Wednesday, 6th December--- 4

Ground Improvement Works at the Airport Site in Bangkok

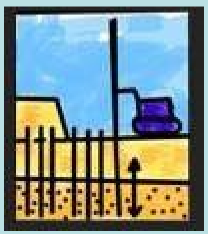
COMMON GROUND IMPROVEMENT TECHNIQUES

Thick soft fined grained soil (silt and clay)

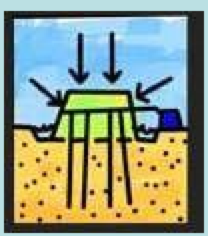
“Mixture” of soils

Thick loose coarse grained soil (sand and gravel)

CONSOLIDATION methods



Prefabricated Vertical Drains

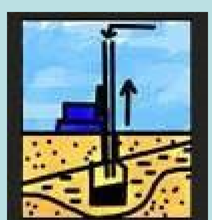


Vacuum Consolidation™

REINFORCEMENT methods

Semi-Rigid Inclusion (cement grout etc.)

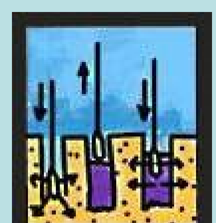
Controlled Modulus Columns™



Soil Mixing Columns

Natural Inclusion (sand, stone, etc.)

