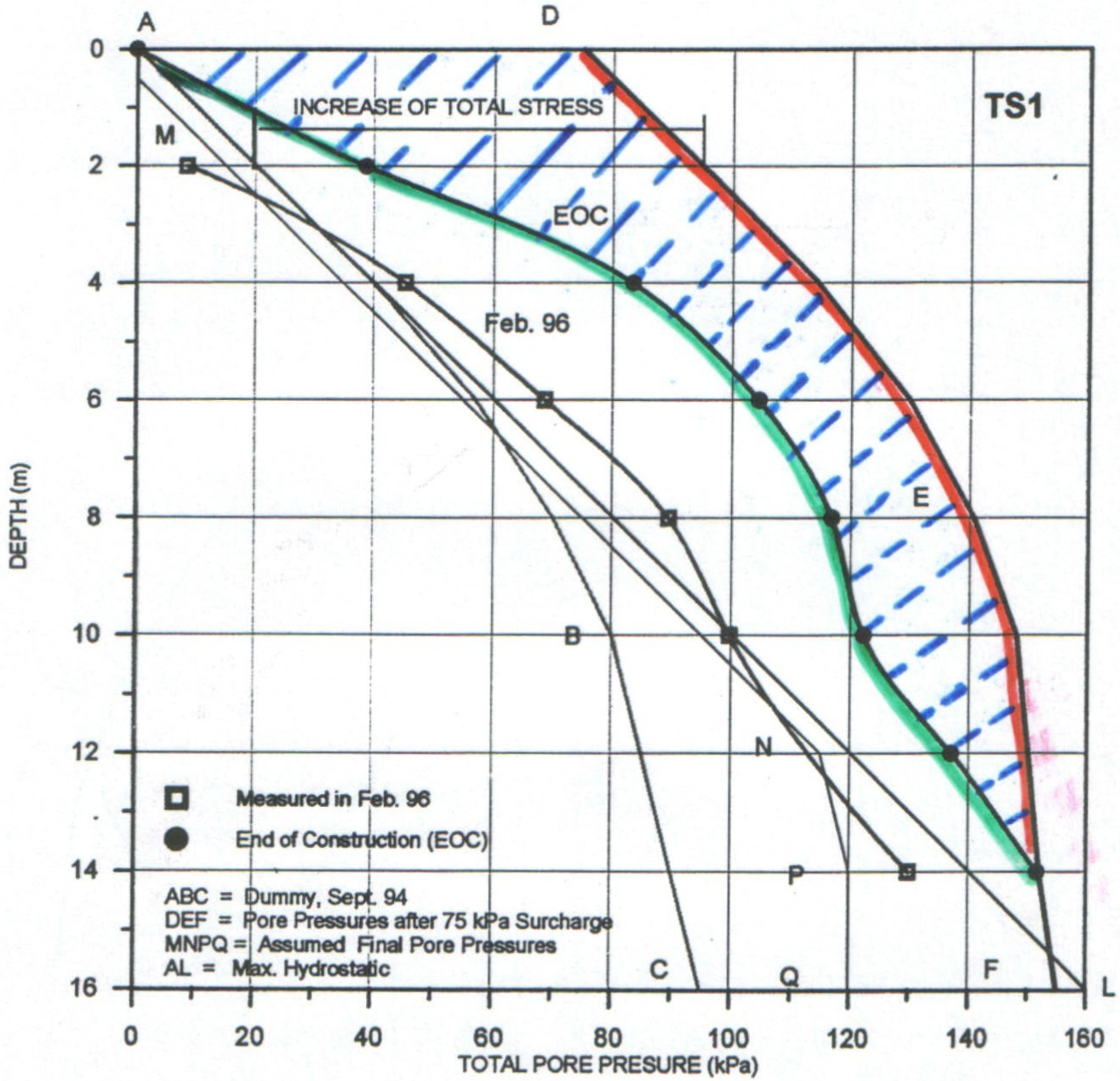
Fig. 6.26: Piezometric Drawdowns (Initial and Assumed Final Values)



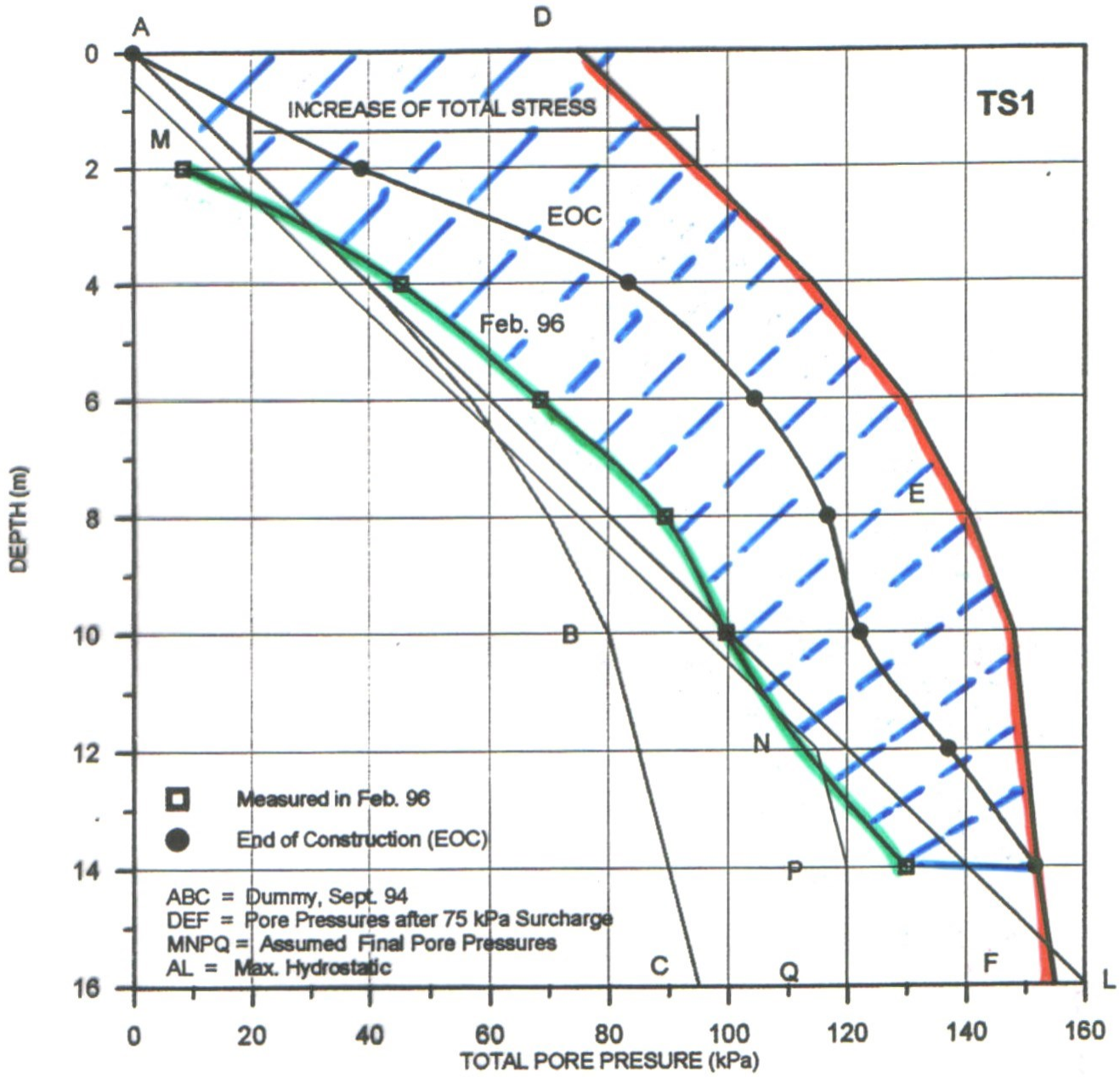
PORE PRESSURE PROFILES IN TS3



PORE PRESSURE PROFILE IN TS1



PORE PRESSURE PROFILE IN TS1



PORE PRESSURE PROFILE IN TS1

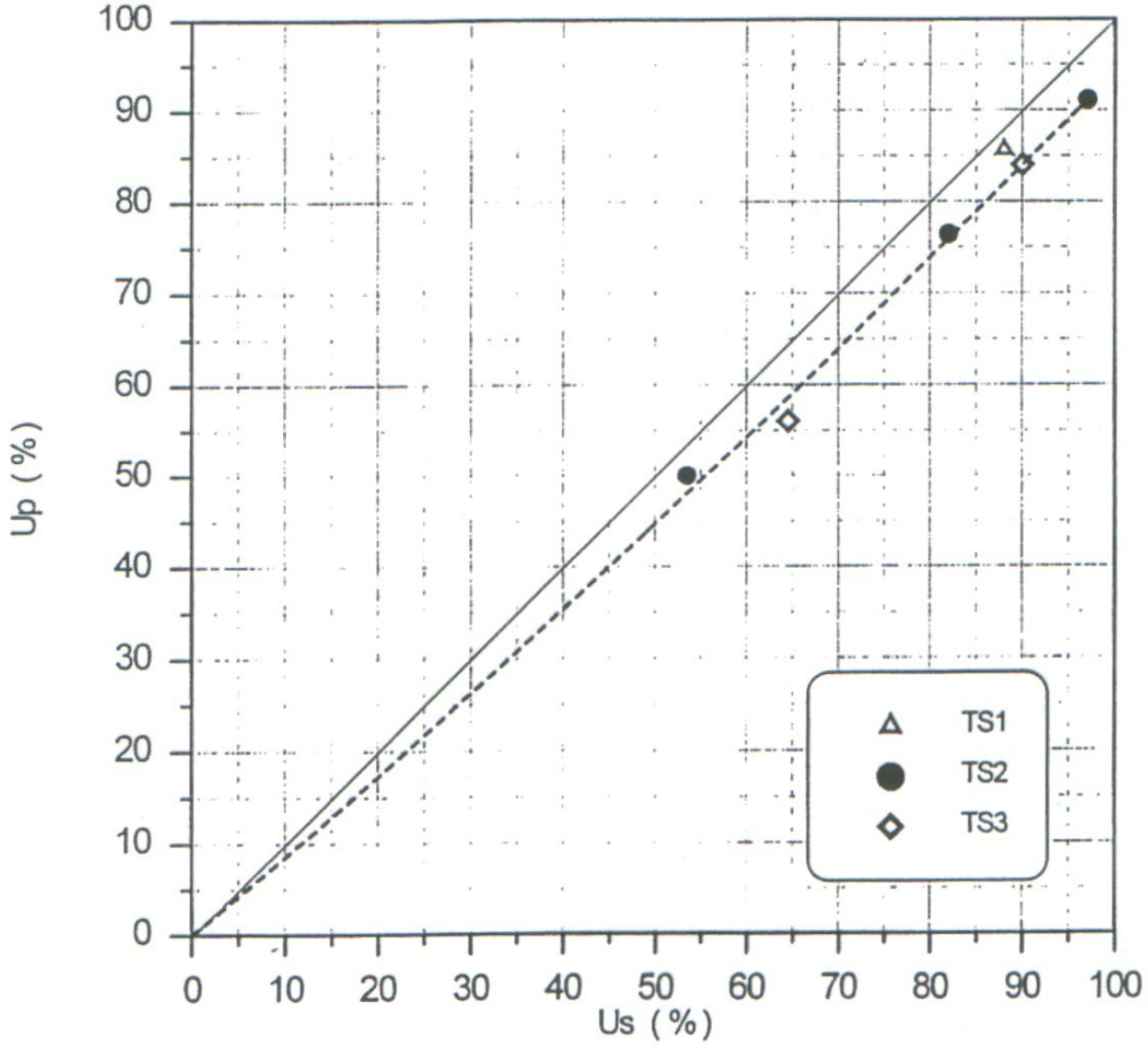
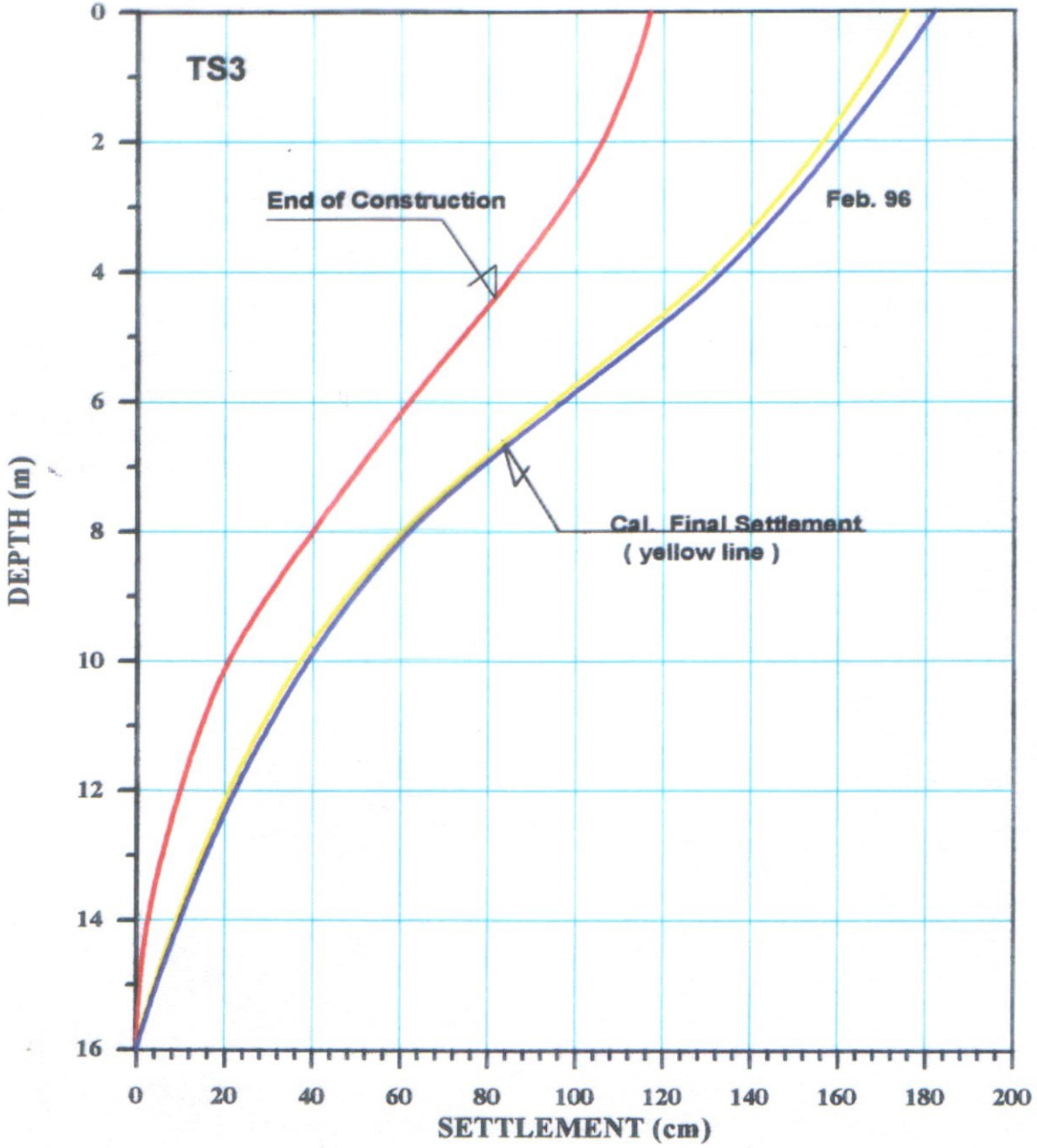
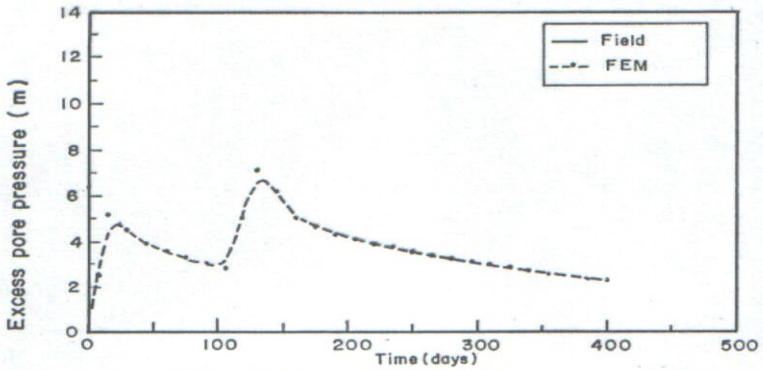


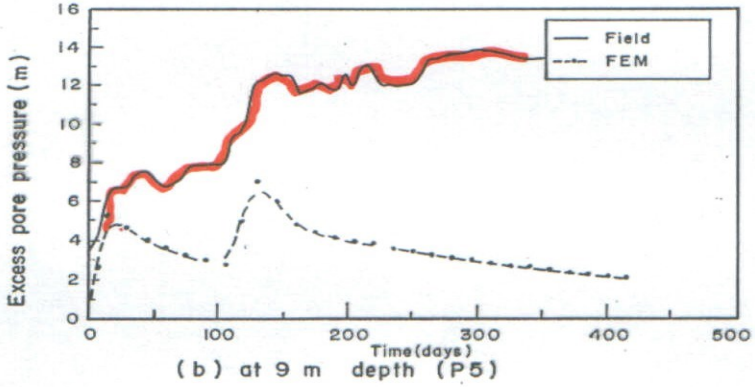
Fig. 14 Relation of Degree of Consolidation from Settlement (Us and Up)



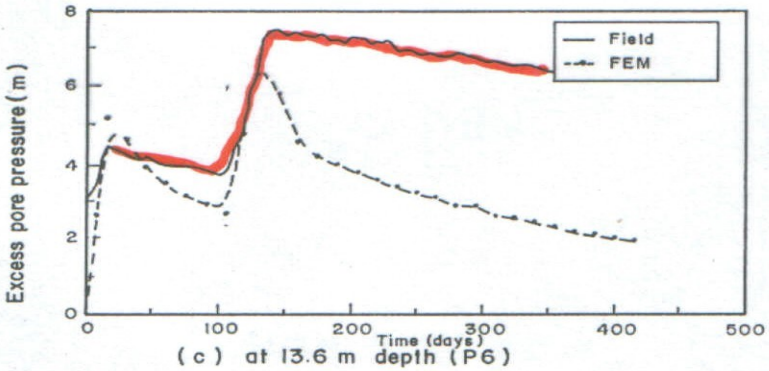
SETTLEMENT PROFILES OF TS3



(a) at 4.6 m depth (P4)