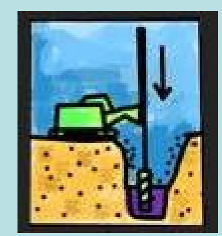
Dynamic Replacement™



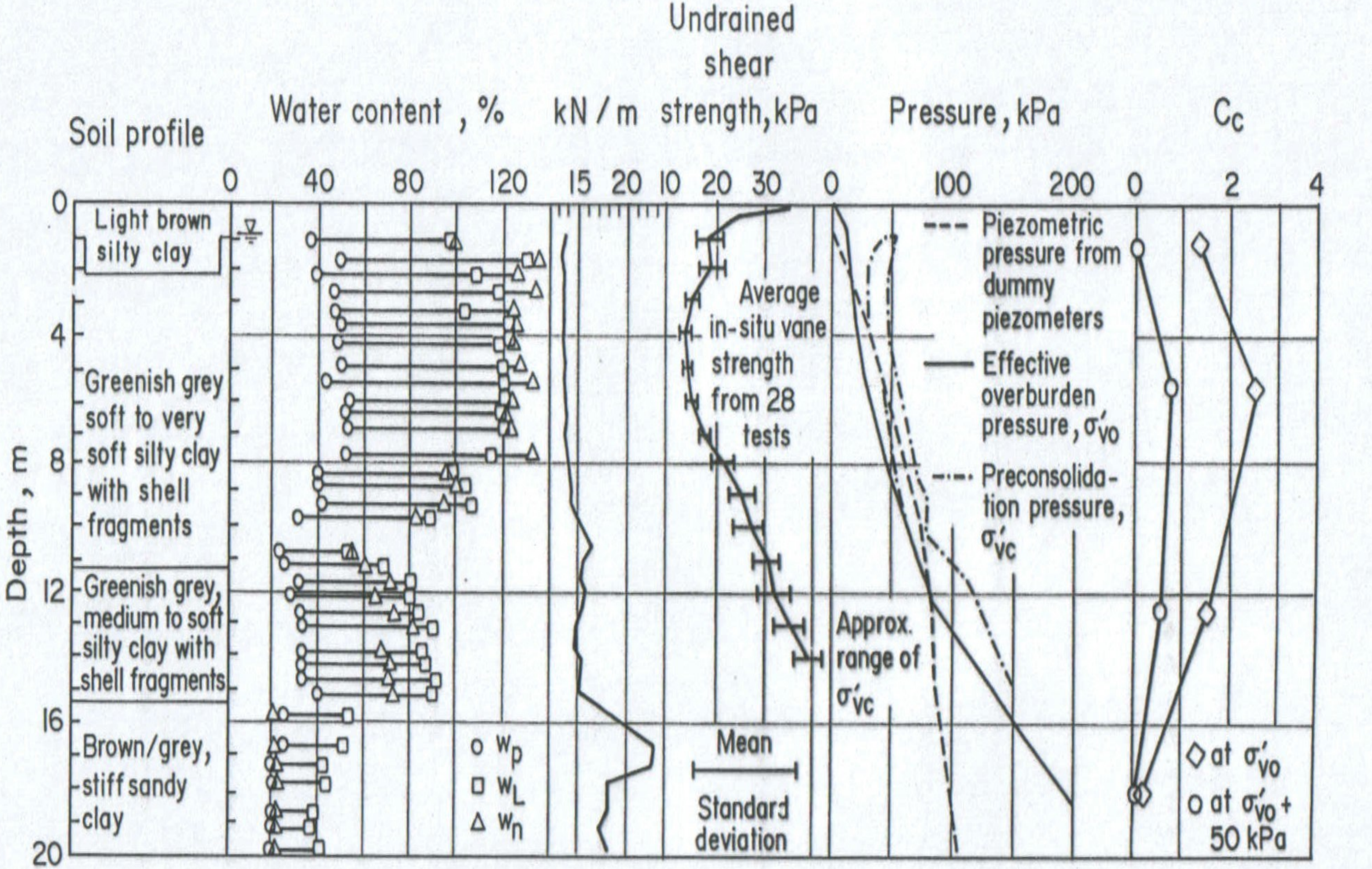
Vibro Replacement

COMPACTION methods

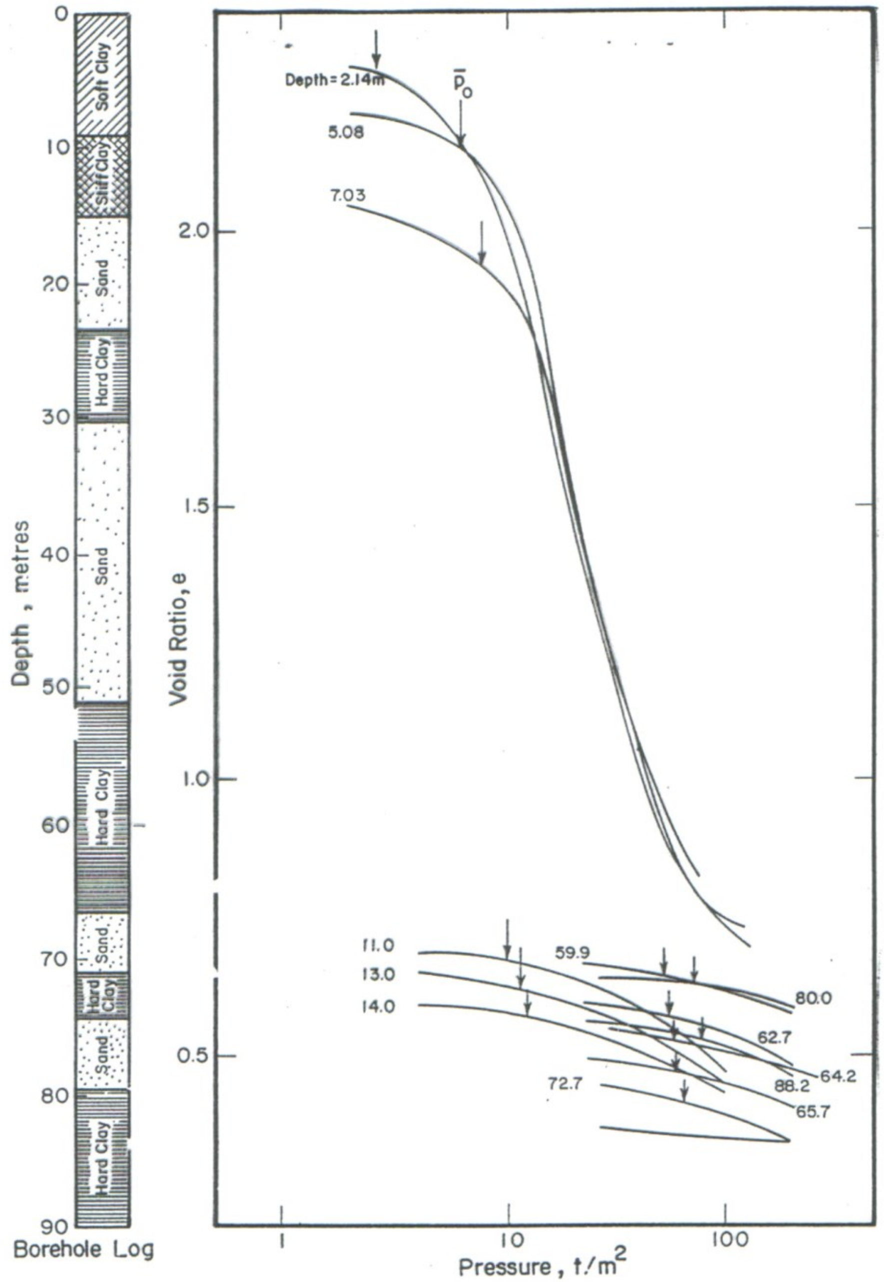
Dynamic Compaction™



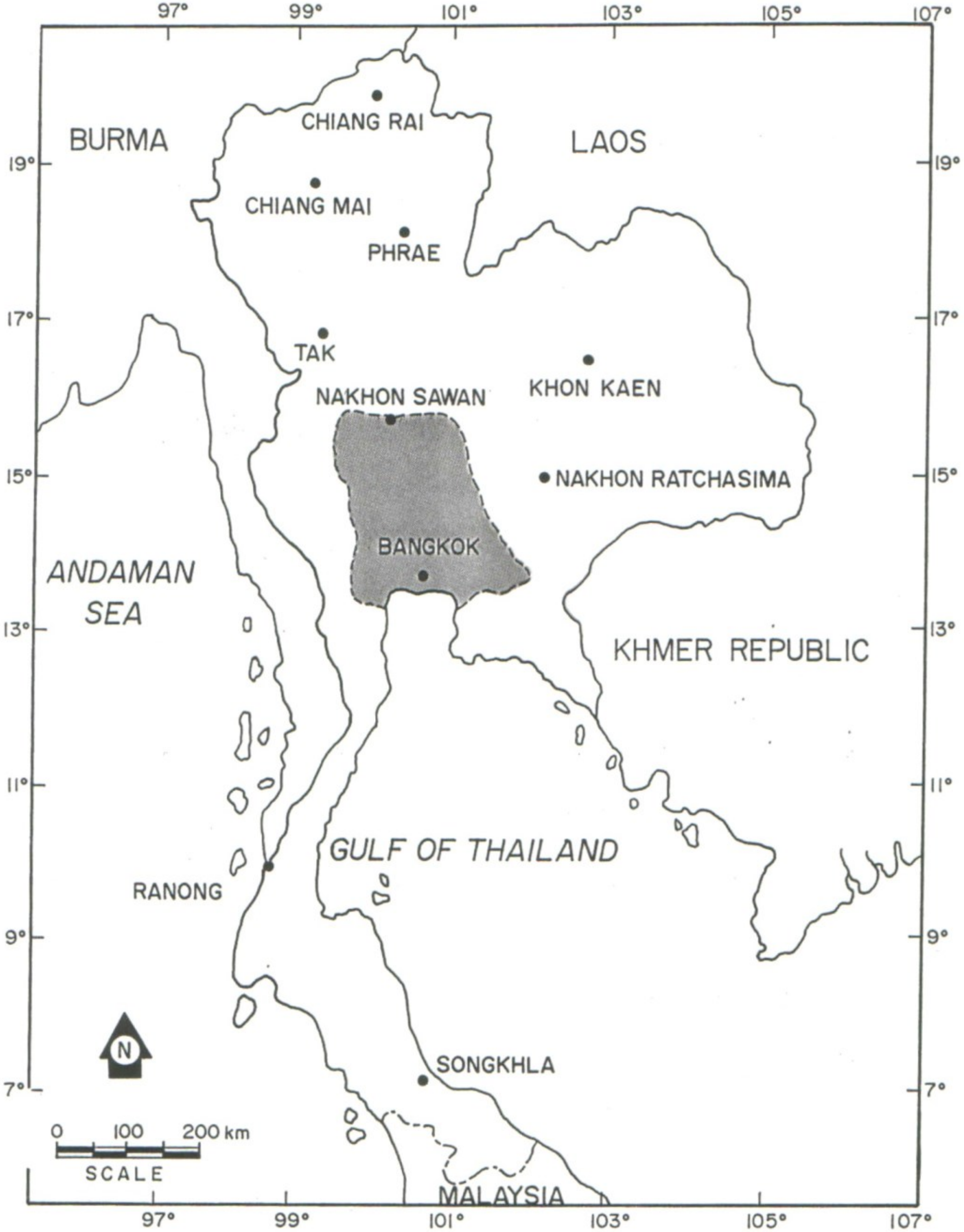
Vibro Compaction



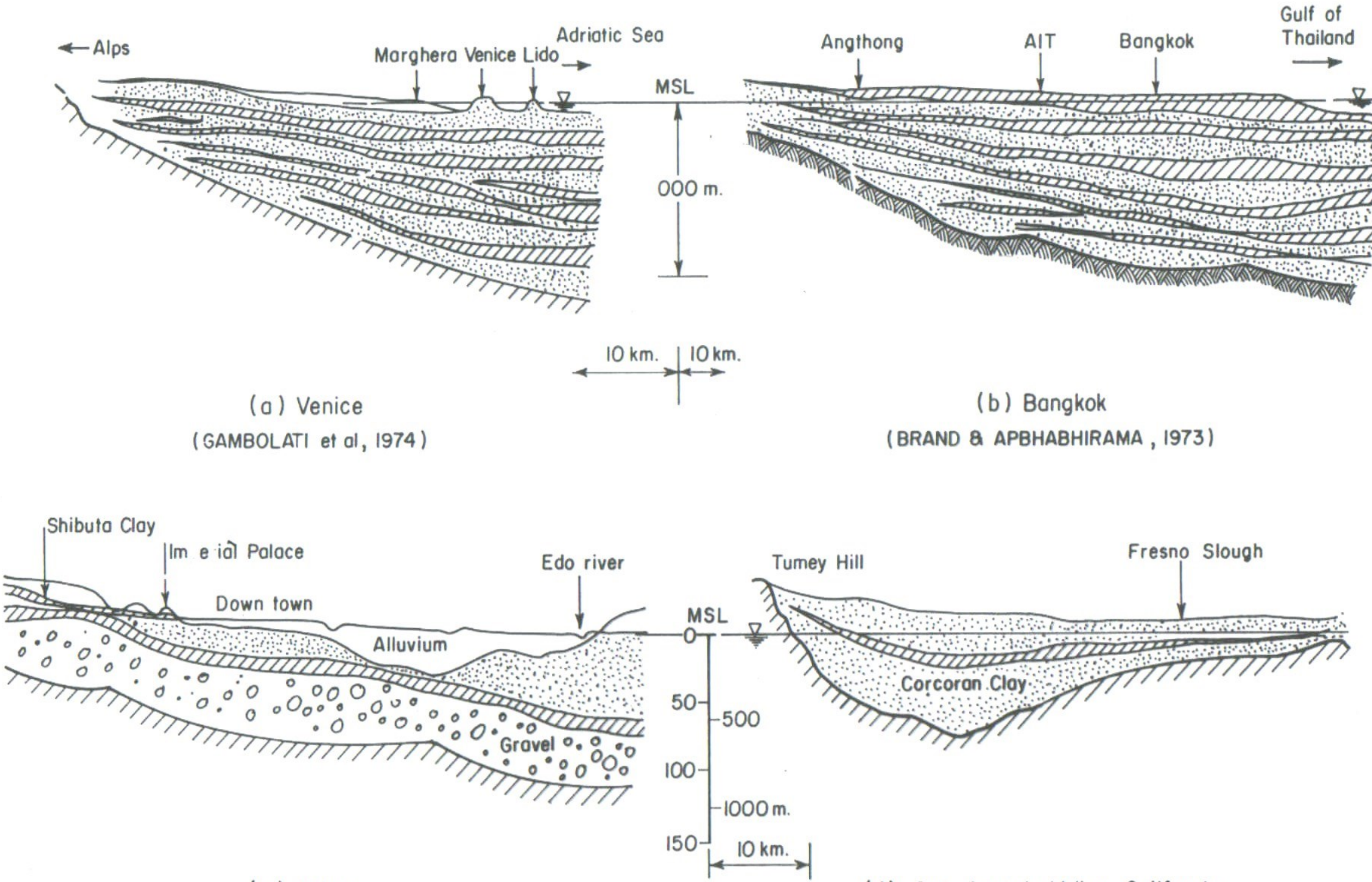
Geotechnical characteristics of soft Bangkok clay at Bangpli



Typical Clay Compression Curves at AIT (after TAESIRI, 1976 & KANJANAKAROON, 1977)



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



(a) Venice
(GAMBOLATI et al, 1974)

(b) Bangkok
(BRAND & APBHABHIRAMA, 1973)

(c) Tokyo
(NAKANO et al, 1969)

(d) San Joaquin Valley, California
(POLAND et al, 1975)

Geologic Profiles of Some Areas of Land Subsidence

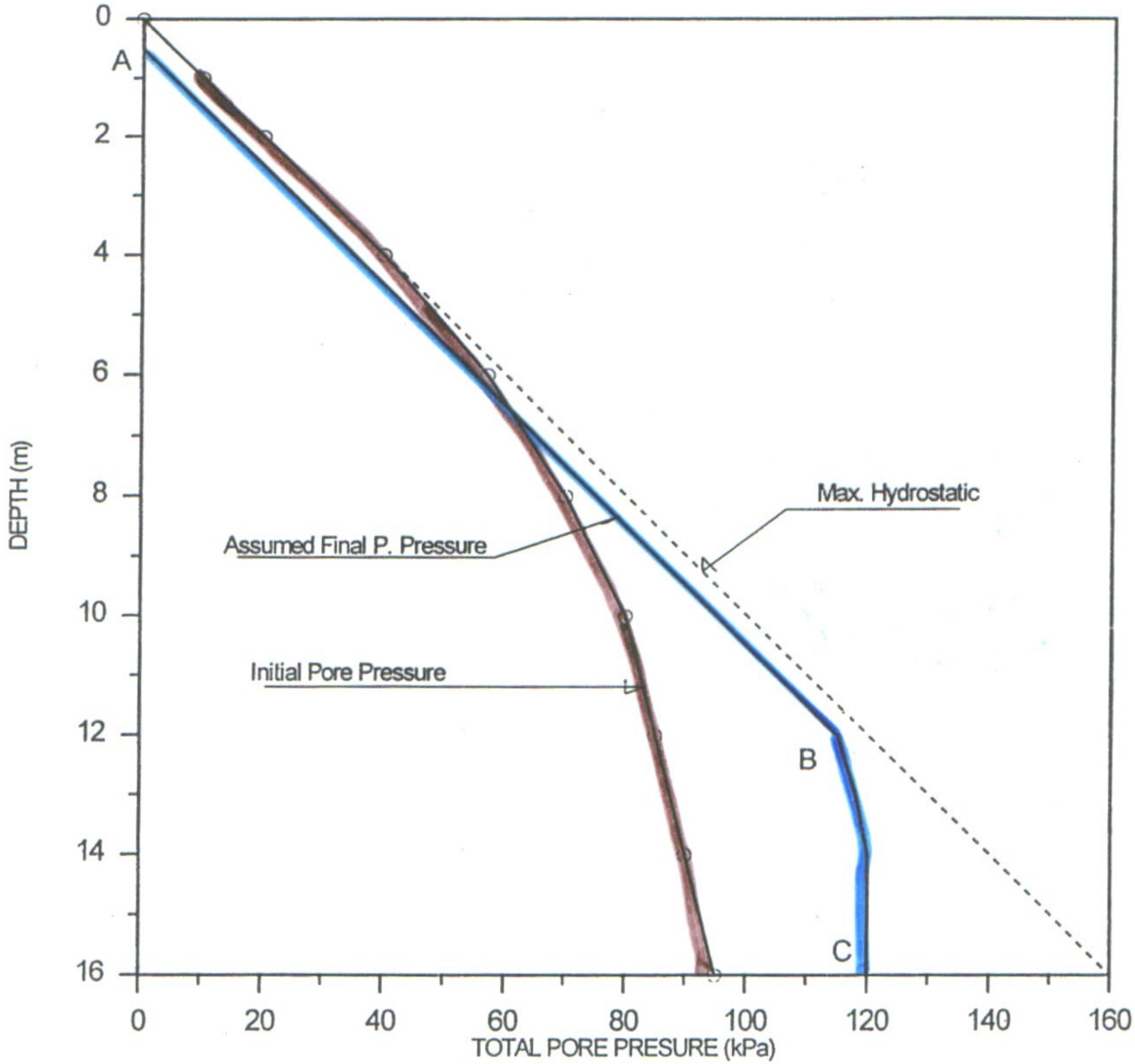
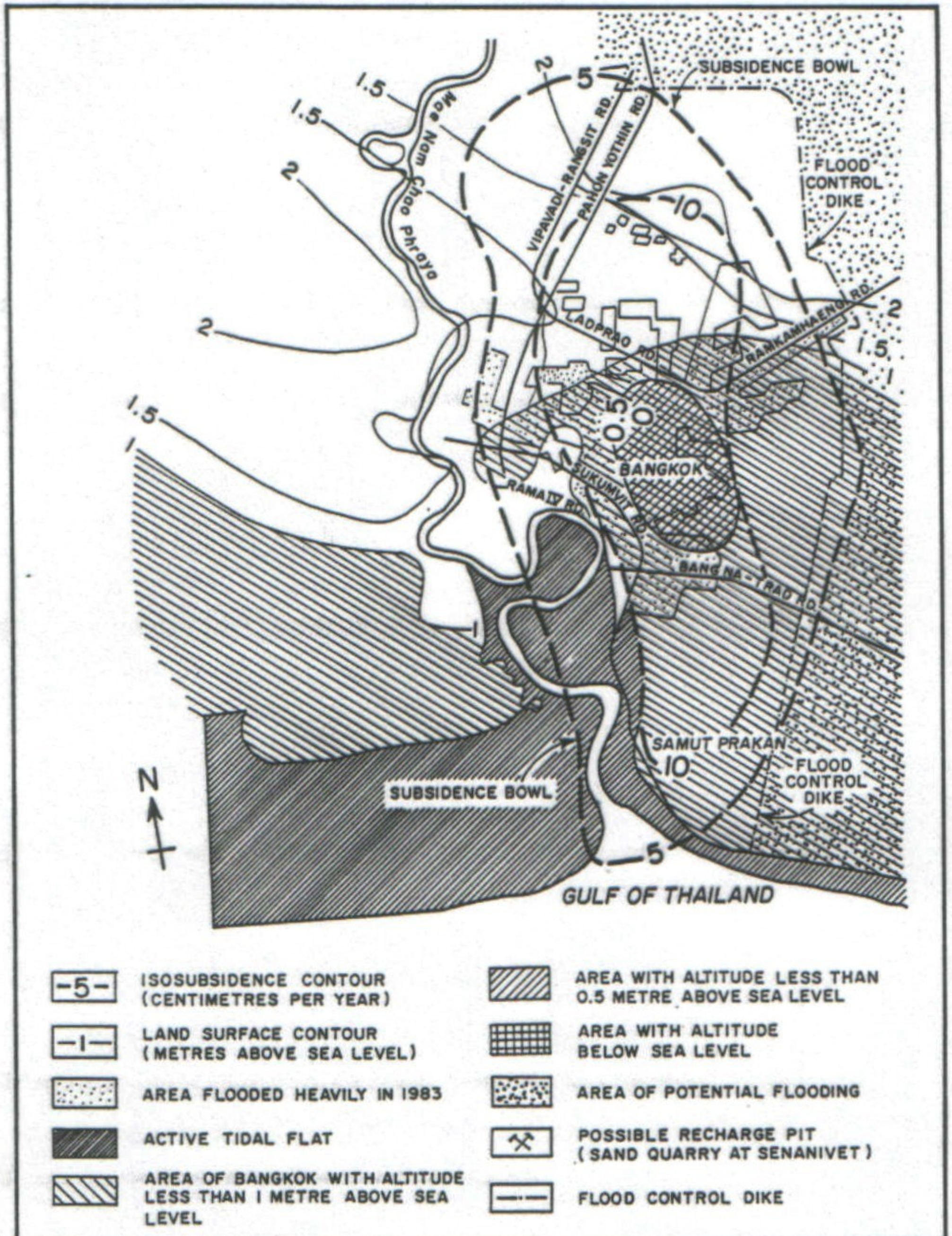
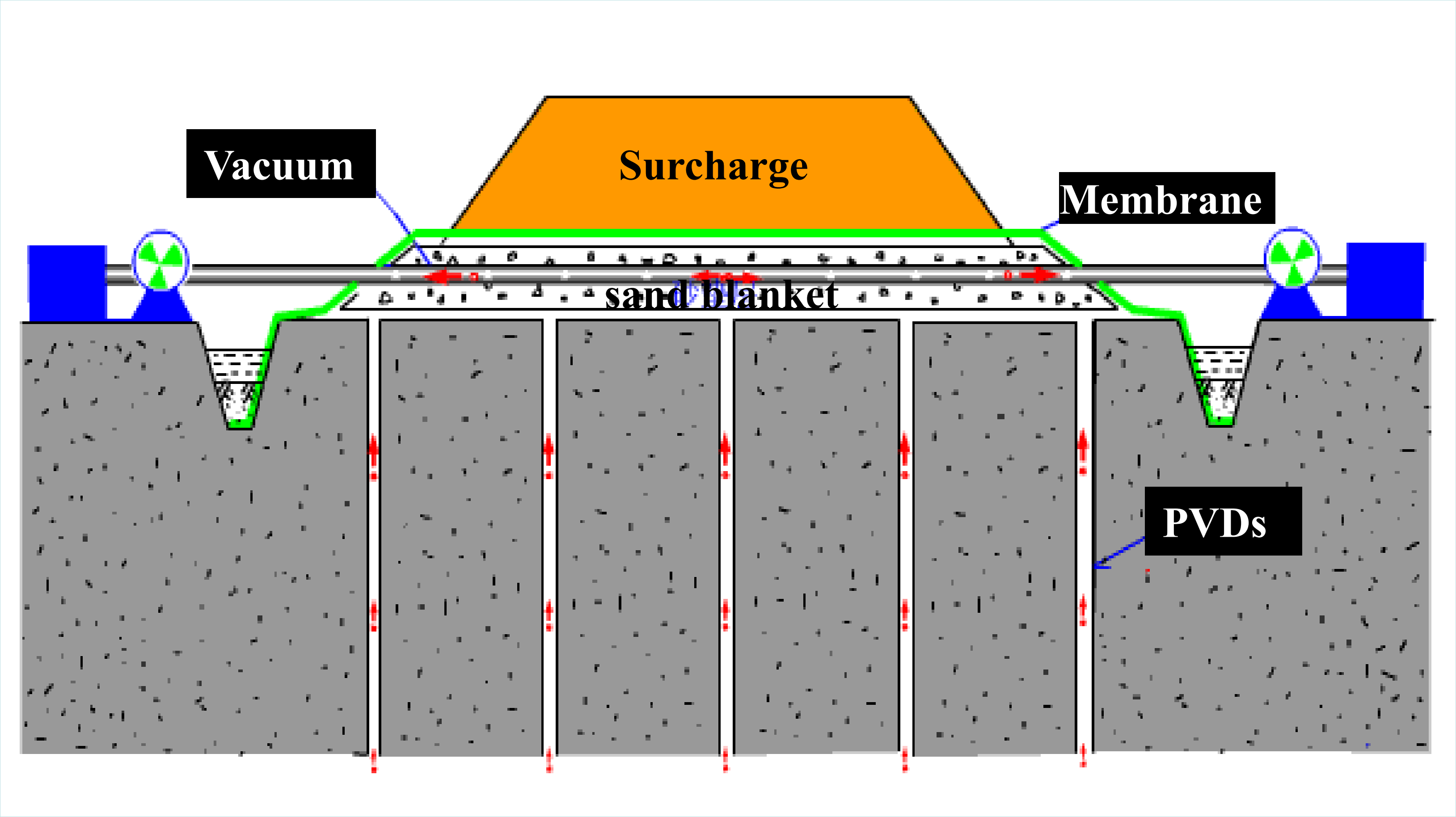