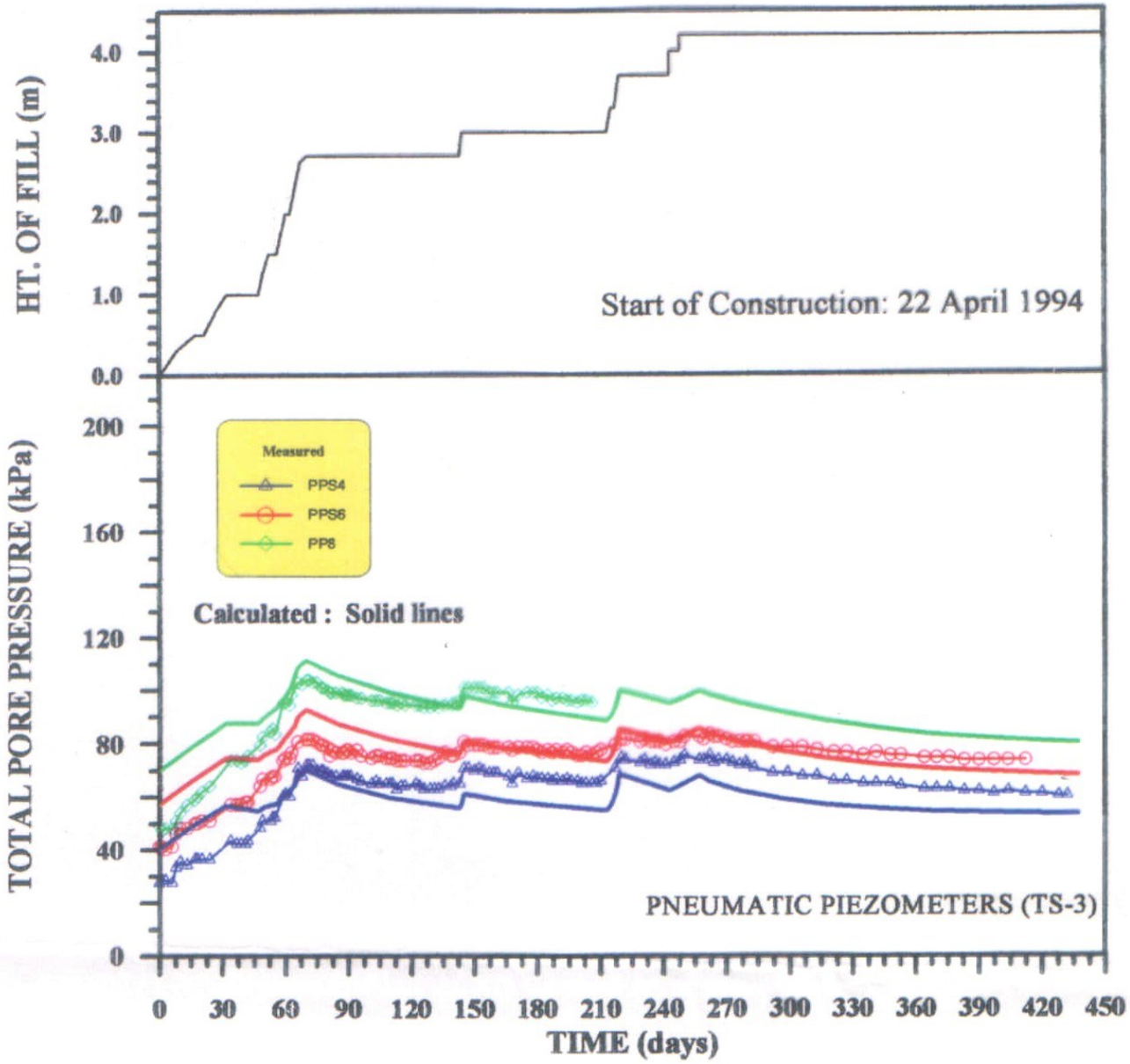


(b) at 9 m depth (P5)



(c) at 13.6 m depth (P6)

Center Line Excess Pore Pressure - Time Relationships of the Embankment with Vertical Drains



Comparison of FEM Calculated and Measured Total Pore Pressures at Different Depths 4 m, 6 m, and 8 m for TS3 Embankment

Figure 3.1 Back calculation of pore pressure dissipation TS3

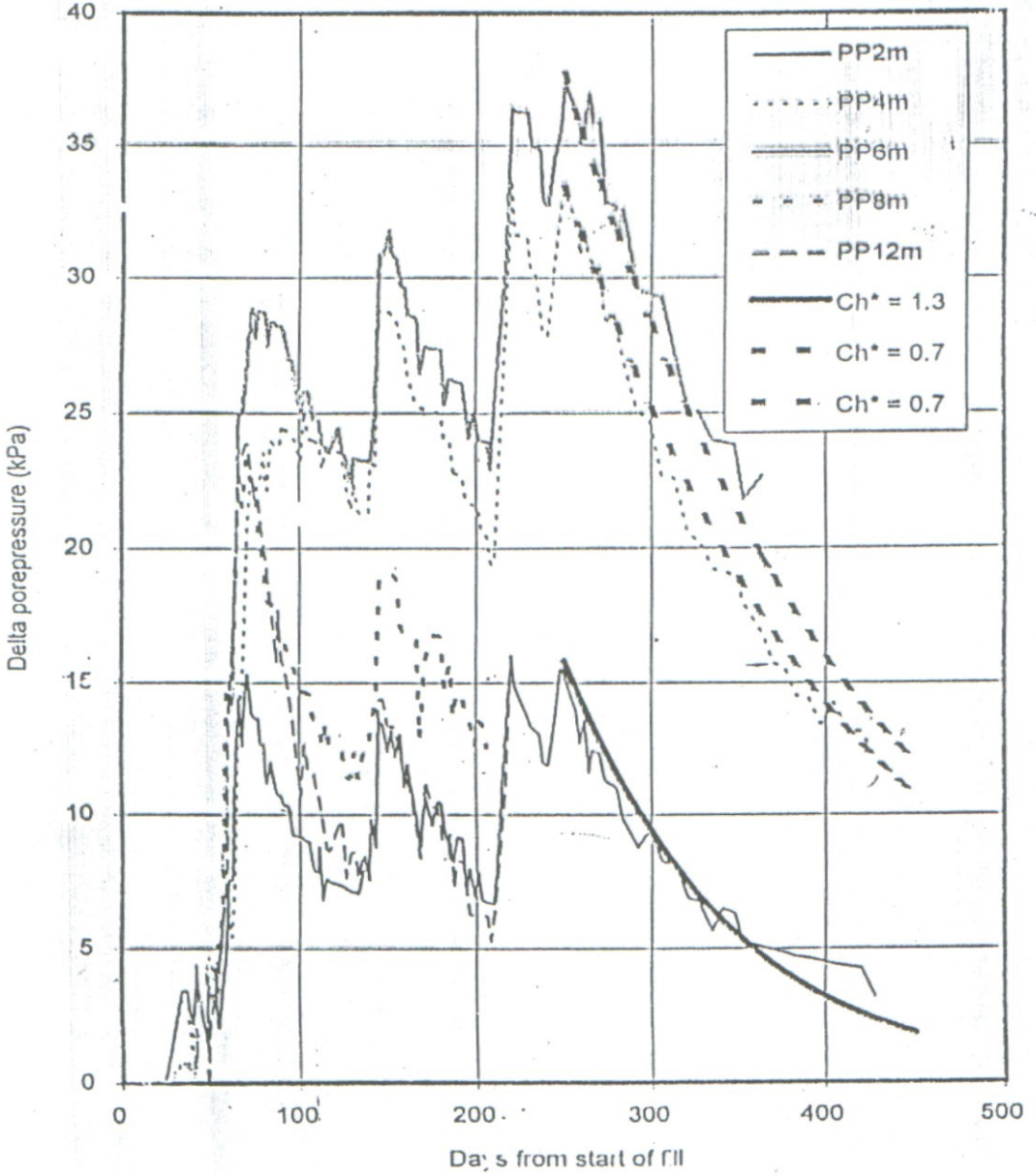
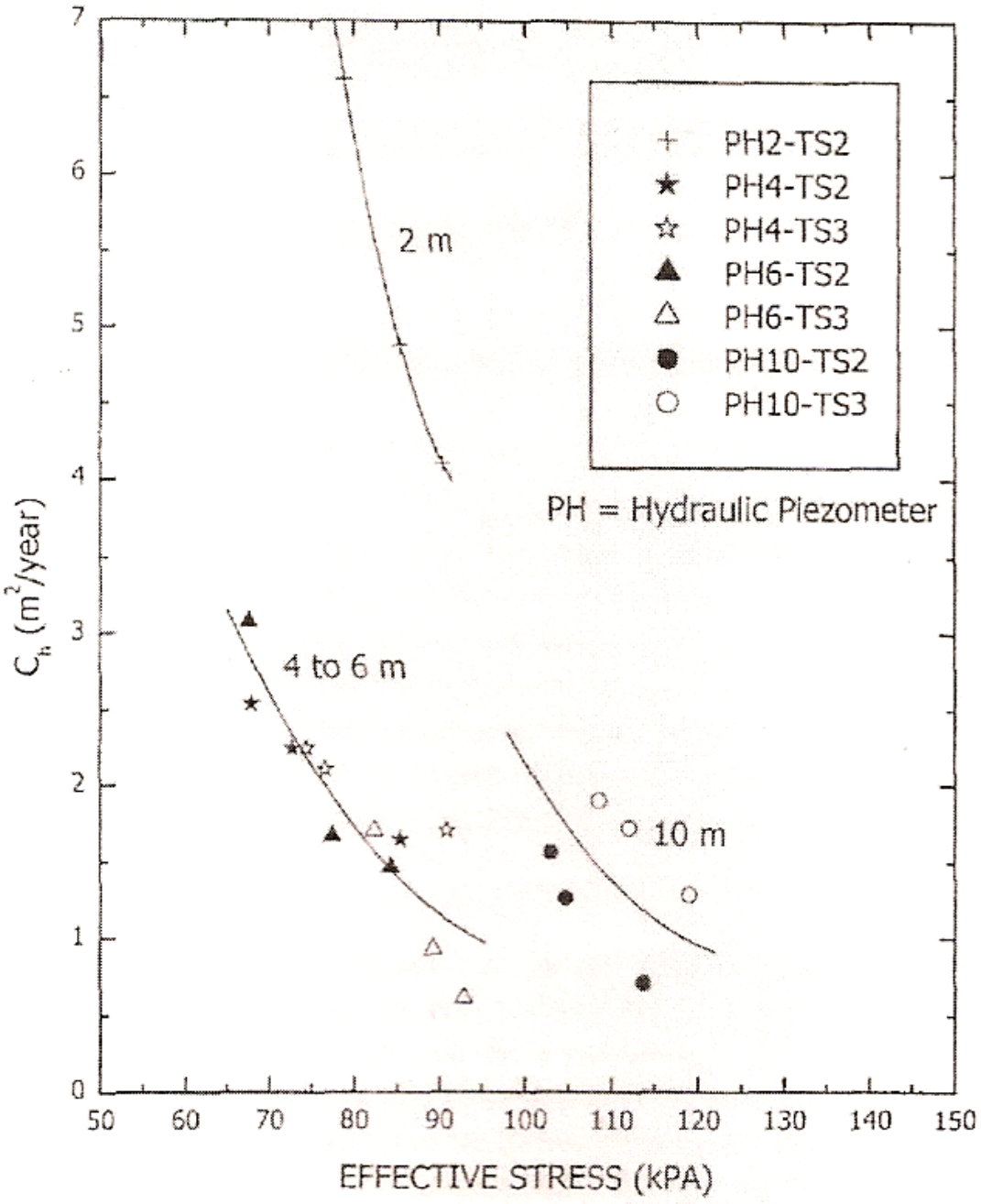
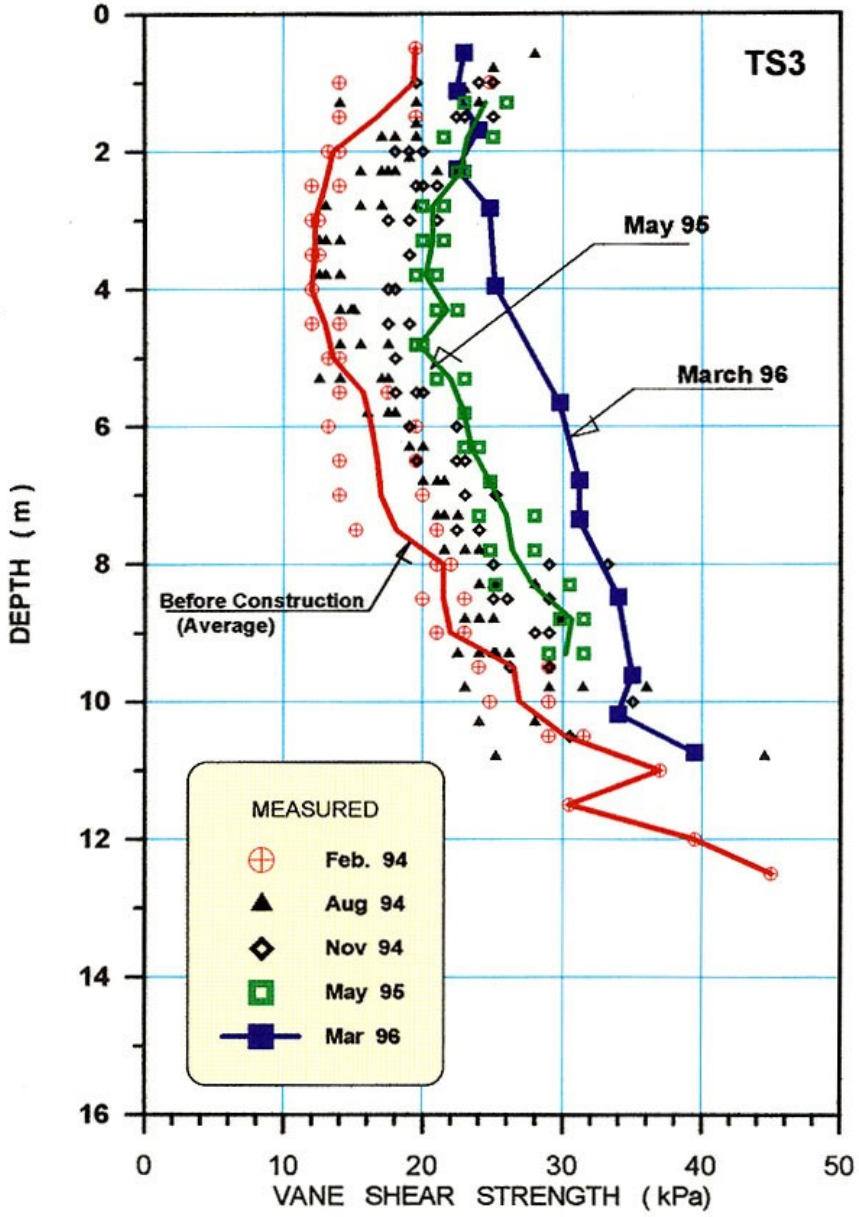
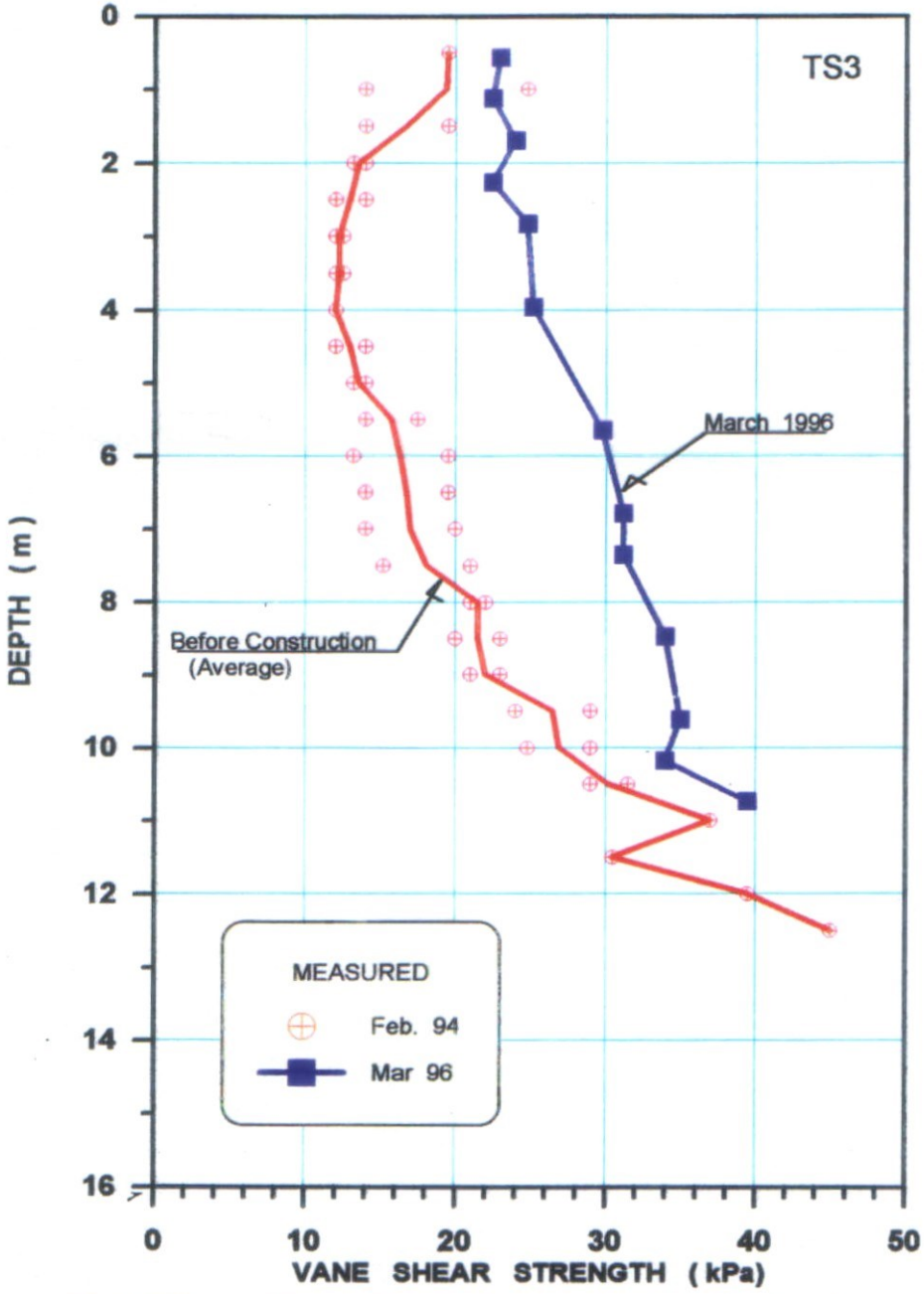


Fig. 15. Plot of back-calculated C_h values from the pore-pressure data versus effective stress.