Fig. 6.26: Piezometric Drawdowns (Initial and Assumed Final Values)



Sources : Prinya Nutalaya (AIT), Dept. Mineral Resources, Japan International Cooperation Agency (JICA)

Principle



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

Full scale field tests of prefabricated vertical drains (PVD) for the Second Bangkok International Airport

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

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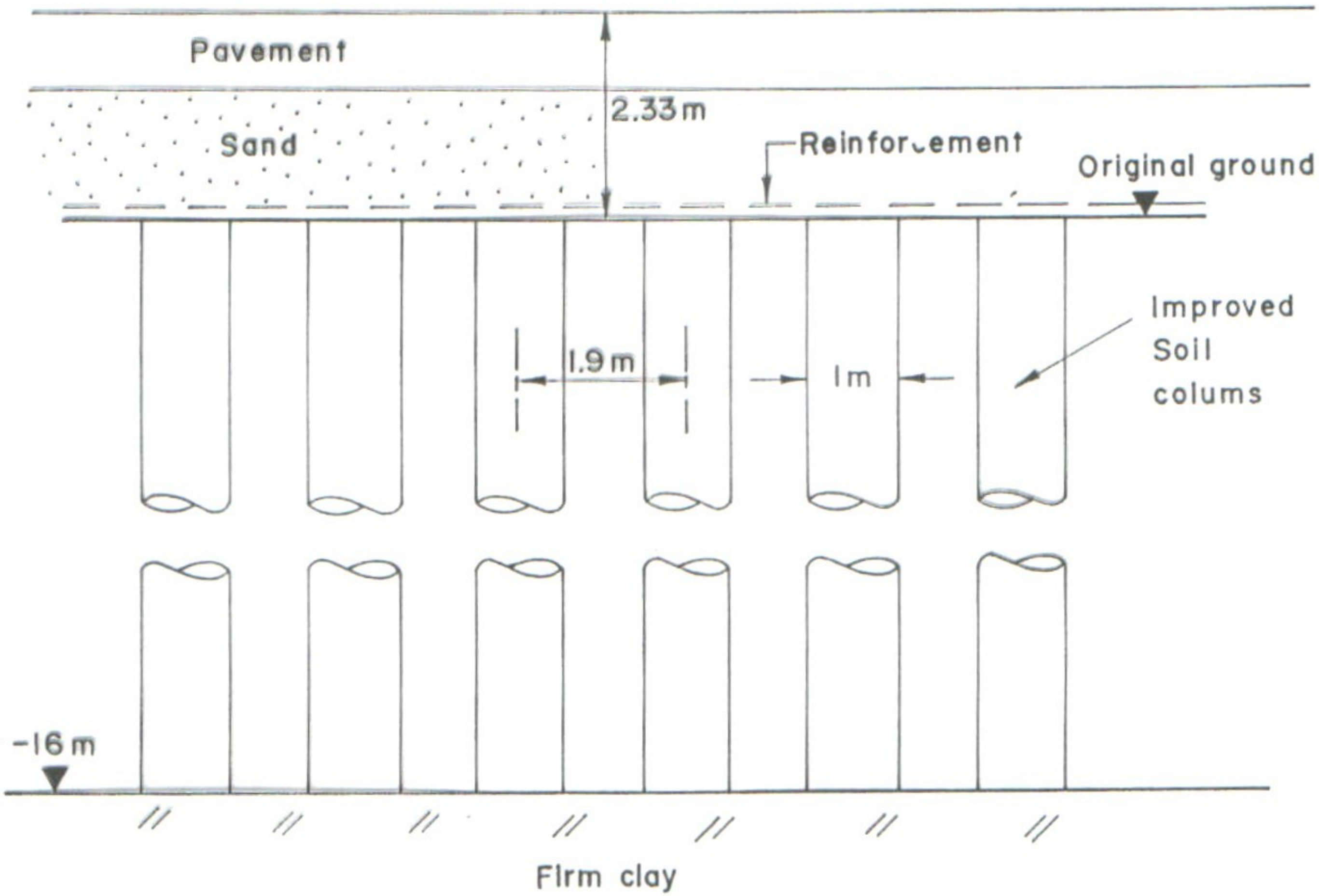
Alternative Design Concepts

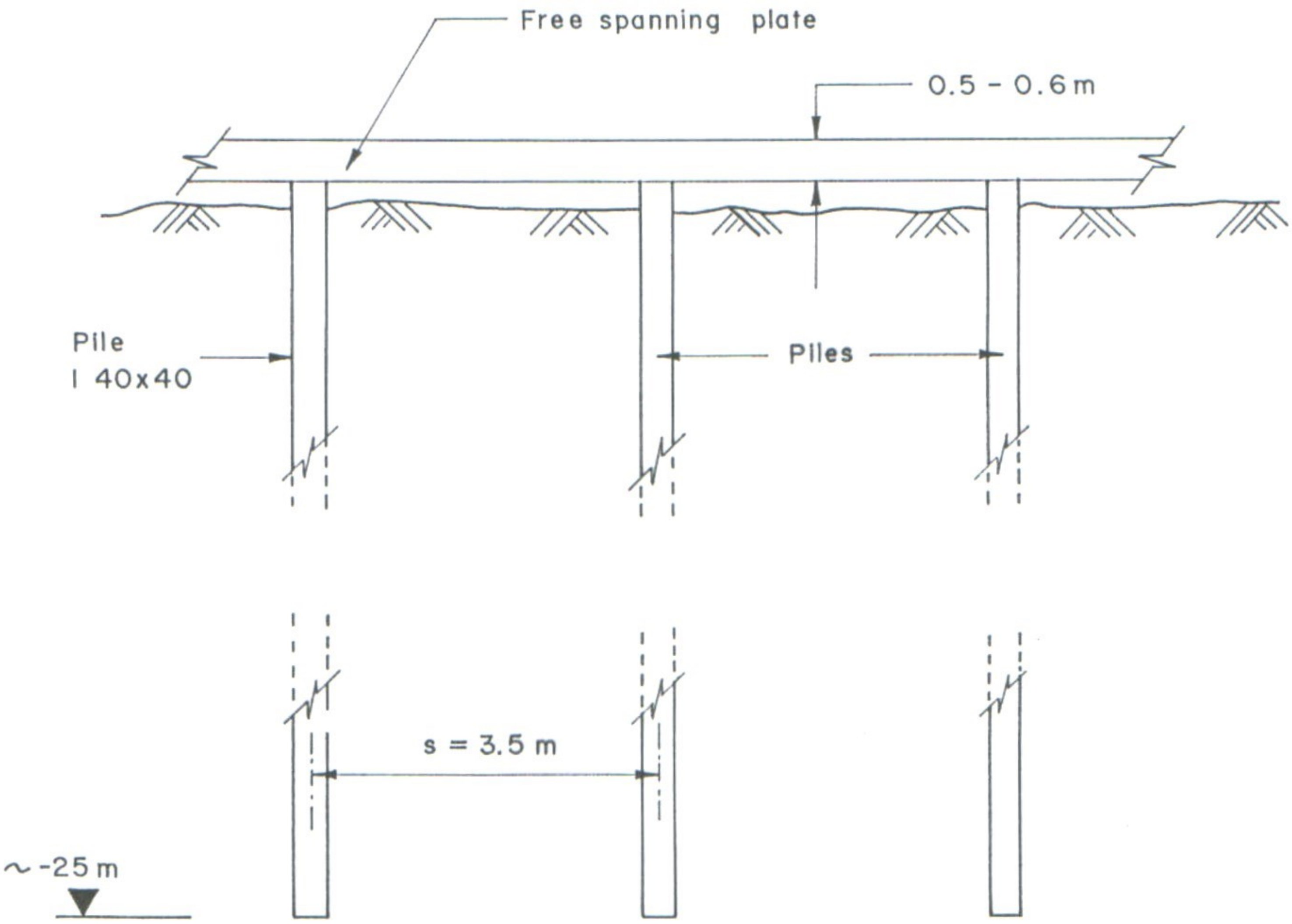
- Preçonsolidation with Vertical Drain
- Deep Soil Improvement
- Piles supporting a free spanning concrete slab
- Relief Piles with Caps
- Light Weight Fill Material

DEEP GROUND IMPROVEMENT
PROPOSED CONCEPTUAL DESIGN

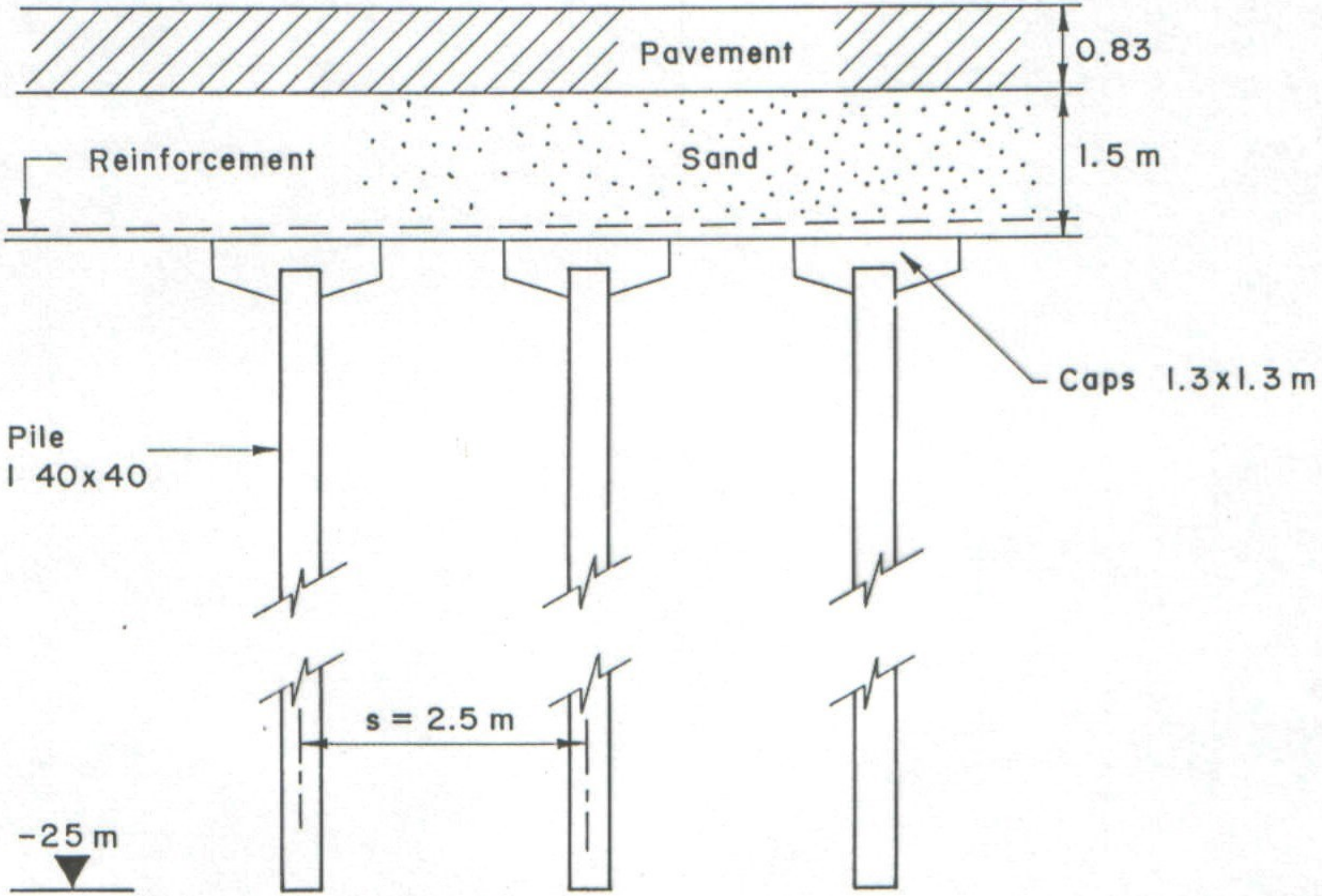


Reinforcement: Minimum tension capacity = 10t/m

Improved soil columns: $q_u \geq 200 t/m^2$

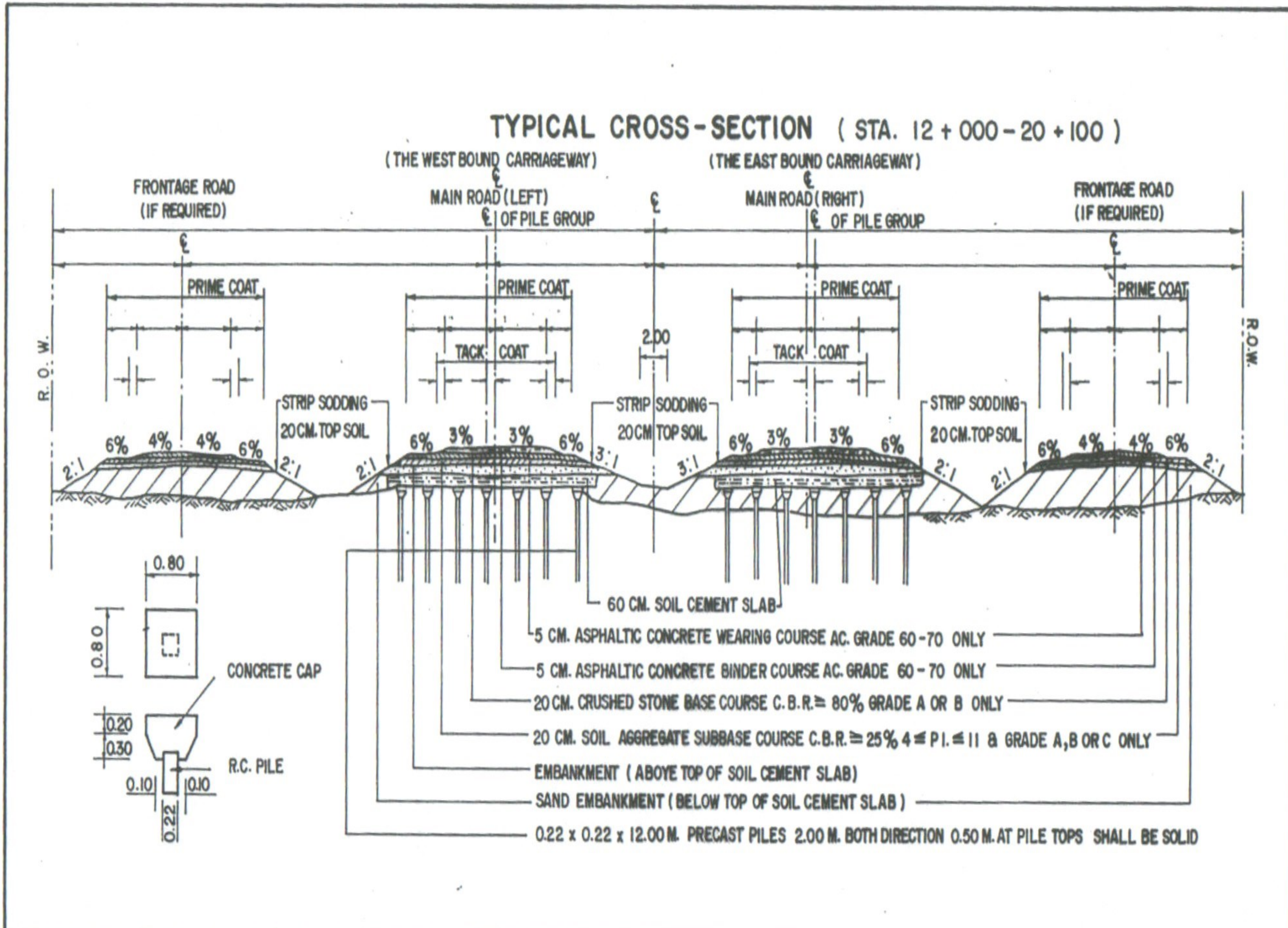


Piles: Allowable load 103 t



Piles: Allowable load 103t

Reinforcement : Woven polyester tension capacity 25 t/m


TYPICAL CROSS-SECTION (STA. 12 + 000 - 20 + 100)
(THE WEST BOUND CARRIAGEWAY)
(THE EAST BOUND CARRIAGEWAY)
FRONTAGE ROAD (IF REQUIRED)
MAIN ROAD (LEFT)
OF PILE GROUP
MAIN ROAD (RIGHT)
OF PILE GROUP
FRONTAGE ROAD (IF REQUIRED)
R. O. W.
R.O.W.
PRIME COAT
TACK COAT
2.00
STRIP SODDING
20 CM. TOP SOIL
6% 4% 4% 6%
6% 3% 3% 6%
2:1
3:1
0.80
0.80
CONCRETE CAP
0.20
0.30
R.C. PILE
0.10
0.10
0.22
60 CM. SOIL CEMENT SLAB
5 CM. ASPHALTIC CONCRETE WEARING COURSE AC. GRADE 60-70 ONLY
5 CM. ASPHALTIC CONCRETE BINDER COURSE AC. GRADE 60 - 70 ONLY
20 CM. CRUSHED STONE BASE COURSE C.B.R.≥ 80% GRADE A OR B ONLY
20 CM. SOIL AGGREGATE SUBBASE COURSE C.B.R.≥ 25% 4 ≤ P.I. ≤ 11 & GRADE A, B OR C ONLY
EMBANKMENT (ABOVE TOP OF SOIL CEMENT SLAB)
SAND EMBANKMENT (BELOW TOP OF SOIL CEMENT SLAB)
0.22 x 0.22 x 12.00 M. PRECAST PILES 2.00 M. BOTH DIRECTION 0.50 M. AT PILE TOPS SHALL BE SOLID