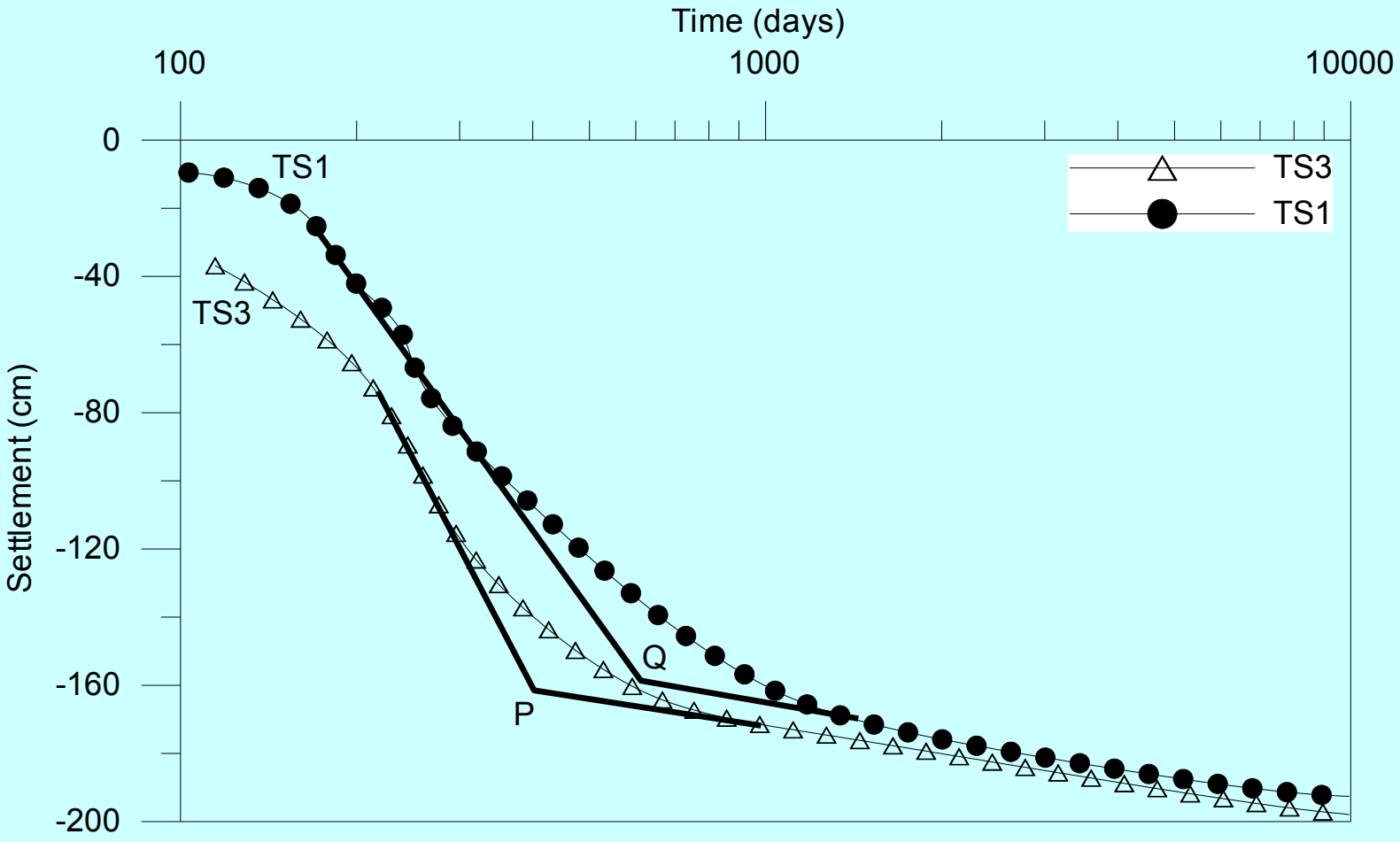


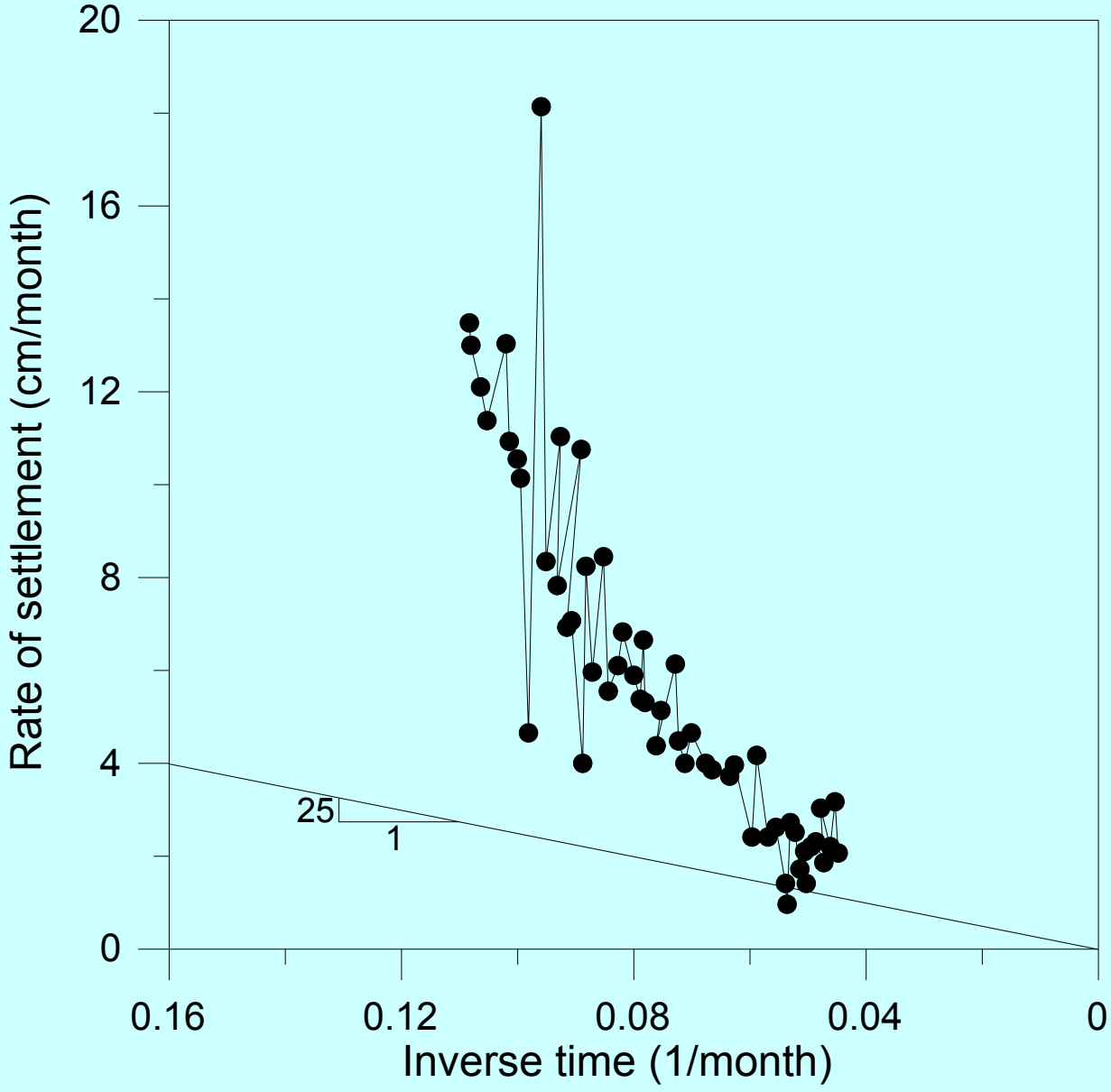


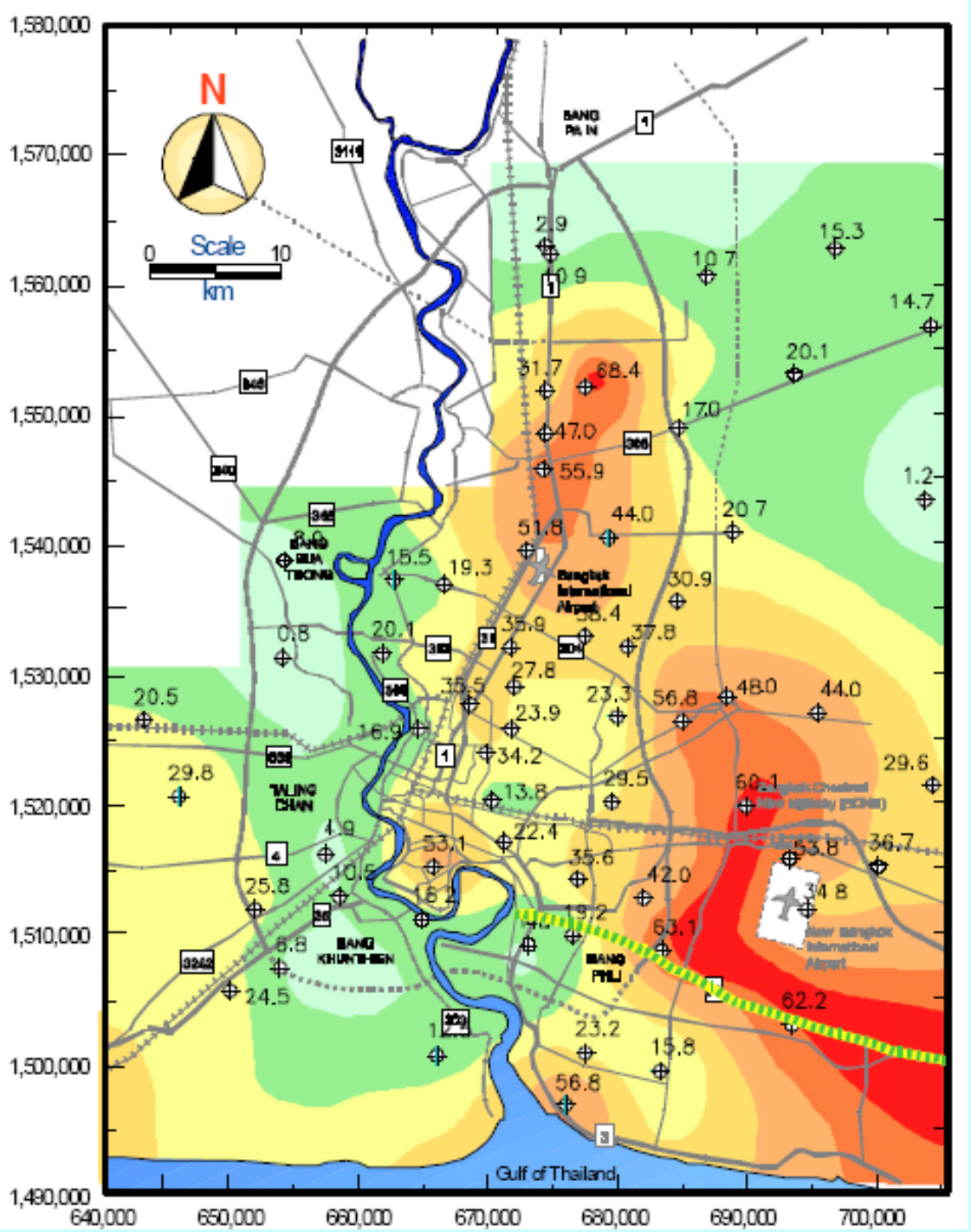
Field Vane Shear Strength Measured in Embankment TS3



Field Vane Shear Strength Measured in Embankment TS3


Time (days)
100
1000
10000
0
-40
-80
-120
-160
-200
Settlement (cm)
TS1
TS3
TS3
TS1
Q
P


20
16
12
8
4
0
Rate of settlement (cm/month)
25
1
0.16
0.12
0.08
0.04
0
Inverse time (1/month)



Rate of Settlement (mm/year)

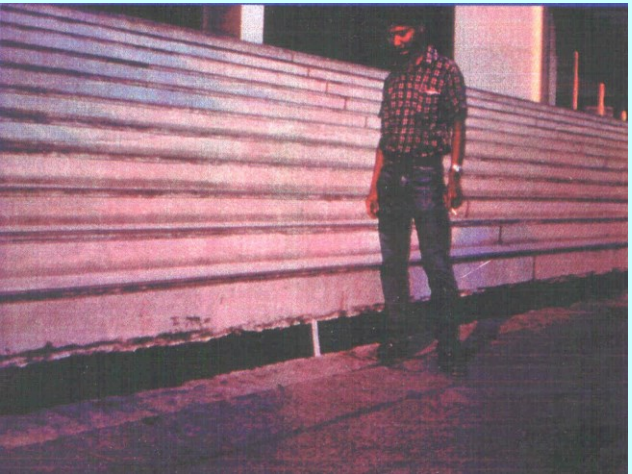
- >60
- 50-60
- 40-50
- 30-40
- 20-30
- 10-20
- 0-10

LEGEND

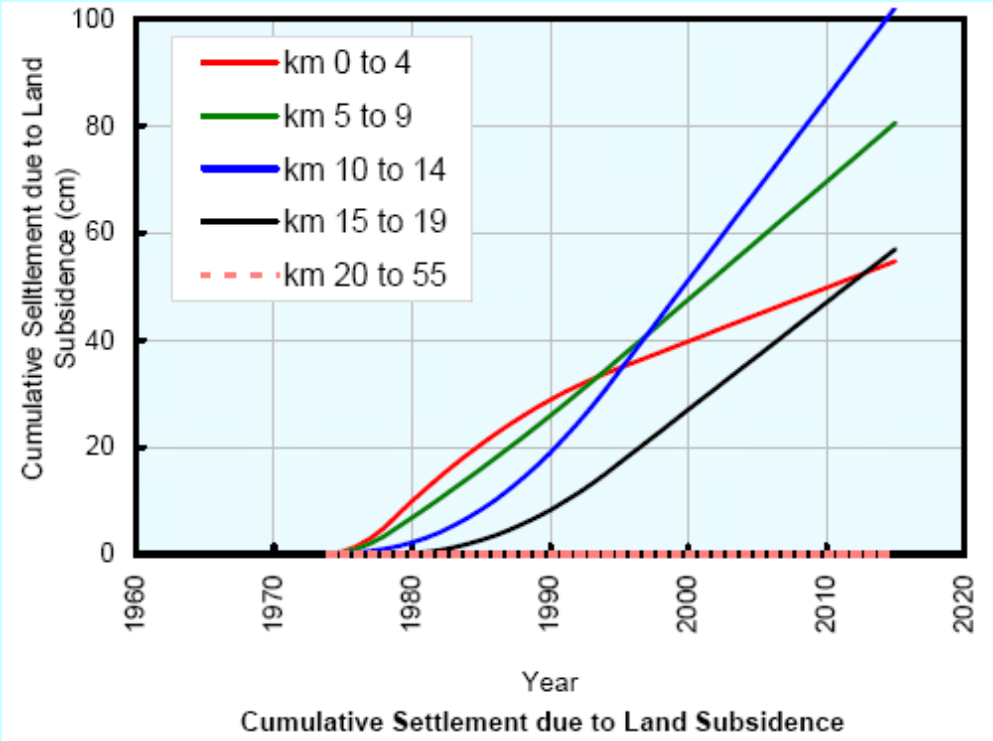
- Project Route
- Existing Major Roads
- Highway under Construction
- Future Highway
- Station for Settlement Monitoring

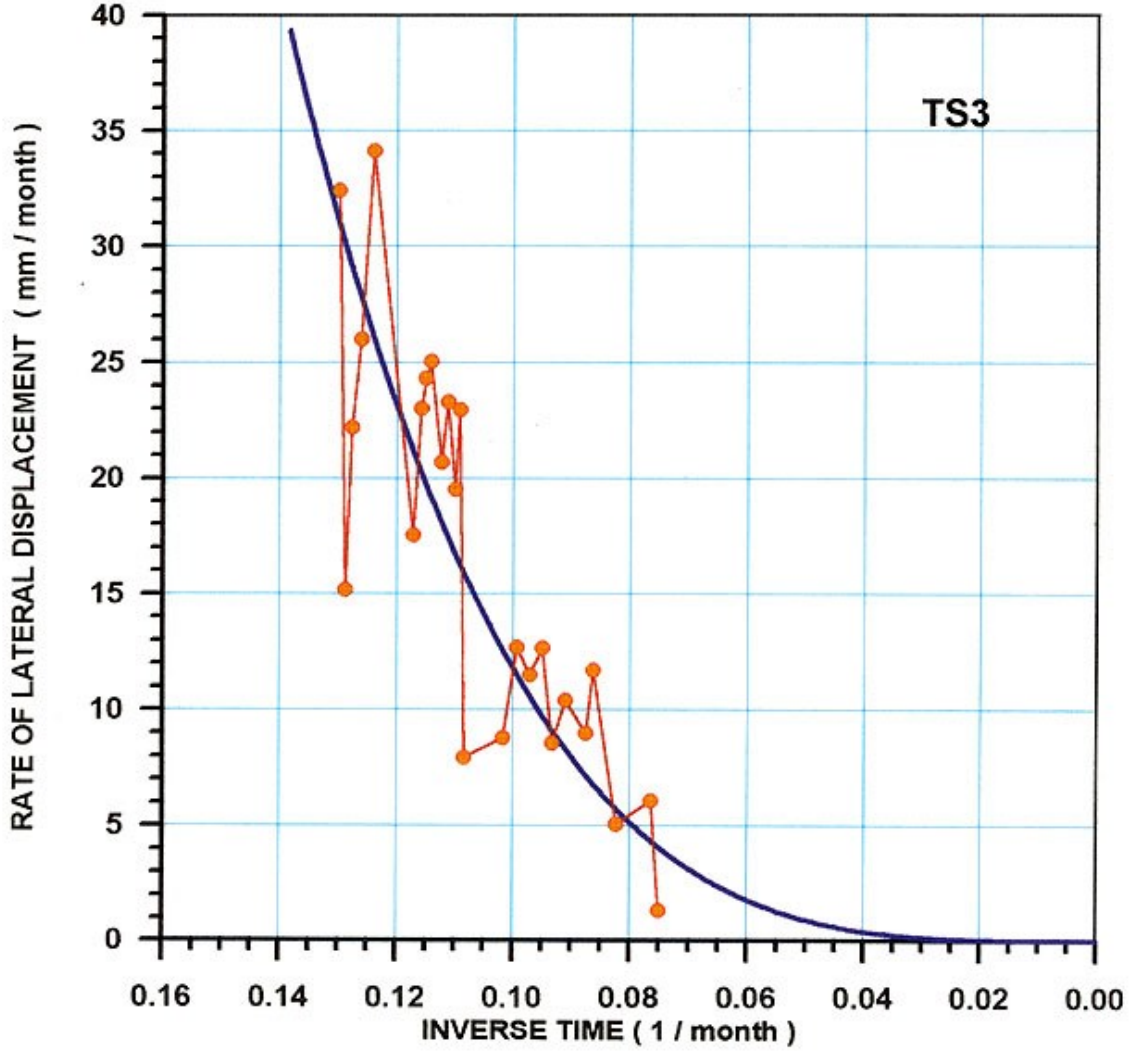
Note:

- Settlement Data from 1995 to 1996 were collected from the Department of Mineral Resources (DMR).
- Data are filtered by the Consultant

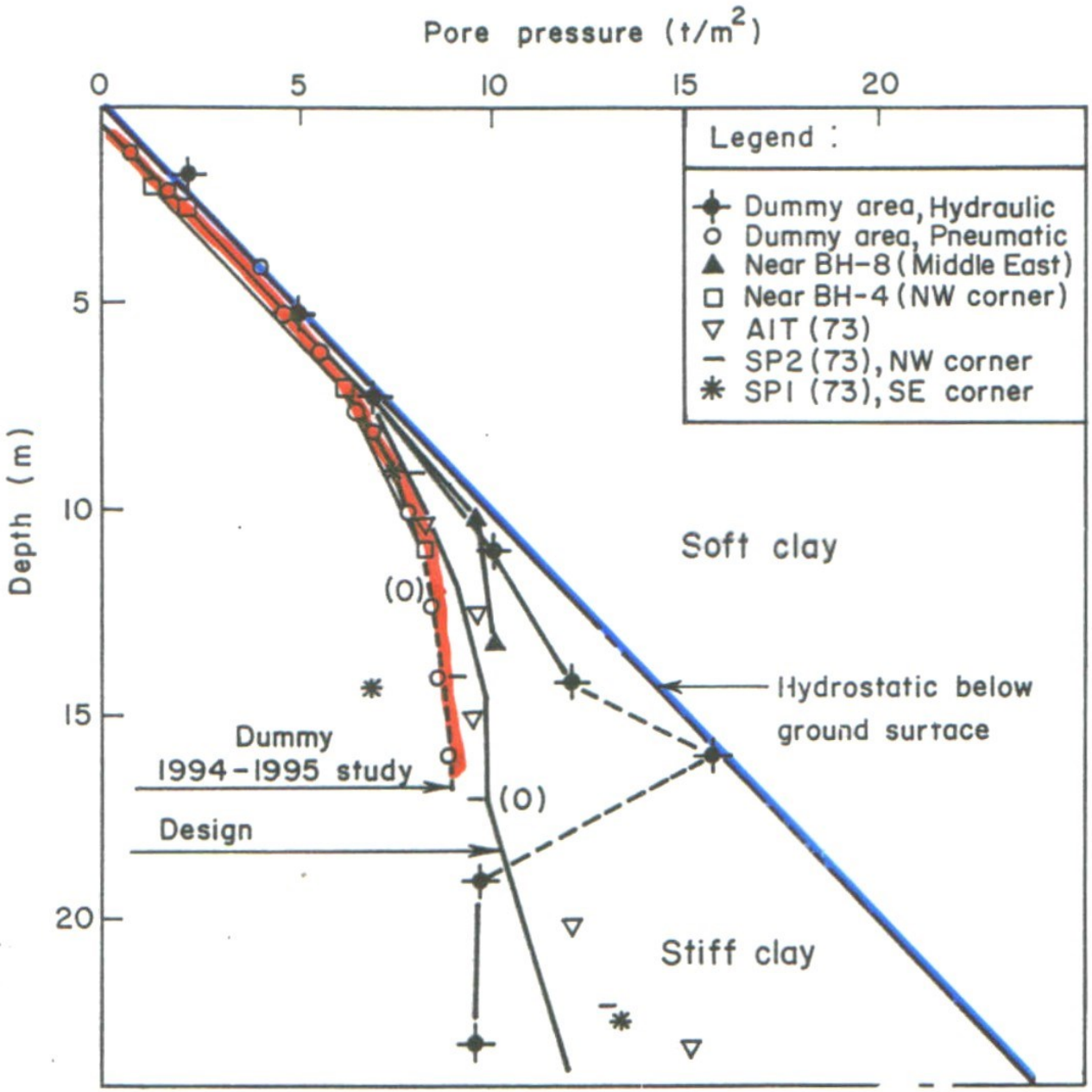



Bang Khen
Nong Chok
Nonthaburi
304
Minburi
BANGKOK
Bangkapi
Phra Khanong
Lat Kabang
Bangna
BANGNA-BANG PLI-BANG PAKONG EXPRESSWAY
303
Bang Plee
Bang Bo
Samut Prakan
3
34
Approx. Scale (km)
0 5 10
N
CHONBURI

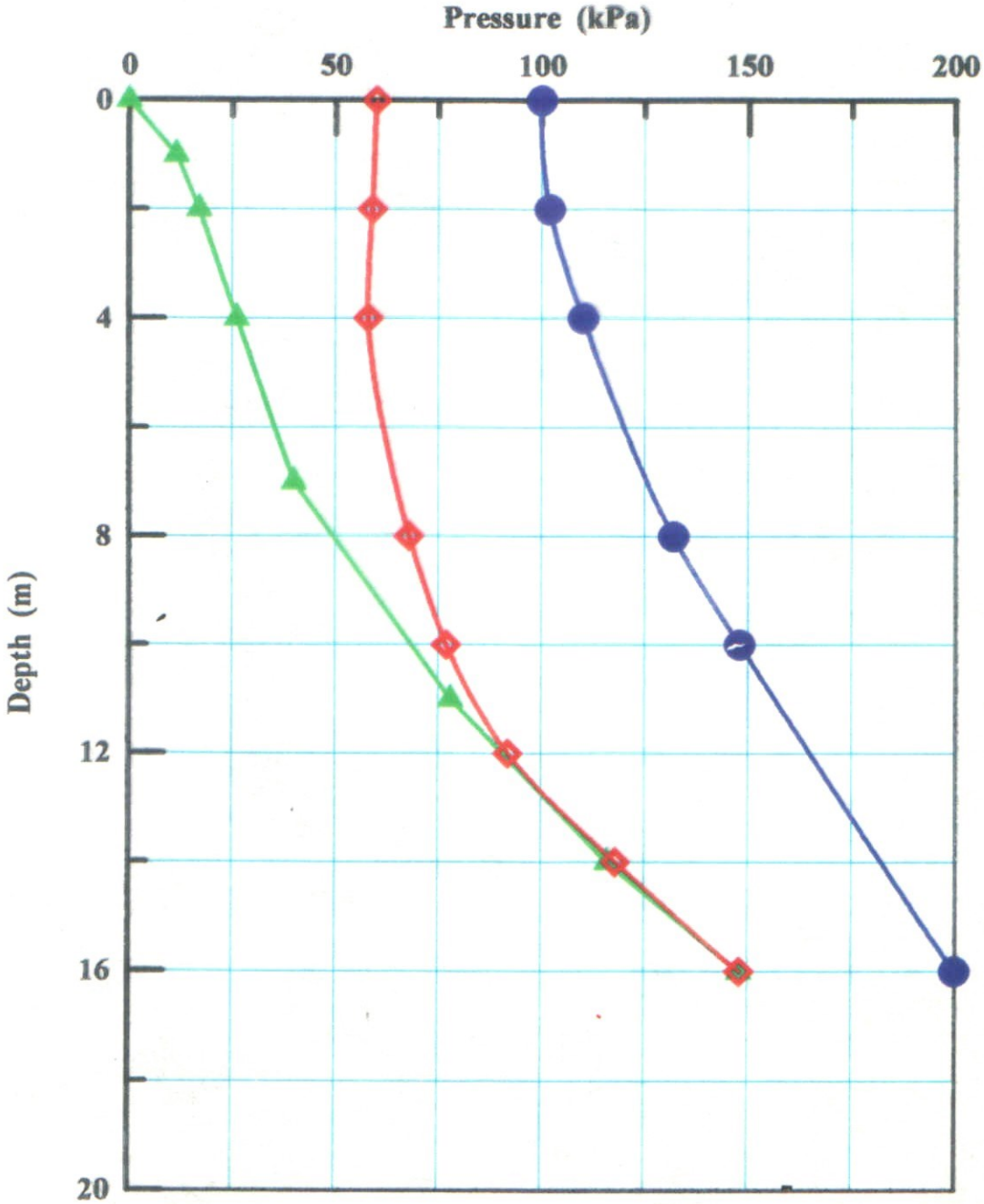

km 0 to 4
km 5 to 9
km 10 to 14
km 15 to 19
km 20 to 55
Cumulative Sellitlement due to Land Subsidence (cm)
0
20
40
60
80
100
1960
1970
1980
1990
2000
2010
2020
Year
Cumulative Settlement due to Land Subsidence



RATE OF LATERAL DISPLACEMENT vs INVERSE TIME PLOT FOR TS3



Variation of Piezometric Pressures with Depth


Pressure (kPa)
0
50
100
150
200
Depth (m)
0
4
8
12
16
20

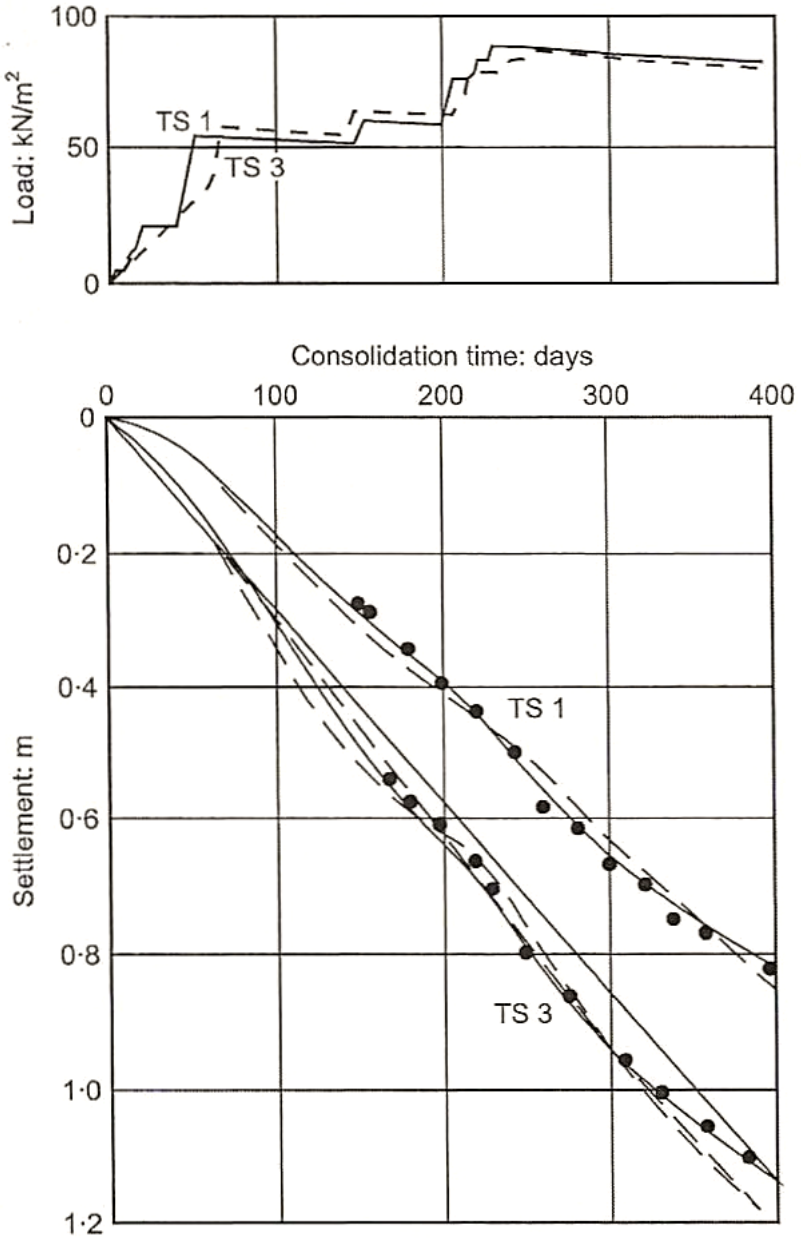


Fig. 7. Test areas at Nong Ngu Hao, Bangkok: measured and calculated settlements and loading conditions for TS 1 and TS 3; broken lines represent equation (1), full lines represent equation (4)

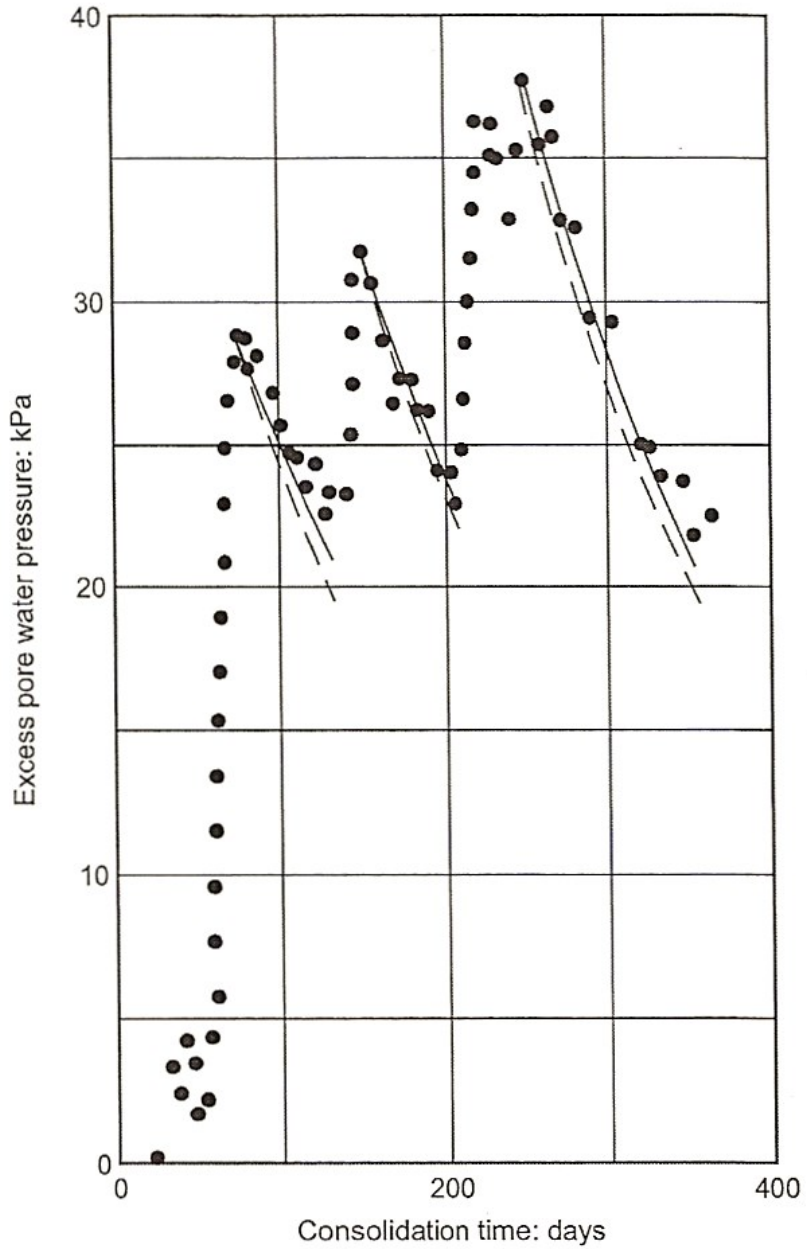


Fig. 8. Test areas at Nong Ngu Hao, Bangkok: measured and calculated excess pore pressure dissipation in test area TS 3; full lines represent equation (4), broken line equation (1)

1. Selection of PVD
2. Construction in the rainy season under flooded conditions
3. Stability of the test embankments
4. Have to really prove that the settlement is due to consolidation and not from undrained yielding without any volume change

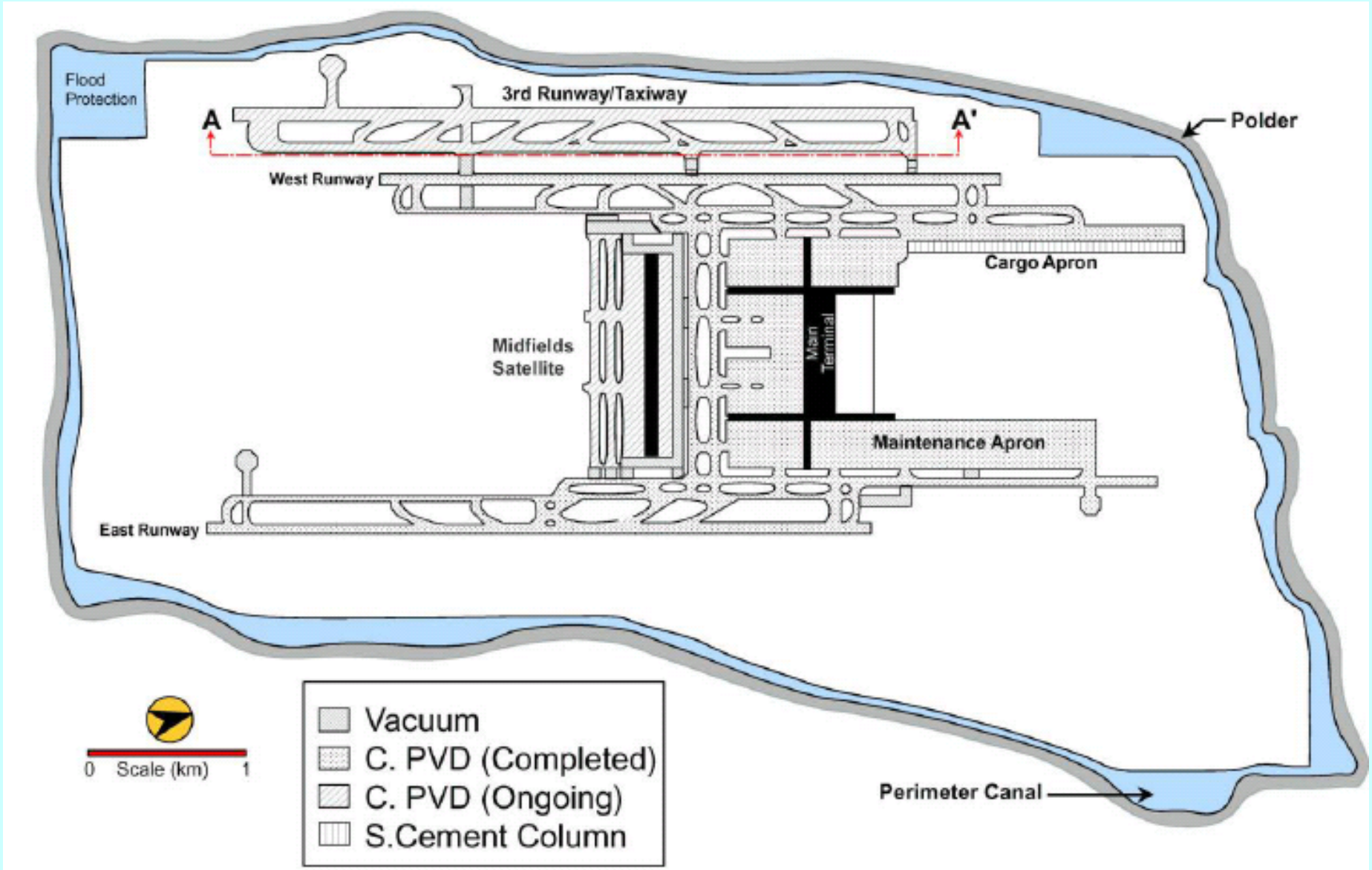
5. The piezometric draw-down due to subsidence made the computation of settlement from pore pressure dissipation difficult.
6. Computations need to convince that the degree of consolidation estimated from pore pressure dissipation and settlement measurements are comparable.
7. Undrained strength measurements should reflect the strength increase due to water content reductions.
8. Reason for continuing settlements.