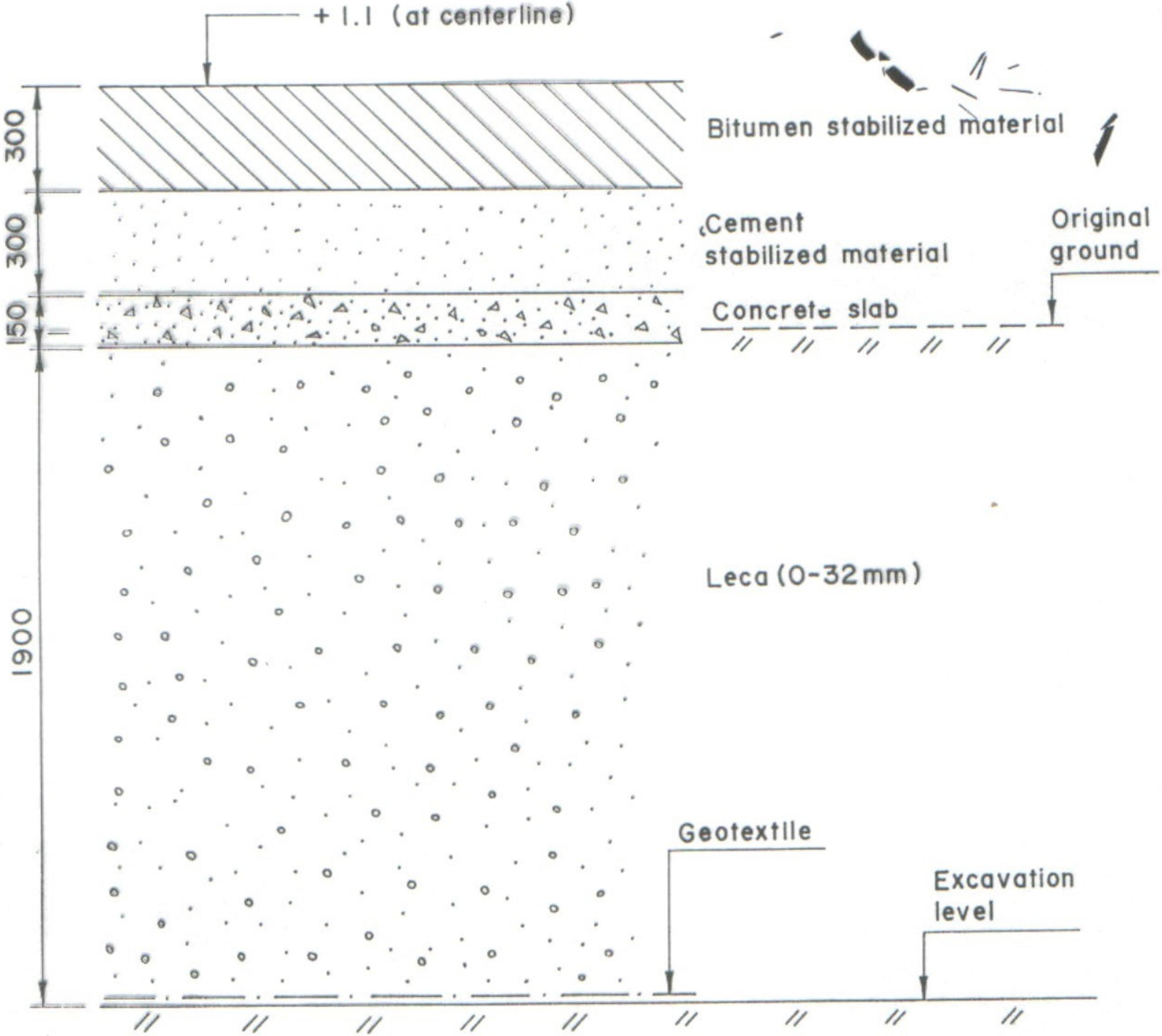

+ 1.1 (at centerline)
Bitumen stabilized material
Cement stabilized material
Original ground
Concrete slab
Leca (0-32mm)
Geotextile
Excavation level
300
300
150
1900

Fig. A-19 PVD Installation in Field

Fig. A-20 PVD Installation in Field

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.

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

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**

Specifications for Properties of PVD

Mechanical Properties for Survivability	SPECIFICATION	
	Federal Highway Authority, USA	Department of Highways, Thailand
Grab Tensile Strength (ASTM D-4672)	355N	350N
Puncture Strength (ASTM D-3728)	220N	200N
Burst Strength (ASTM D-3786)	900 kPa	900 kPa
Trapezoidal Shear Strength (ASTM D-4637)	110N	100N