The performance of the test embankment was satisfactory in the sense

1. **The consolidation settlements were 90 pc of the total settlement.**
2. **The degree of consolidation computed from the pore pressure dissipation is of the same order as those computed from settlement measurements.**

3. The strength increase was the same as the strength increase corresponding to water content reductions.

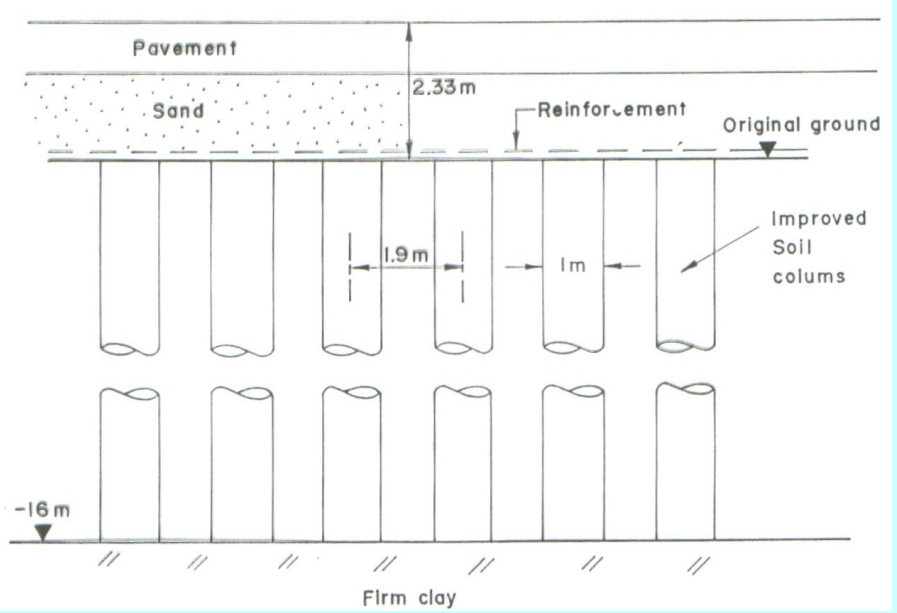
4. The undrained creep and secondary consolidation can be erased with the partial removal of surcharge.

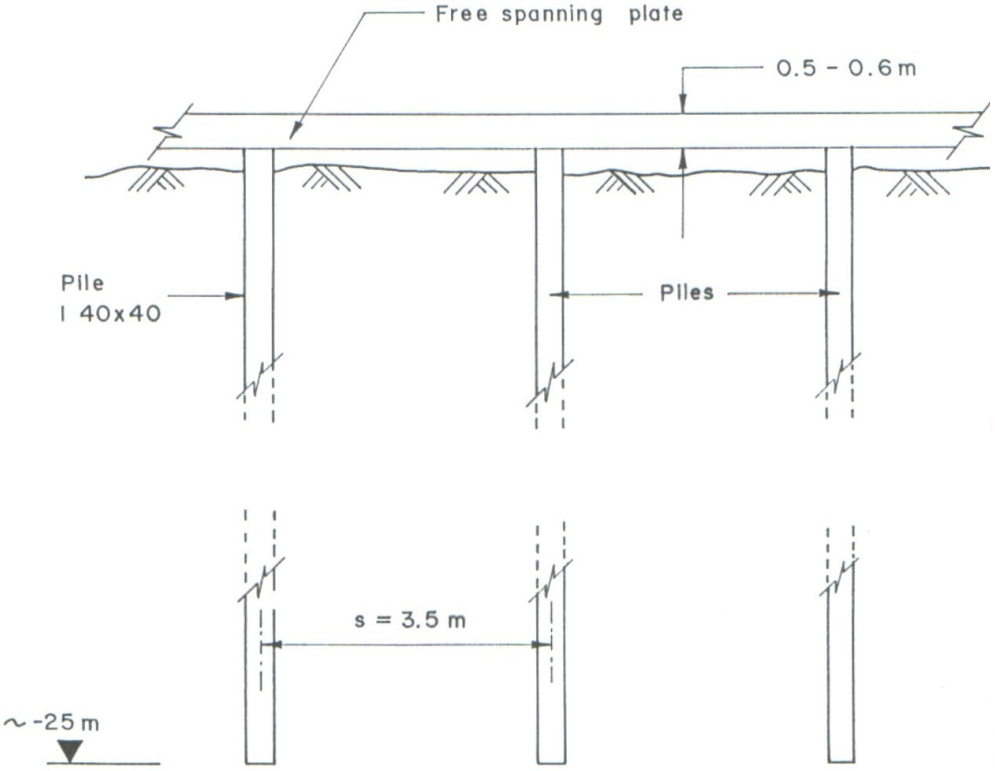


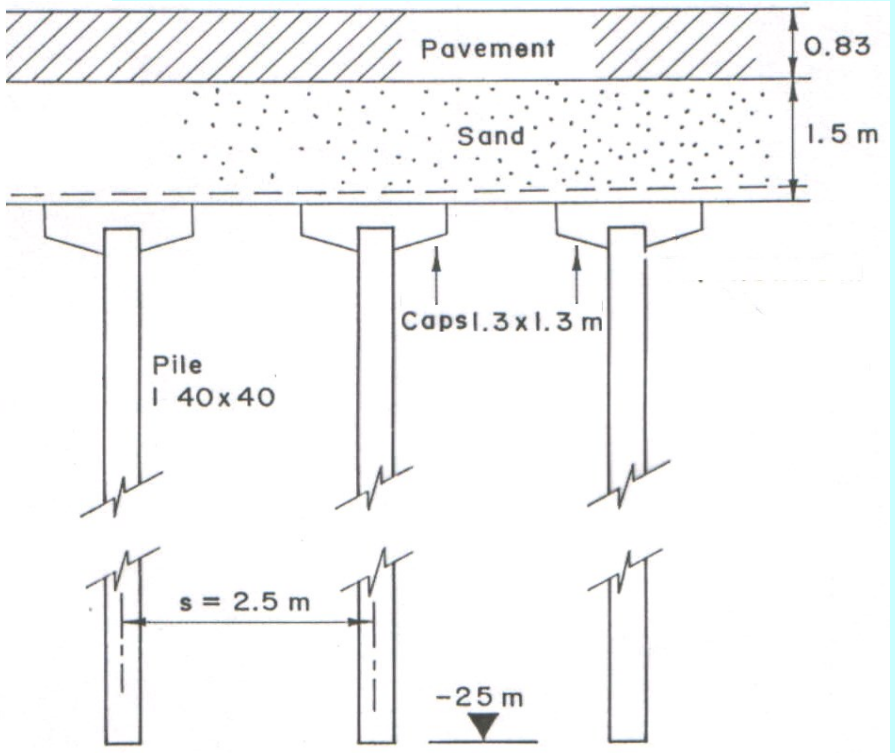
Zones of Ground Improvement

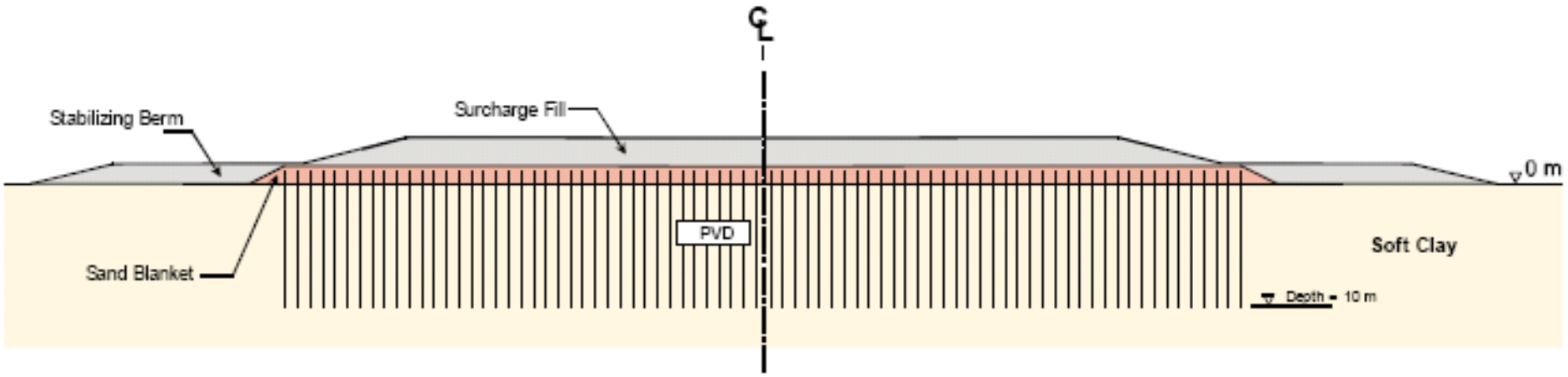
Alternative Design Concepts

- **Preconsolidation with Vertical Drain**
- **Deep Soil Improvement**
- **Piles supporting a free spanning concrete slab**
- **Relief Piles with Caps**

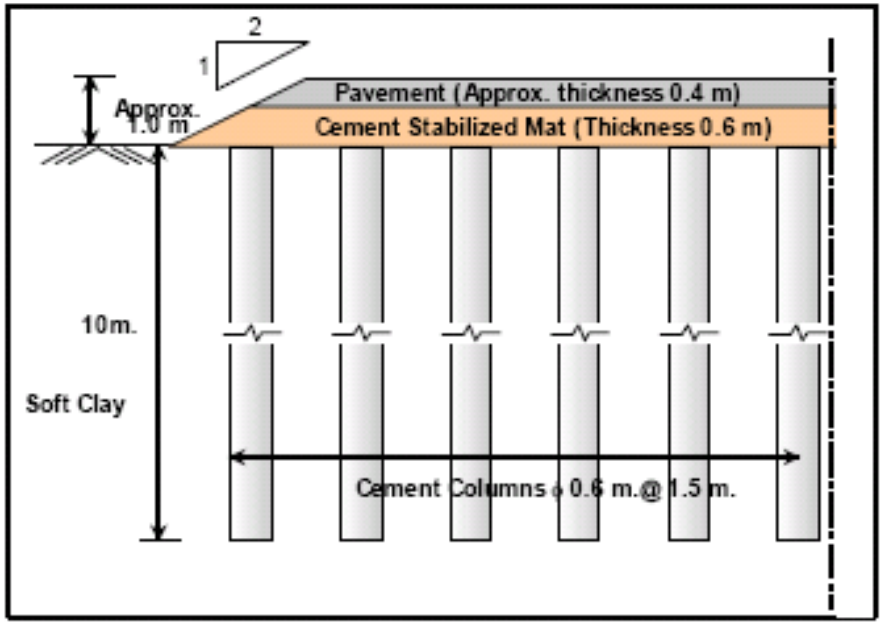


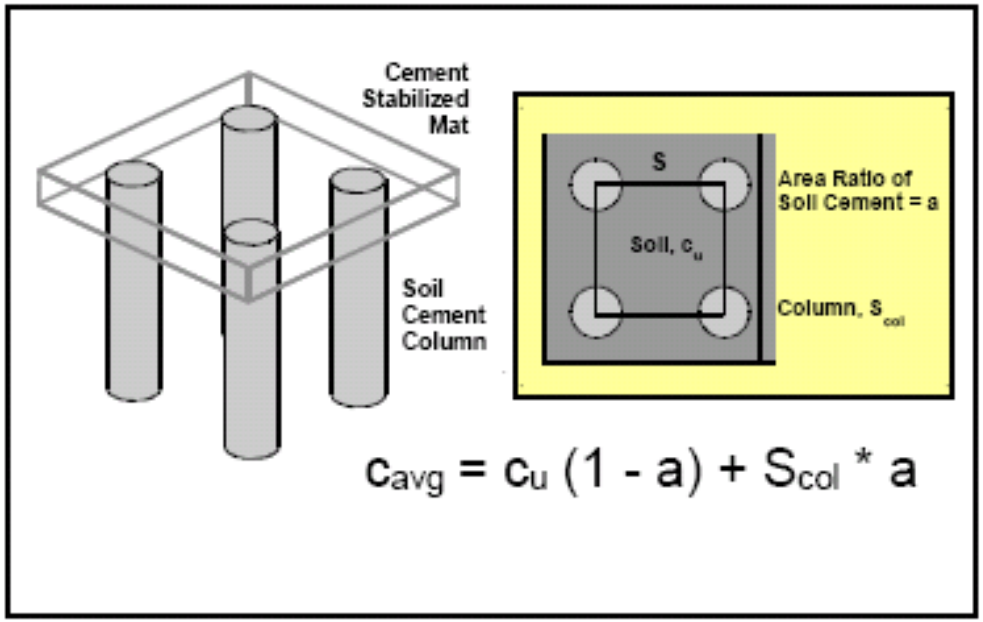




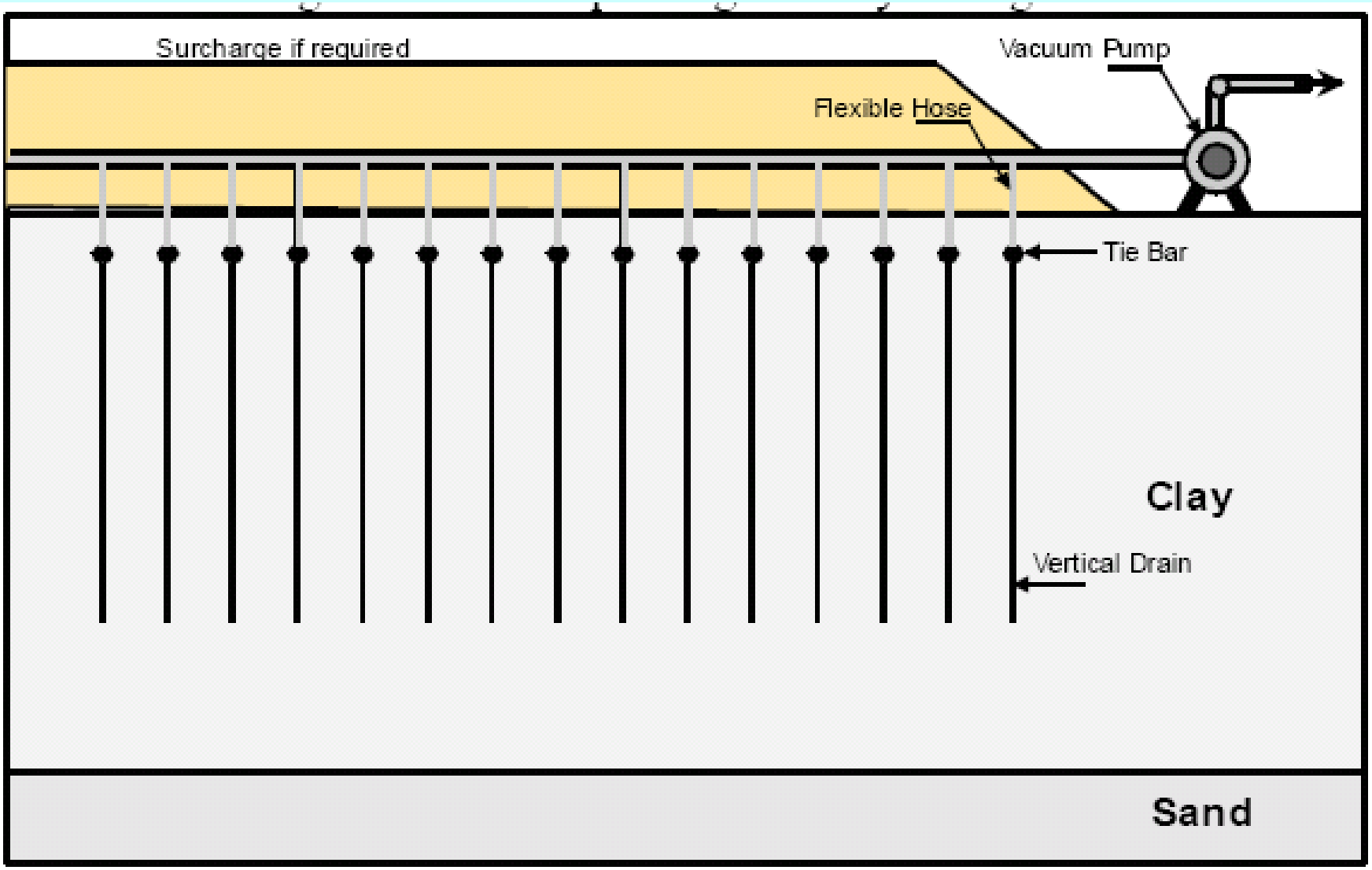


Typical PVD Section of Runway





Typical Soil Cement Column Design



Typical Vacuum Consolidation System

SOIL MECHANICS AND FOUNDATION ENGINEERING EDUCATION IN 1949

- Scope of field limited mainly to:
  - Soil Classification
  - Capillarity and seepage
  - Stress analysis by elasticity
  - Consolidation and settlement analysis
  - Shear strength
  - Slope stability
  - Lateral pressures
  - Bearing capacity
  - Shallow and deep foundations
- Emphasis largely on saturated clays and sands

Virginia Tech

CGPR

DEVELOPMENTS FROM 1950 - 1960

- Slope stability
- Shear strength
- Soil structure, causes of clay sensitivity
- Compacted clay properties
- Pavement design
- Soil stabilization
- Transient loading

Virginia Tech

CGPR

DEVELOPMENTS FROM 1970-1980

- Constitutive modeling
- In-situ testing
- Expansive soils
- Soil dynamics
- Centrifuge testing
- Partly saturated soils
- Geotechnical earthquake engineering
- Underground construction

Virginia Tech

CGPR

RESEARCH WITH HIGH PAYOFF BY 2010

- Faster, less expensive, more reliable in-situ testing
- Rapid methods of site assessment/characterization/investigation
- Less expensive, easier to use high level analysis methods
- New and better ground improvement technologies

Virginia Tech

CGPR

RESEARCH WITH HIGH PAYOFF BY 2010 (Cont.)

- How to apply GIS and the WWW to maximize value of our collective knowledge base
- 21st Century implementation of the Observational Method – real time integration of observations, test data, analysis and design during construction

Virginia Tech

CGPR

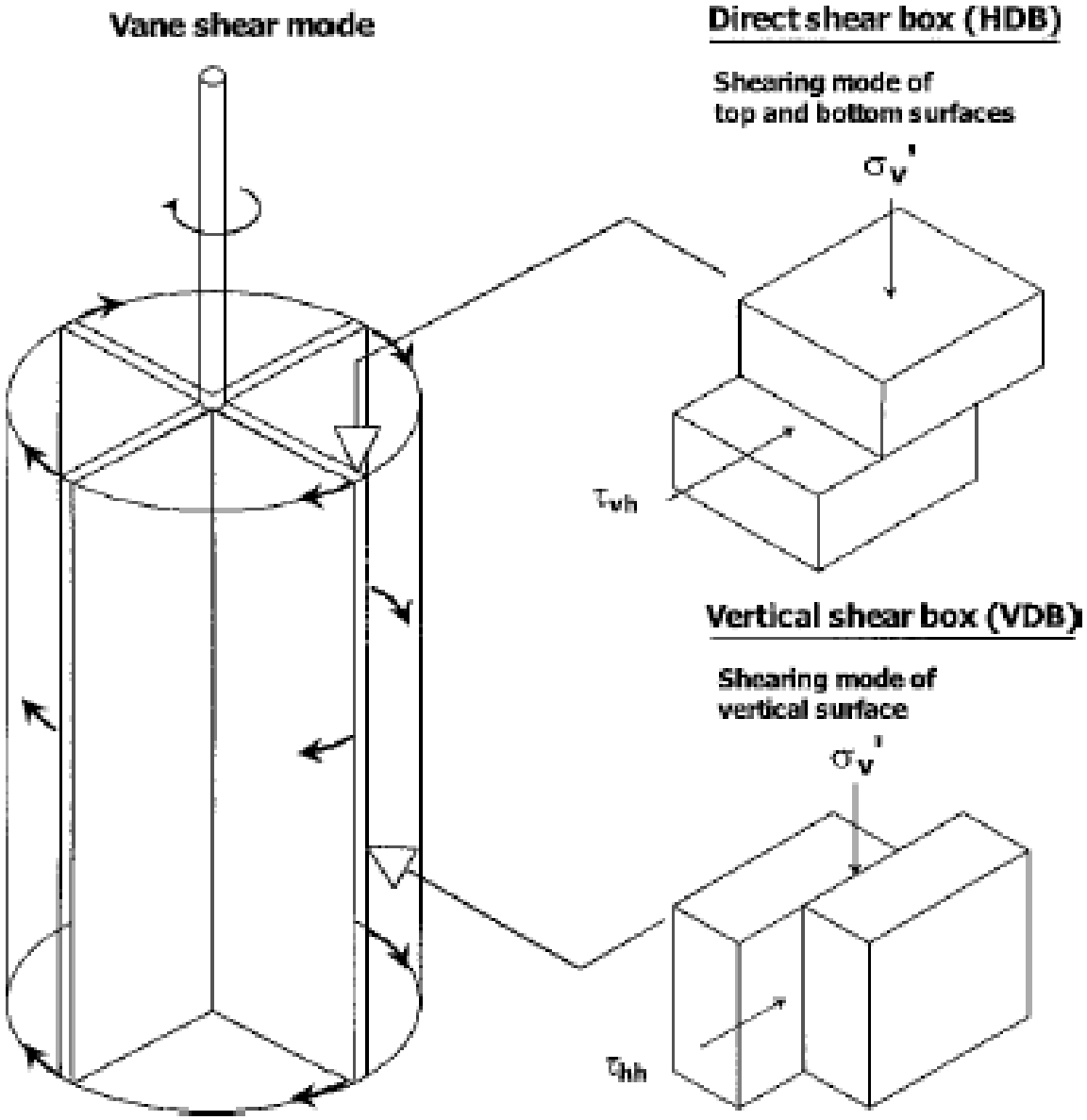
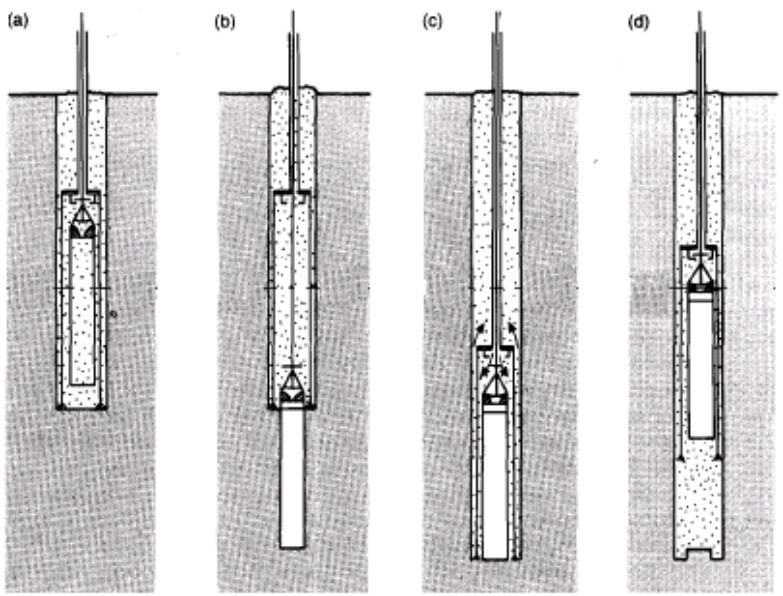
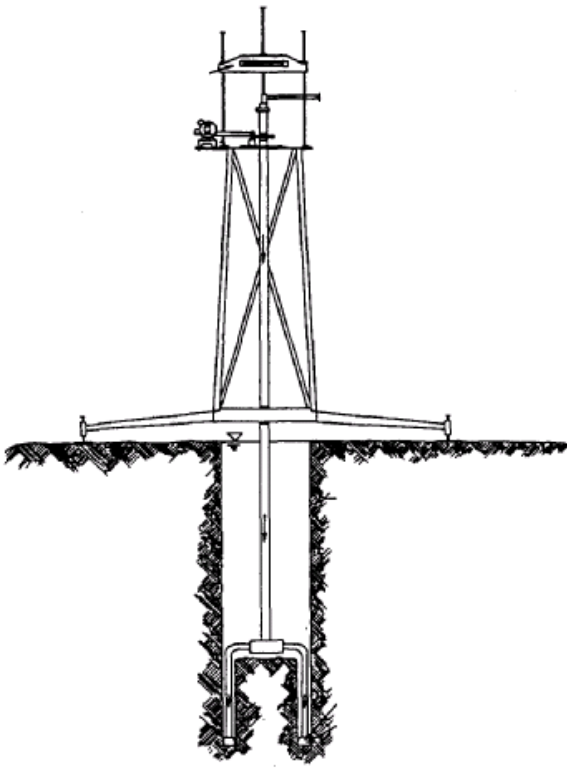
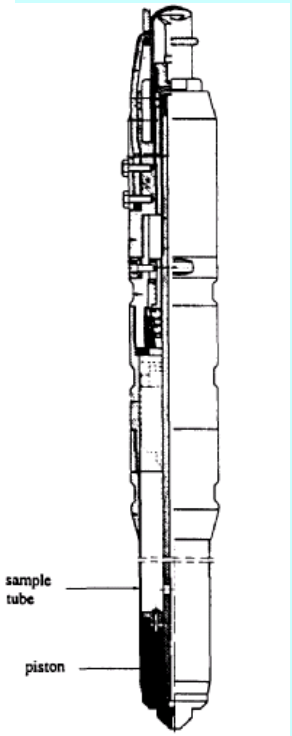


FIG. 1—*Simulation of vane shear mode.*

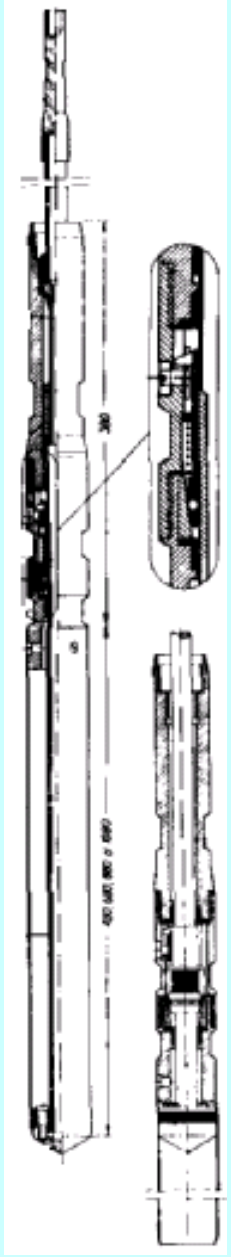


General operation of the Laval sampler

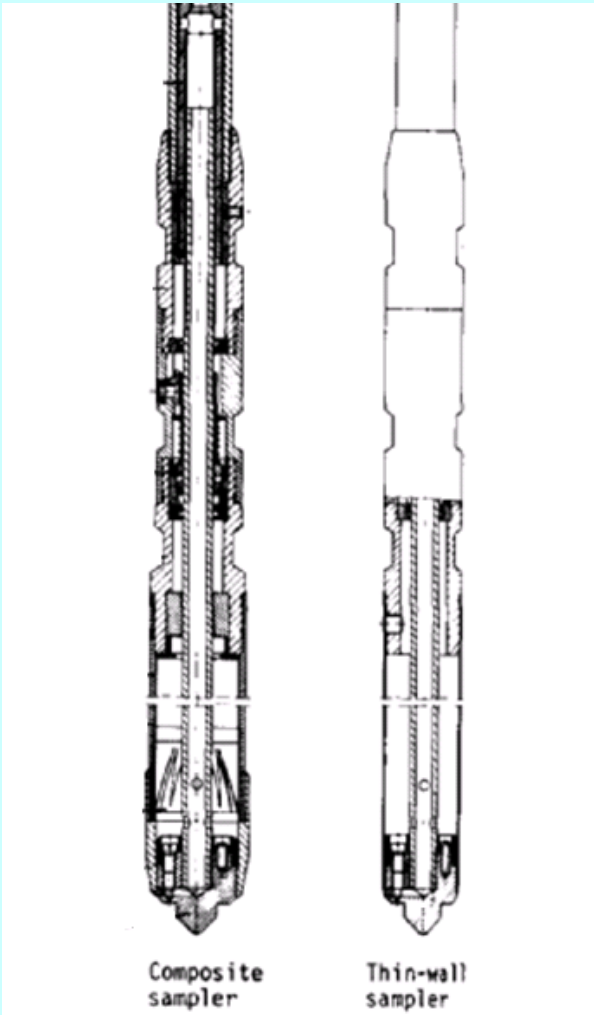




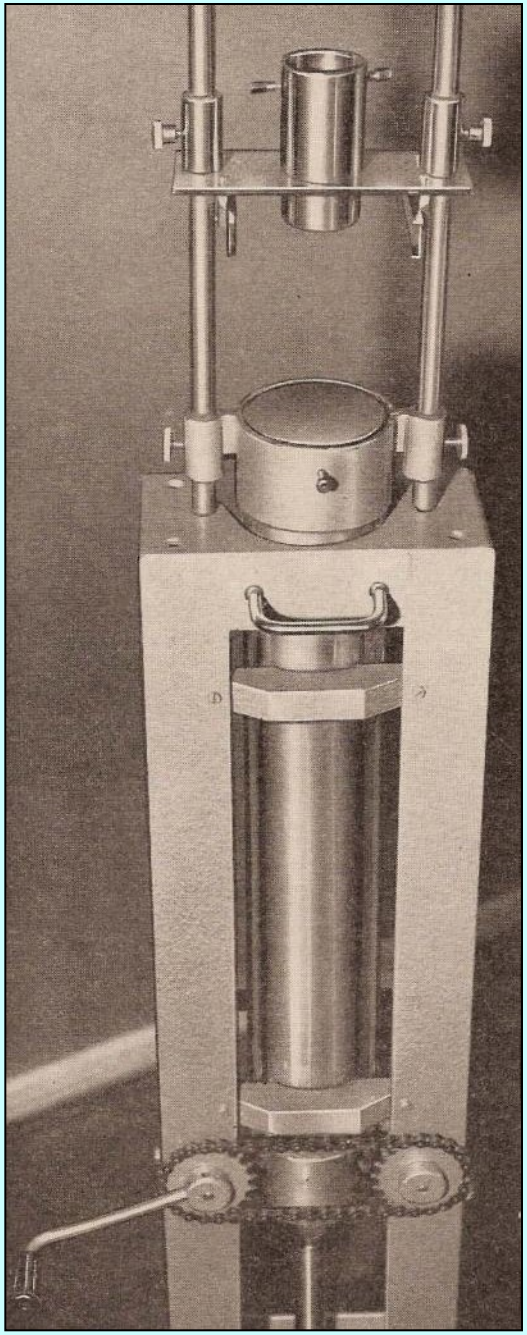
NGI 95 mm piston sampler

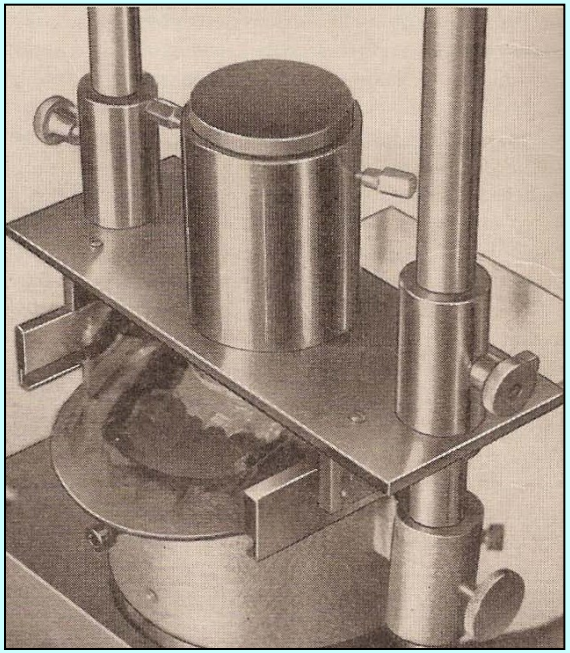


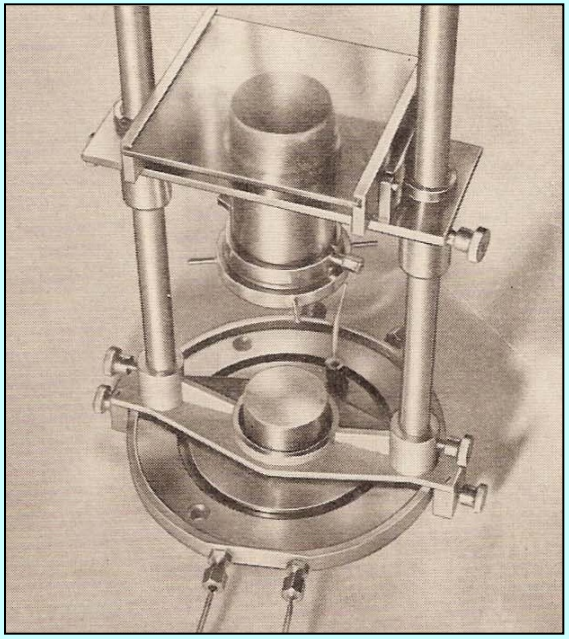
Original NGI 54-mm piston sampler.