Specifications Critreria 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

Fig. A-14 Modified Triaxial Cell: Twisted Specimen

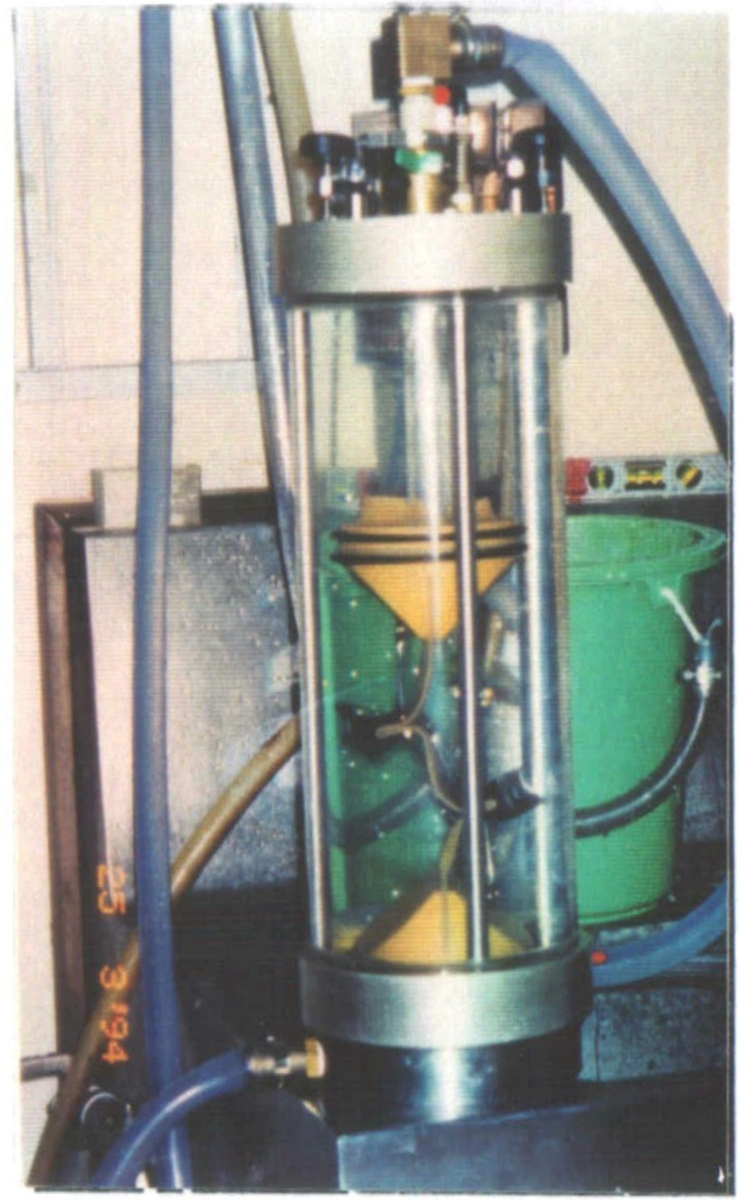


Fig. A-15 Modified Triaxial Cell: Clamped Specimen

Fig. A-16 Large Consolidometer with Central Drainage

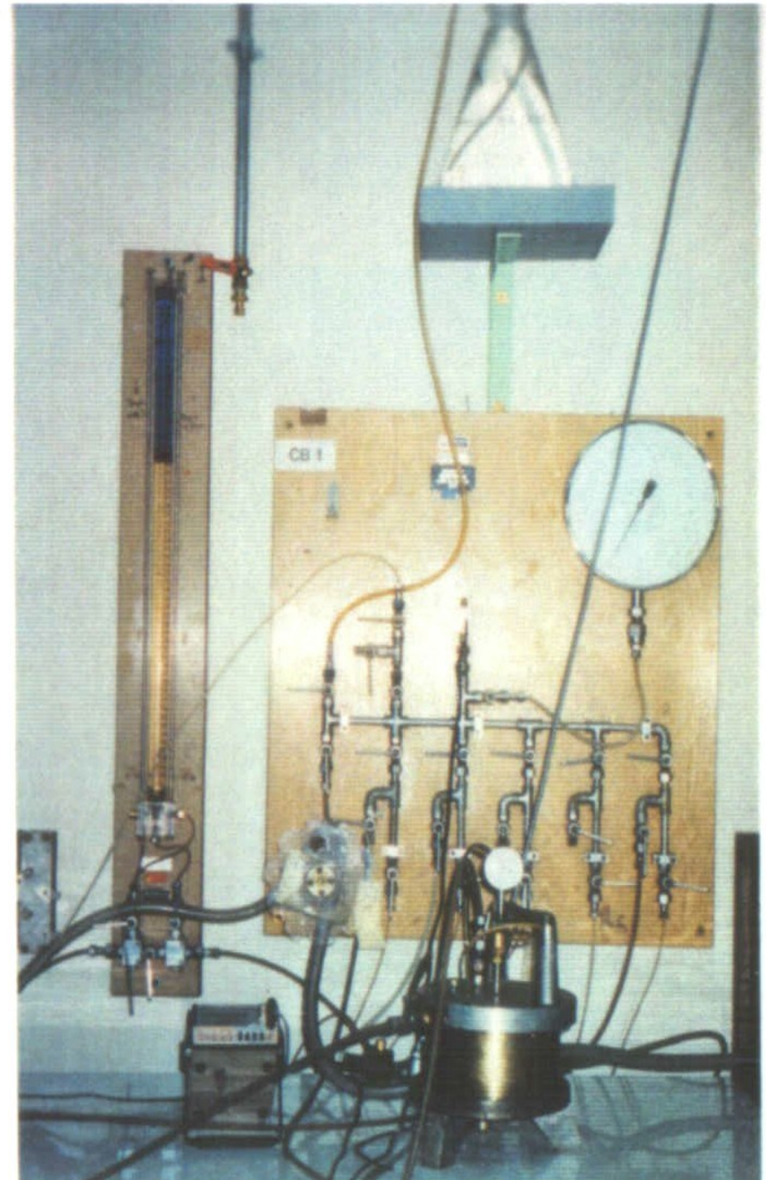


Fig. A-17 Rowe Cell Test Apparatus

Salient Features

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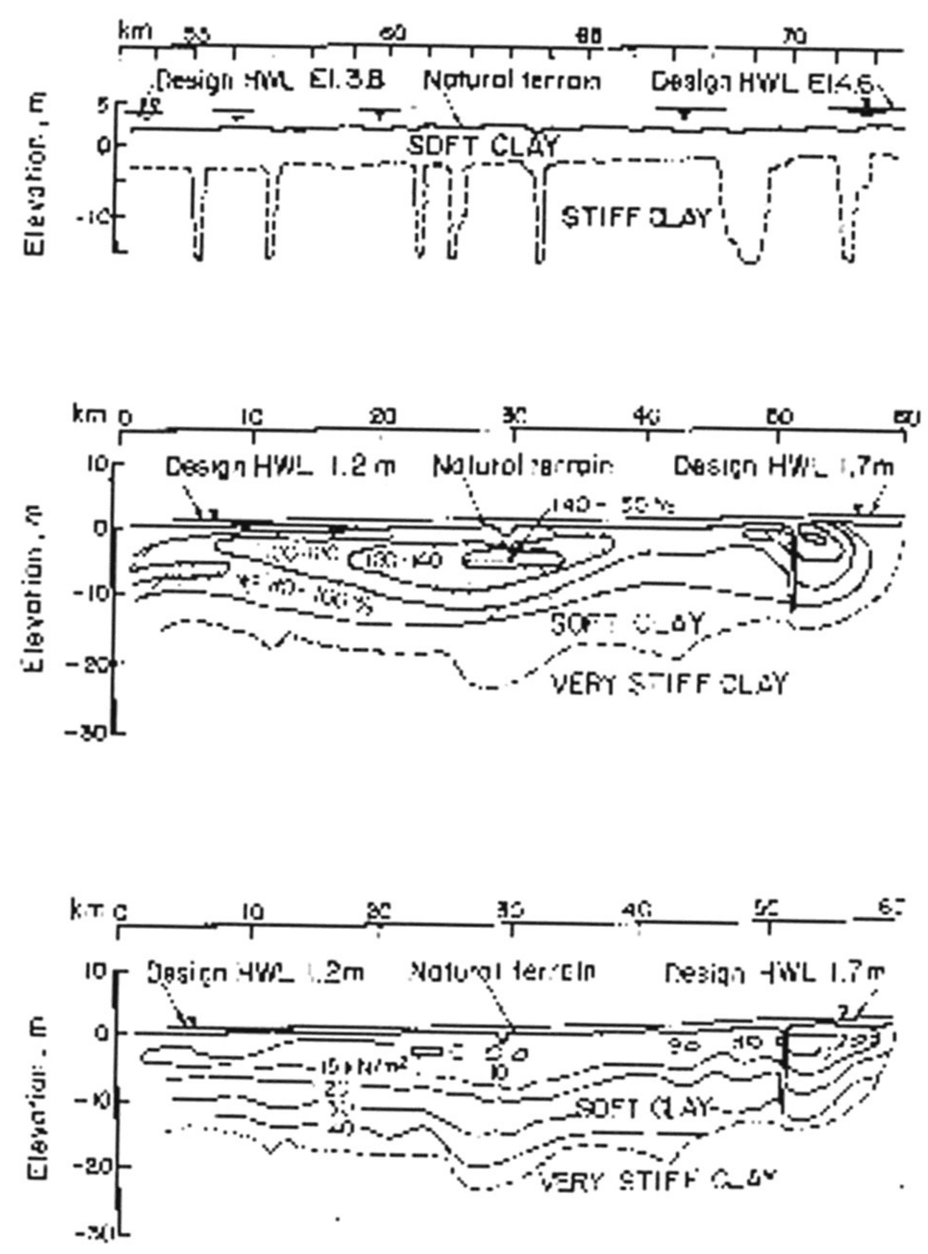


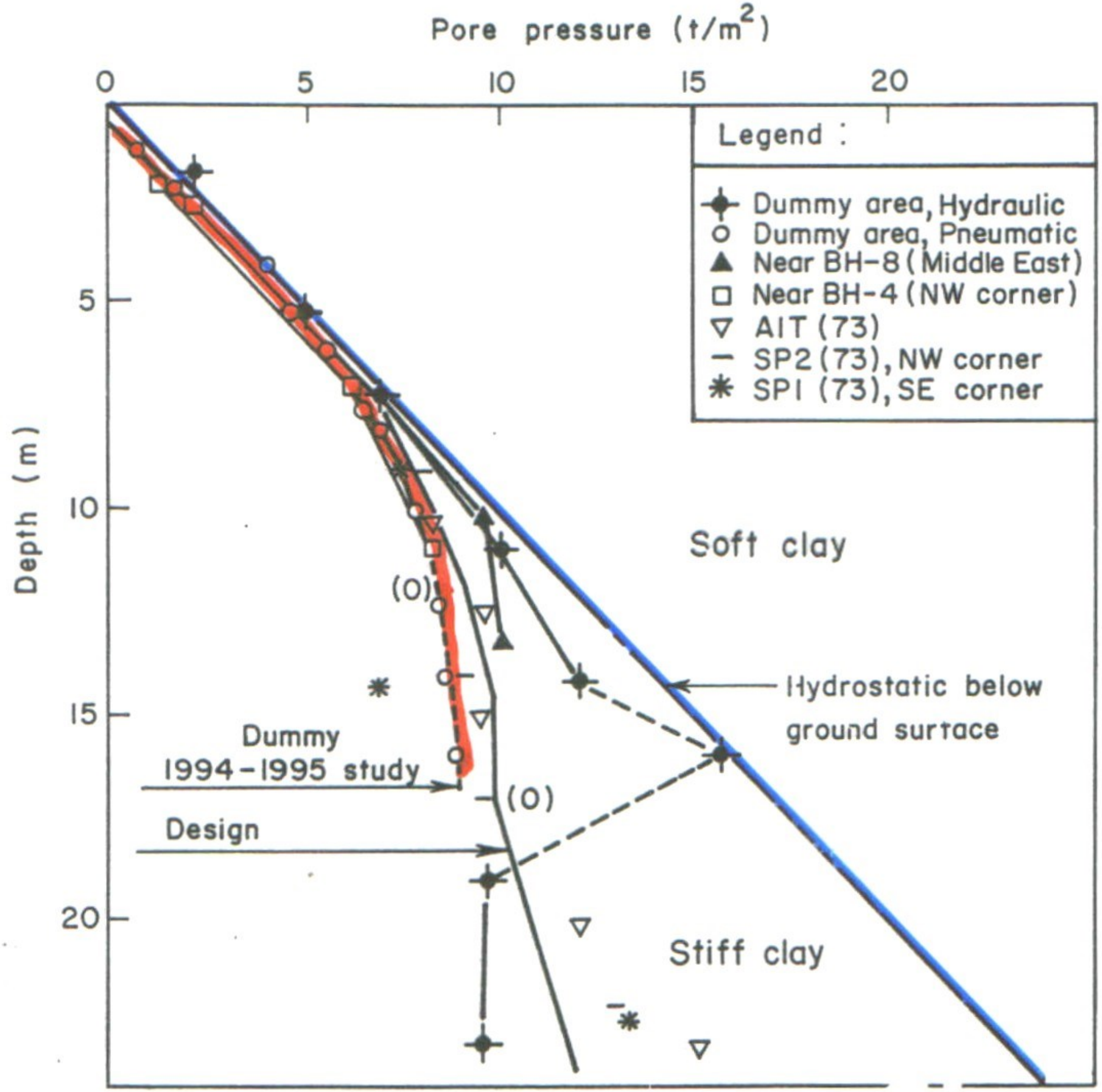
km
Design HWL El.3.8
Natural terrain
Design HWL El.4.6
Elevation, m
SOFT CLAY
STIFF CLAY
km
Design HWL 1.2 m
Natural terrain
Design HWL 1.7m
Elevation, m
SOFT CLAY
VERY STIFF CLAY
km
Design HWL 1.2m
Natural terrain
Design HWL 1.7 m
Elevation, m
SOFT CLAY
VERY STIFF CLAY