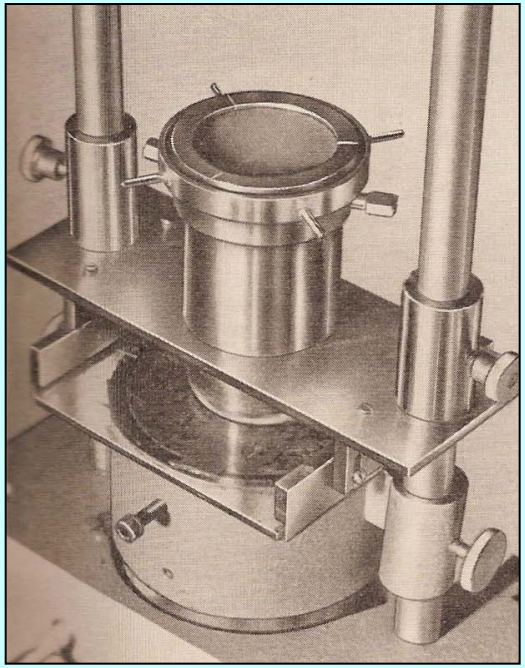


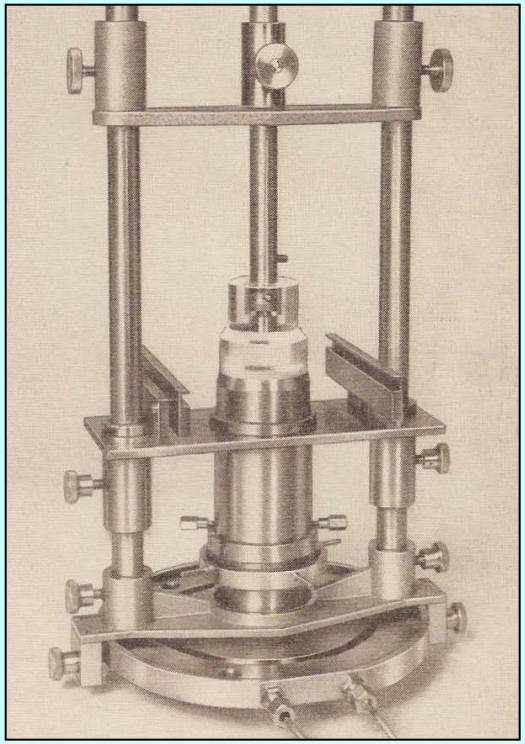
Modified NGI 54-mm piston sampler

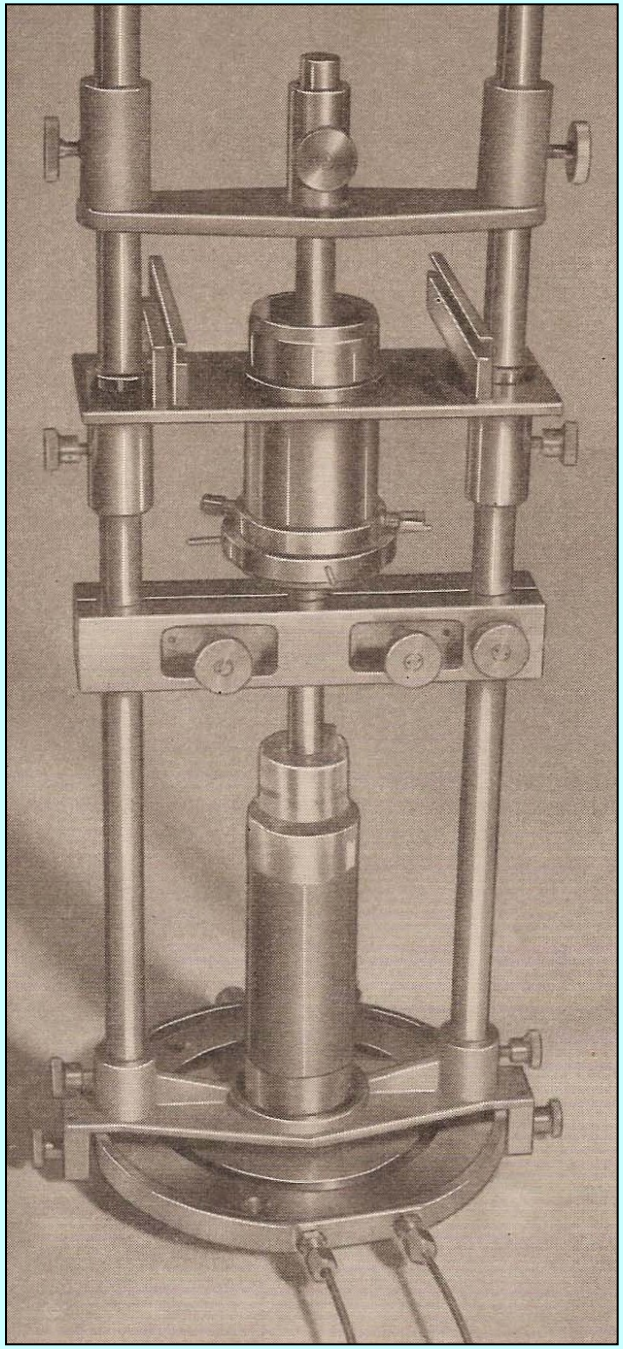


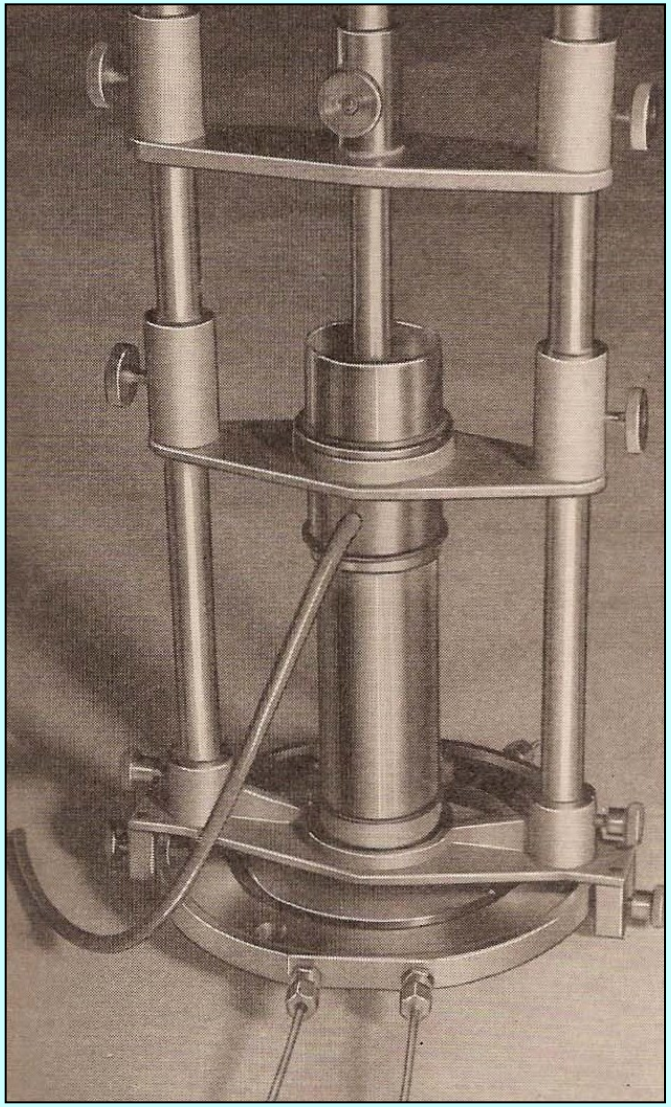


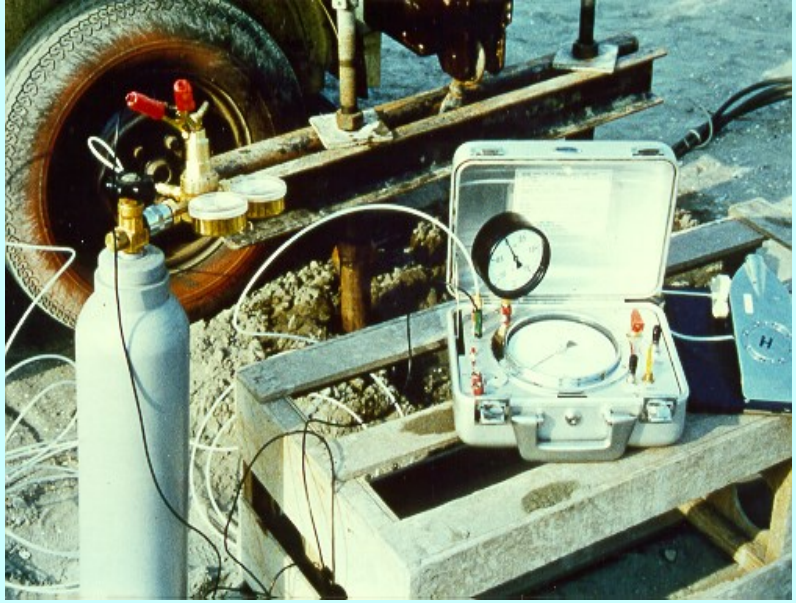













PRESSUREMETER

T. WILLIAM LAMBE, Inc. Consulting Geotechnical Engineer

December 24, 1996

Professor A.S. Balasubramaniam
Asian Institute of Technology
P.O. Box 4
Klong Luang Pathumthani 12120
Thailand

Dear Professor Bala:

The following responds to your letter of 5 December.

I would agree with you that a lot of geotechnical work will occur in the 21st Century. I see a lot of activity by the geotechnical firms. I do not, however, see active and exciting research and devlopment in geotechnical engineering as we saw during the 50's and 60's. I think several factors contribute to the lack of top quality geotechnical research - i.e. lack of research funds; large growth in our Profession which becomes more and more a business; decreased strength of our socities; etc.

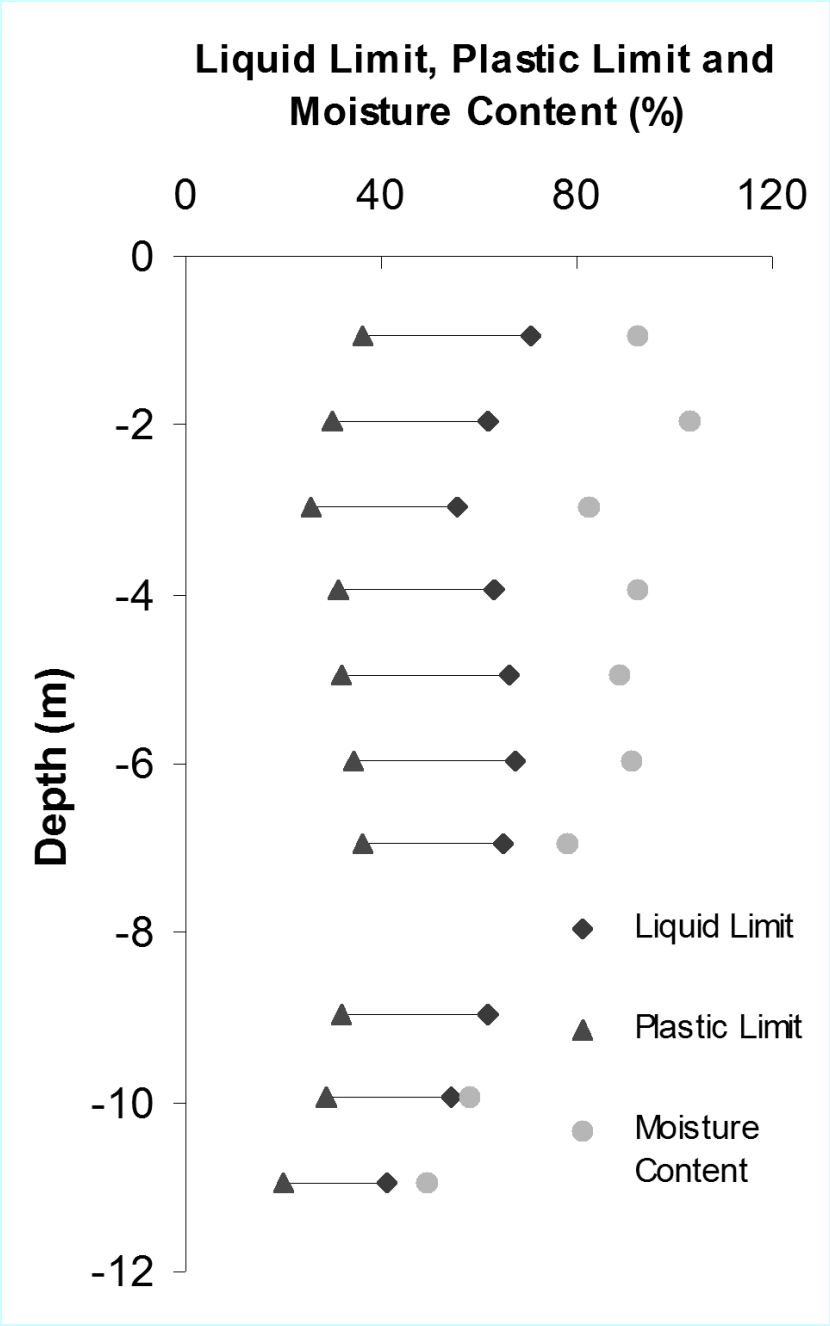
Best wishes for the season and the New Year.

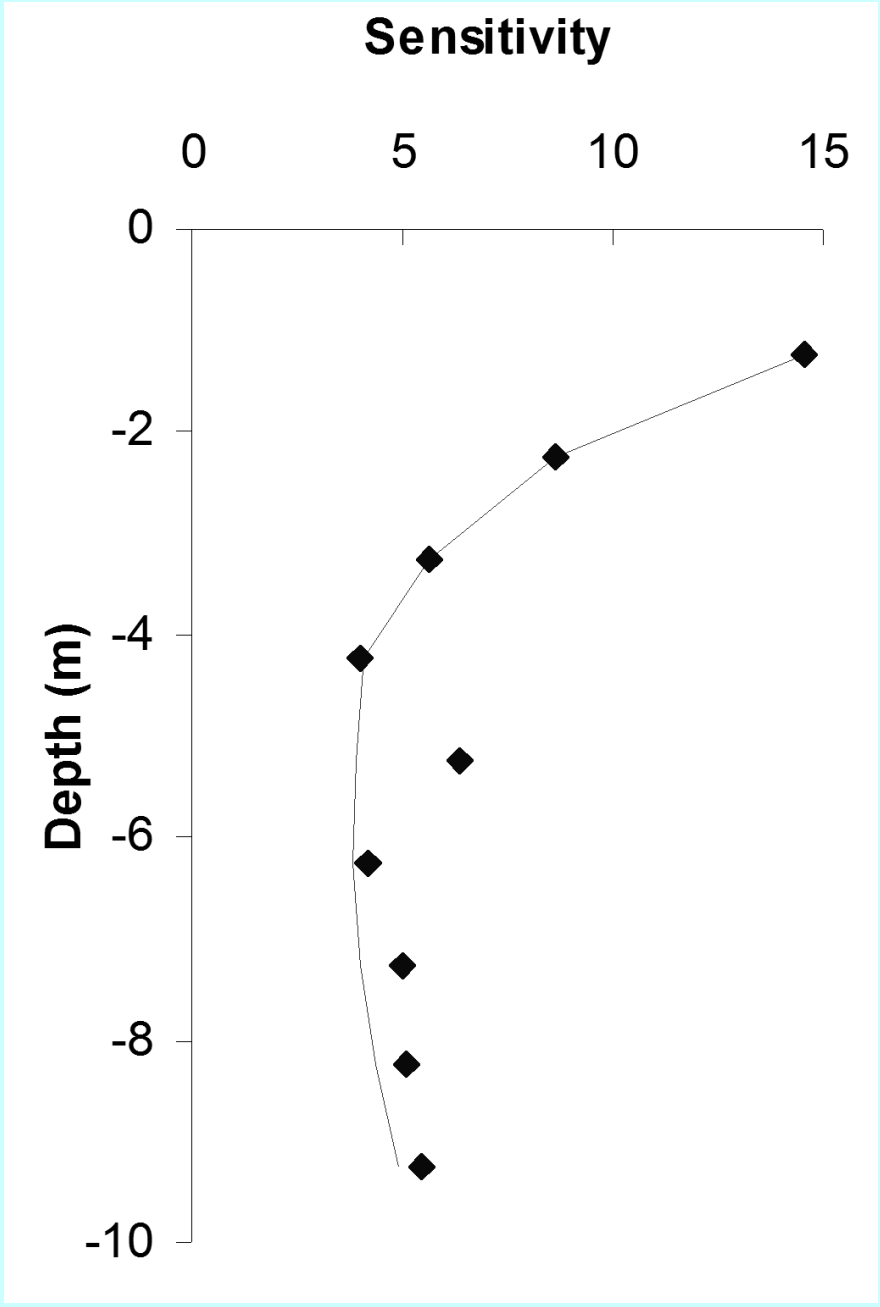
Sincerely yours,

TWL/cl

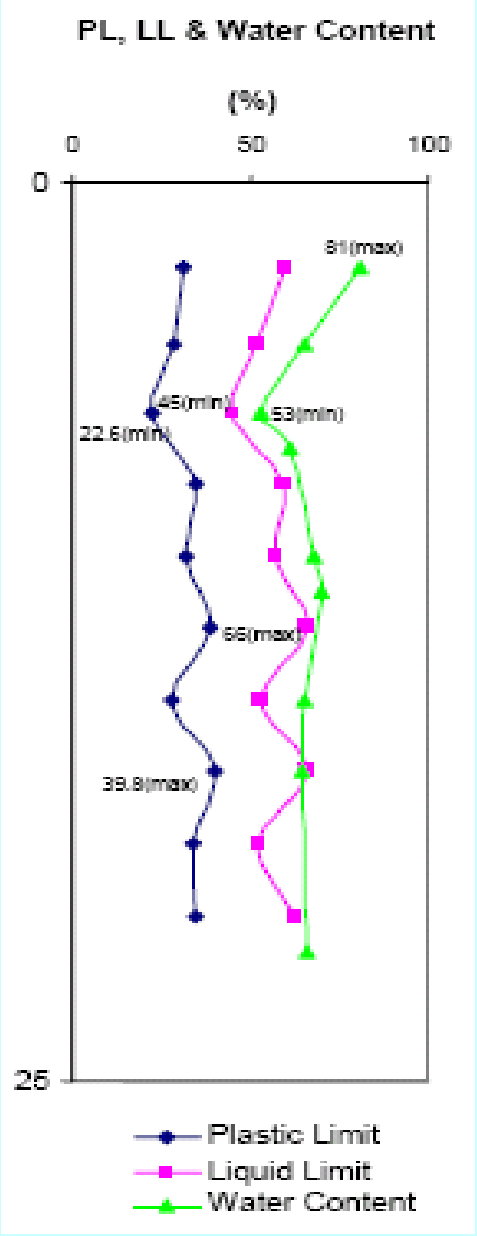
T. William Lambe

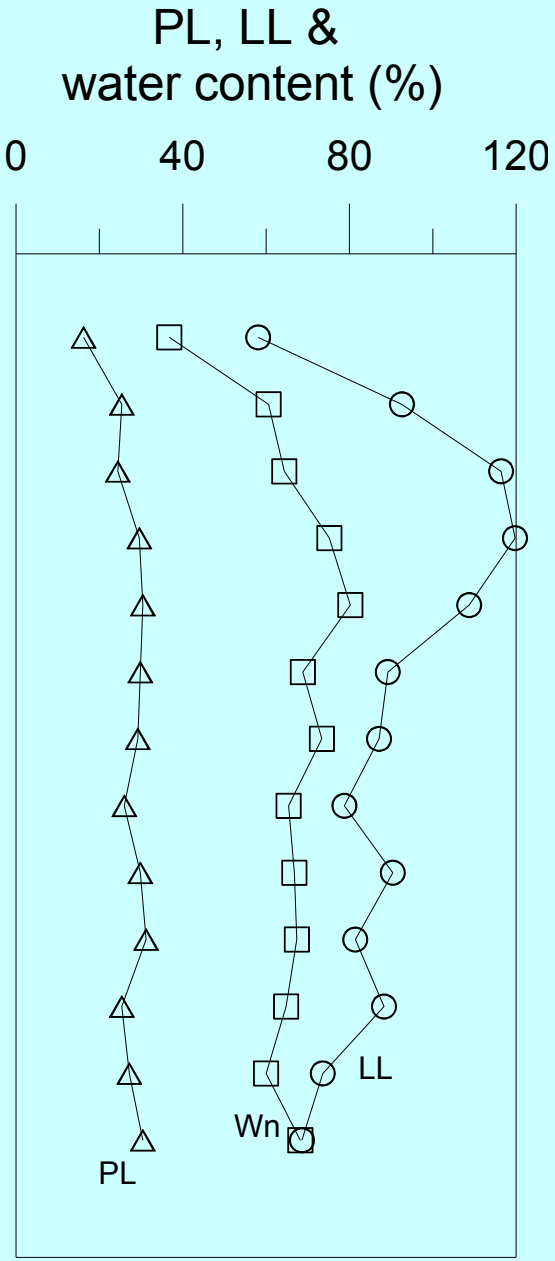
1. Gold Coast Highway
   (stone columns)
2. Sunshine Coast Motorway
   (Vertical drains-PVD)
3. Port of Brisbane Motorway
   (Vertical drains-PVD)





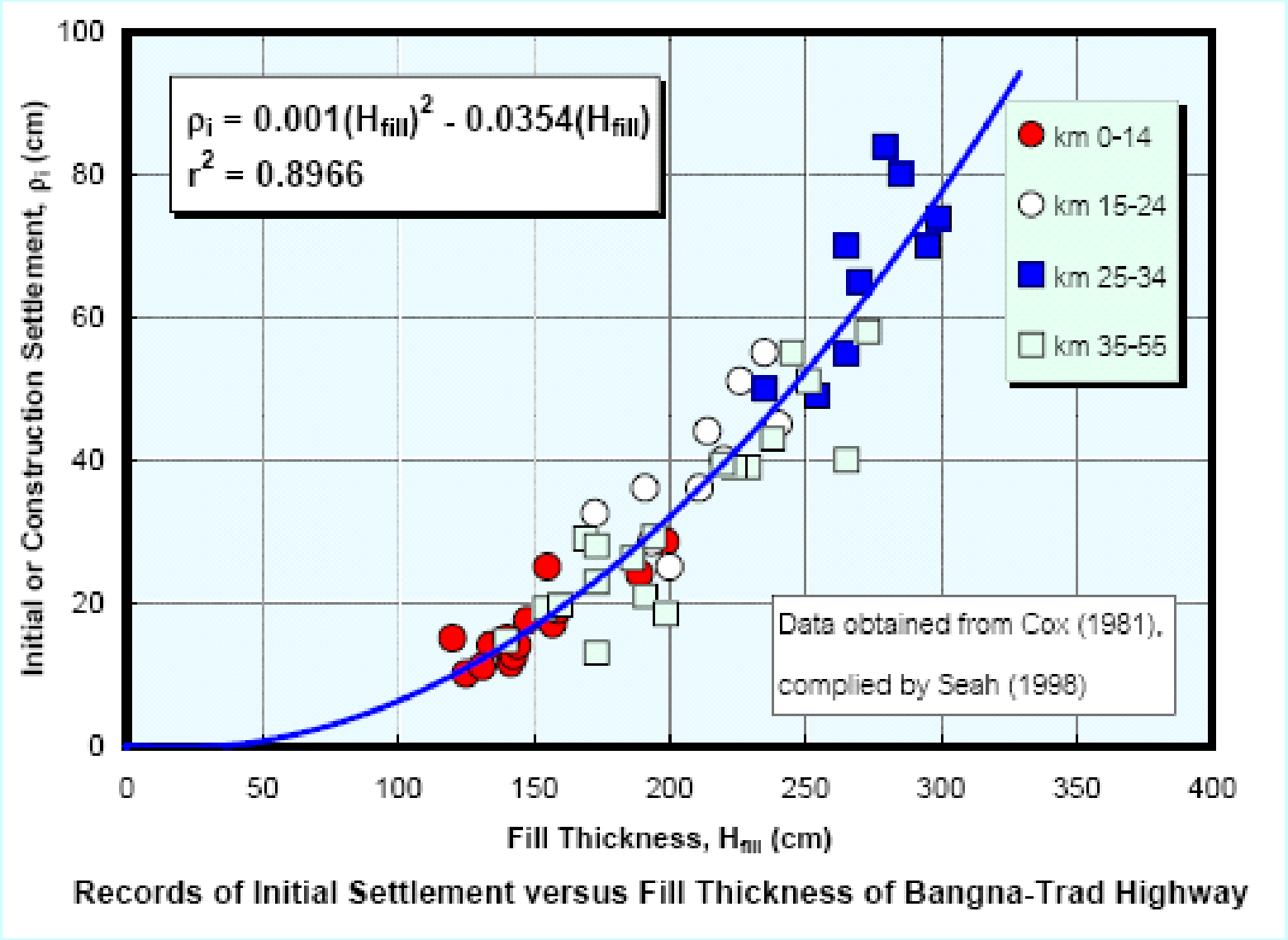
Gold Coast Highway



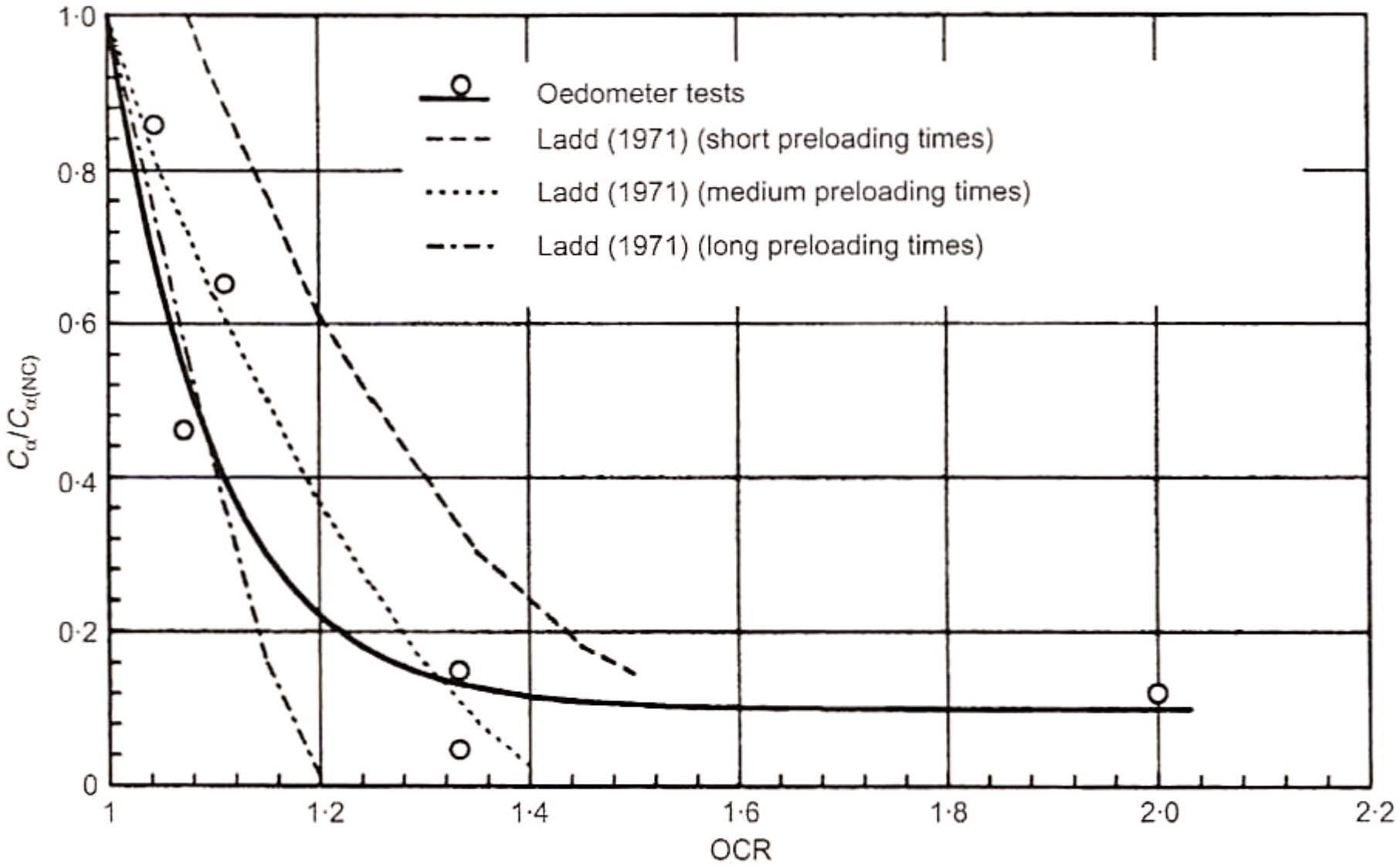


Port of Brisbane Motorway

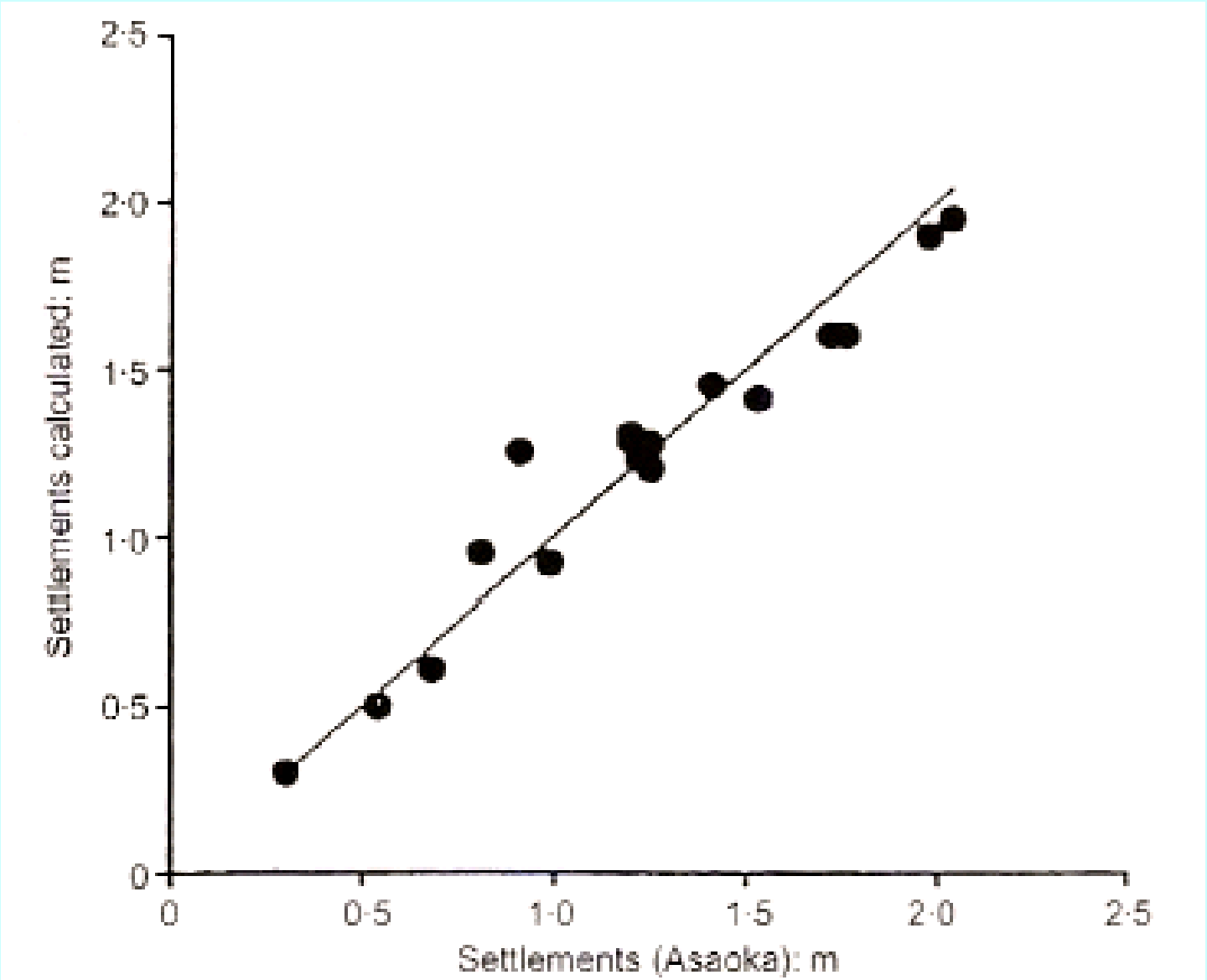
Sunshine Motorway



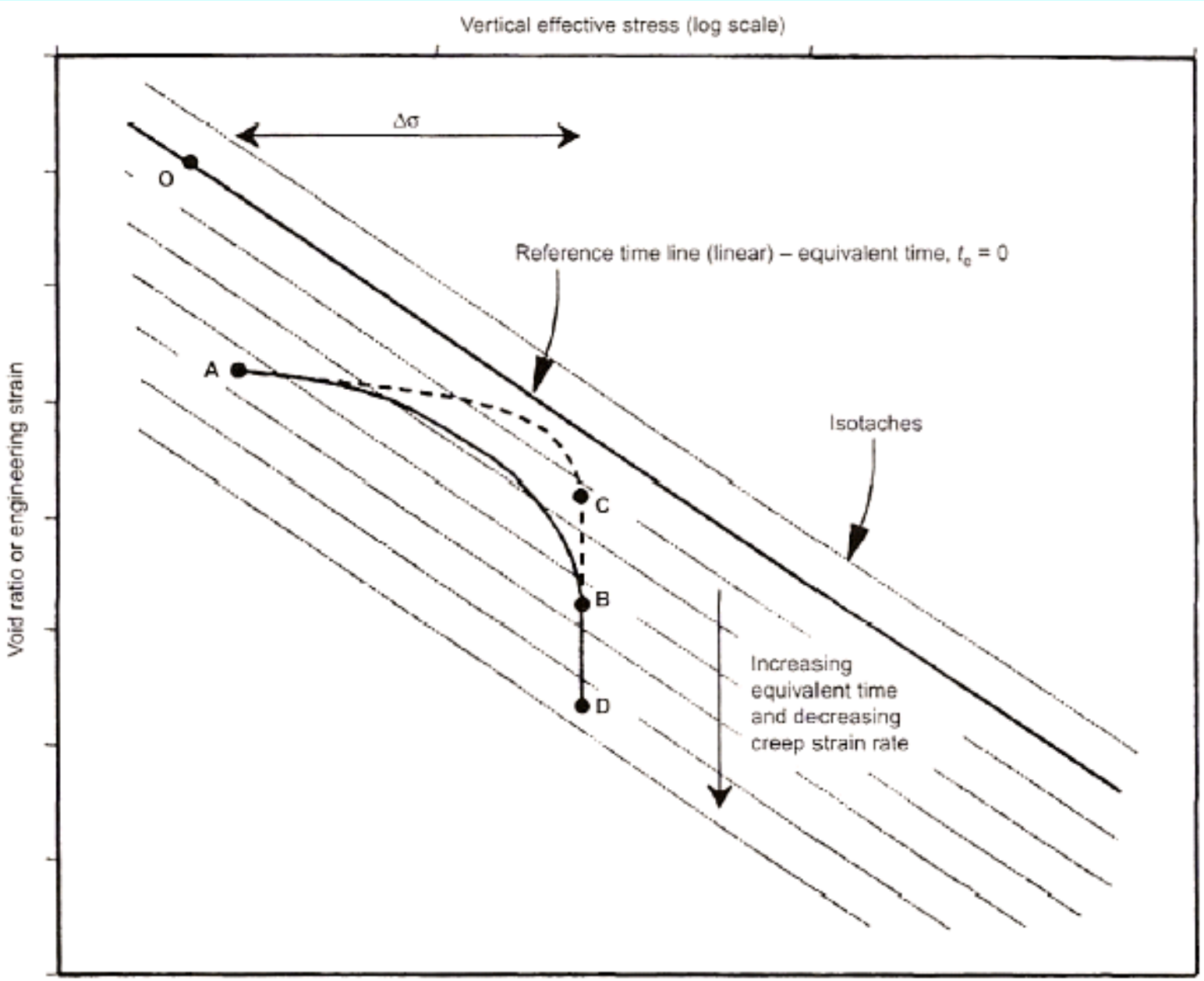
Records of Initial Settlement versus Fill Thickness of Bangna-Trad Highway



Reduction of secondary compression coefficient with overconsolidation ratio observed in oedometer tests



Comparison between measured and calculated settlements



Elastic visco-plastic constitutive model showing isotaches and stress–strain path during compression

How reliable are the soft clay parameters from lab tests for settlement computations- magnitude, rate and secondary settlement;

Is there any magic number to separate immediate settlement from total settlement?

Are the modules etc. good enough to estimate immediate settlement?

Stress path methods etc. are these now forgotten?

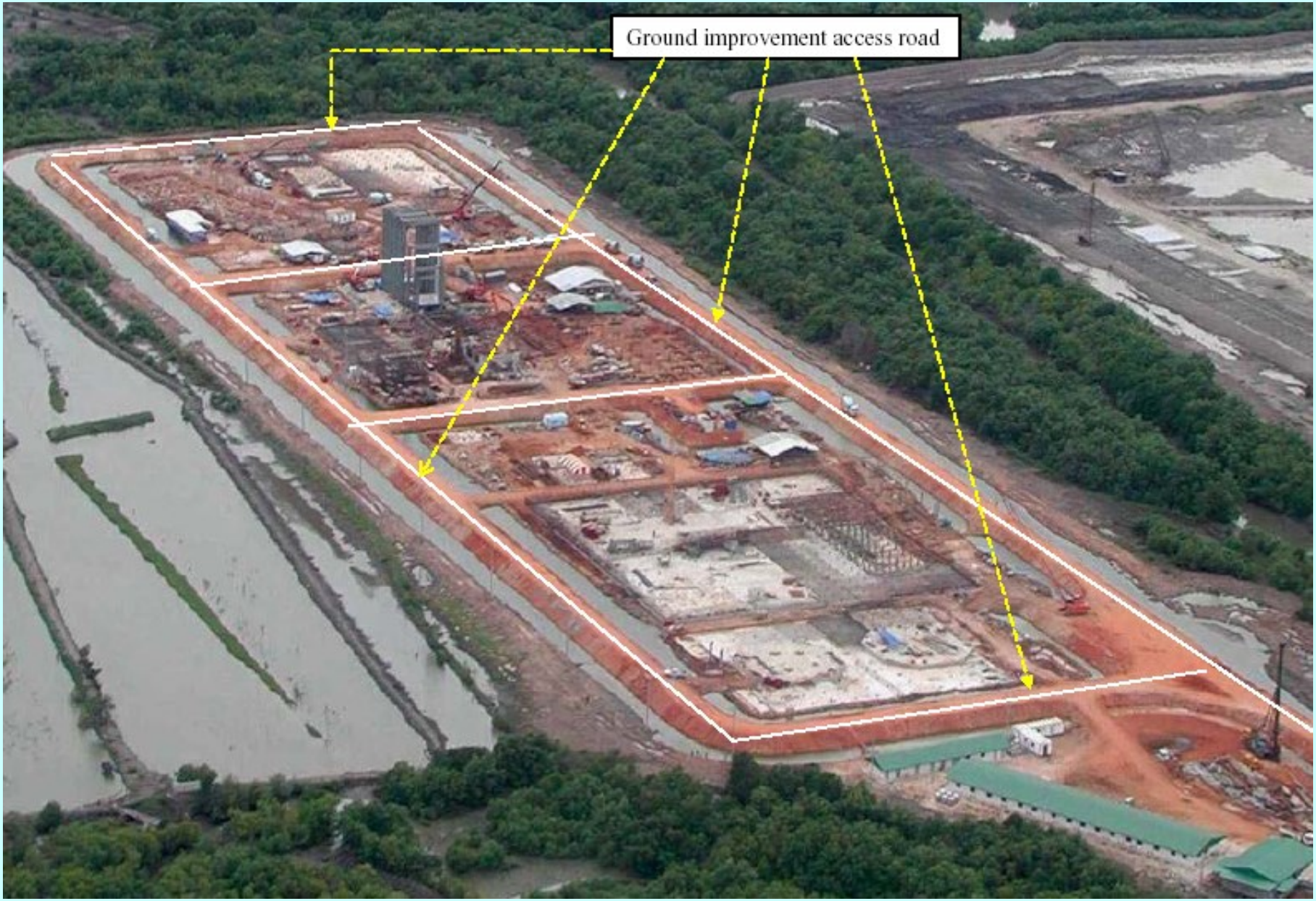
The reliability of the laboratory values depends, of course, very much on the sampling methods and the personnel performing the lab tests.

It is extremely important that the samples are as undisturbed as possible and that the laboratory equipment is good. The laboratory test are carried out by advanced lab personnel

Vacuum consolidation

at

Bangpo Thermal Power Plant-- Bangkok


Ground improvement access road