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.**

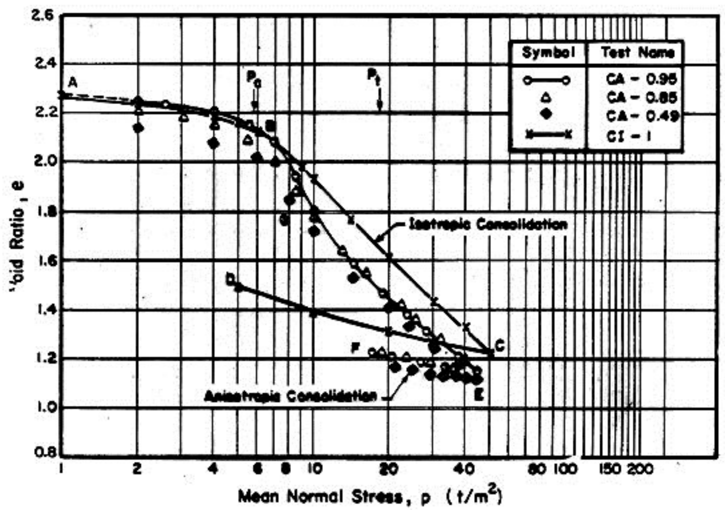
2. **The test embankments and the excavation were carried out to monitor the field settlements under various magnitude of surcharge and to ensure the stability of the embankment and the excavation on a full scale basis and to compare the values estimated from single element laboratory tests and small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

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

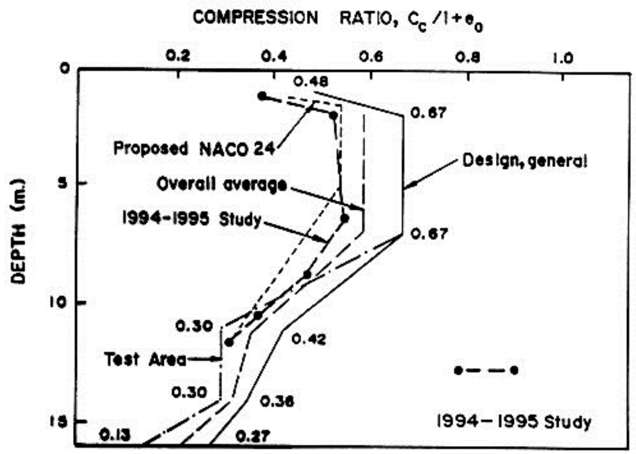
5. The piezometric draw-down due to subsidence made the computation of settlement from pore pressure dissipation difficult.
6. Computations need to convince that the degree of consolidation estimated from pore pressure dissipation and settlement measurements are comparable.
7. Undrained strength measurements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



Variation of Piezometric Pressures with Depth



Typical Consolidation and Swelling Curves



NOTE :
Design values are from proposed
Correlation, $\frac{C_c}{1+e_o} = \left(\frac{w\%}{100}\right) \times 0.58$

Fig. 5(e) Variation of Compression Ratio with Depth

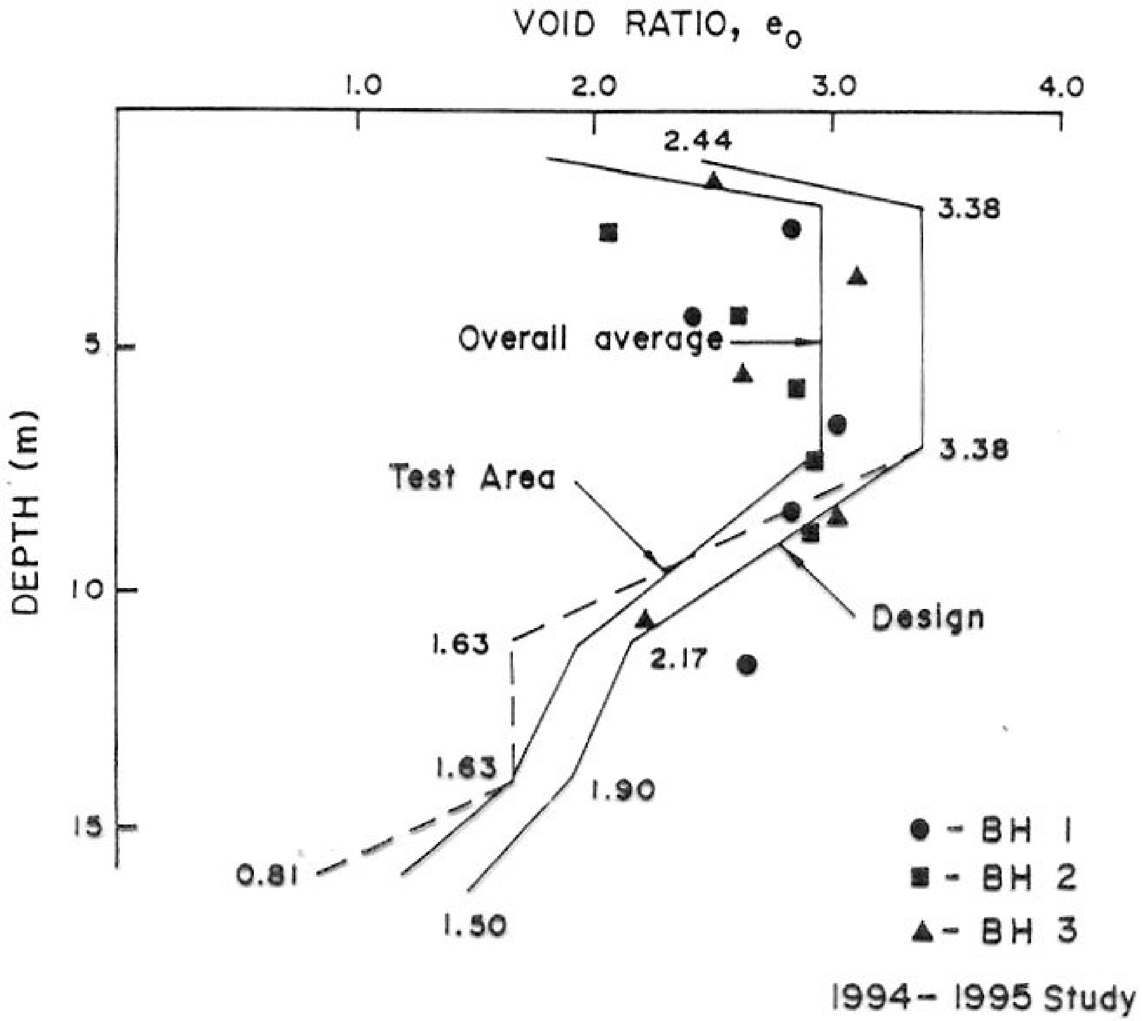
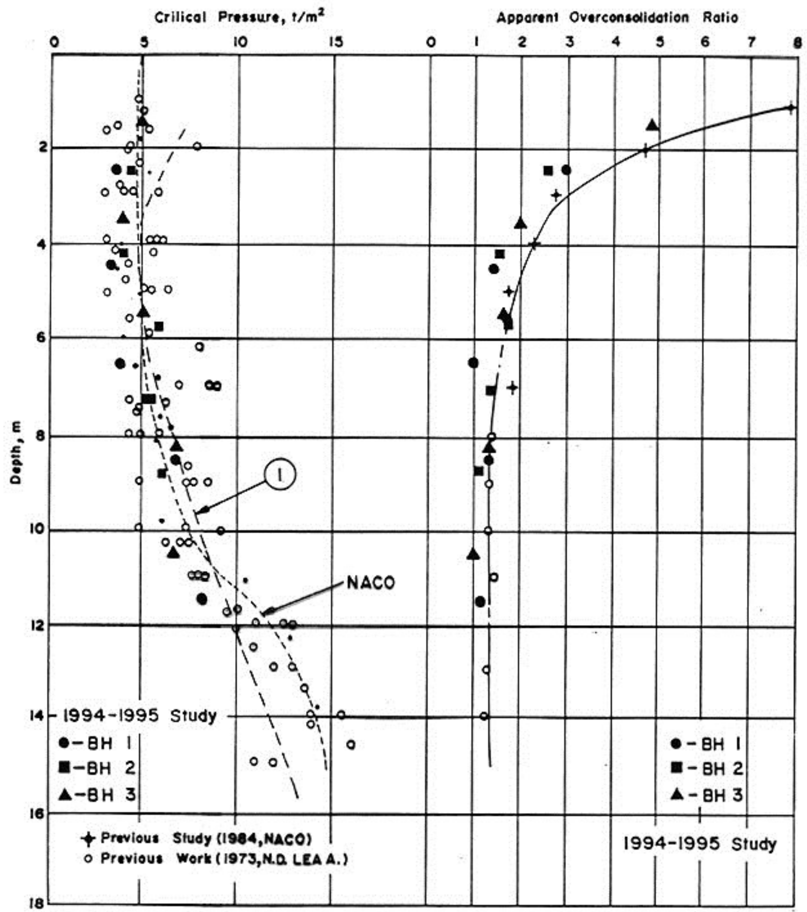
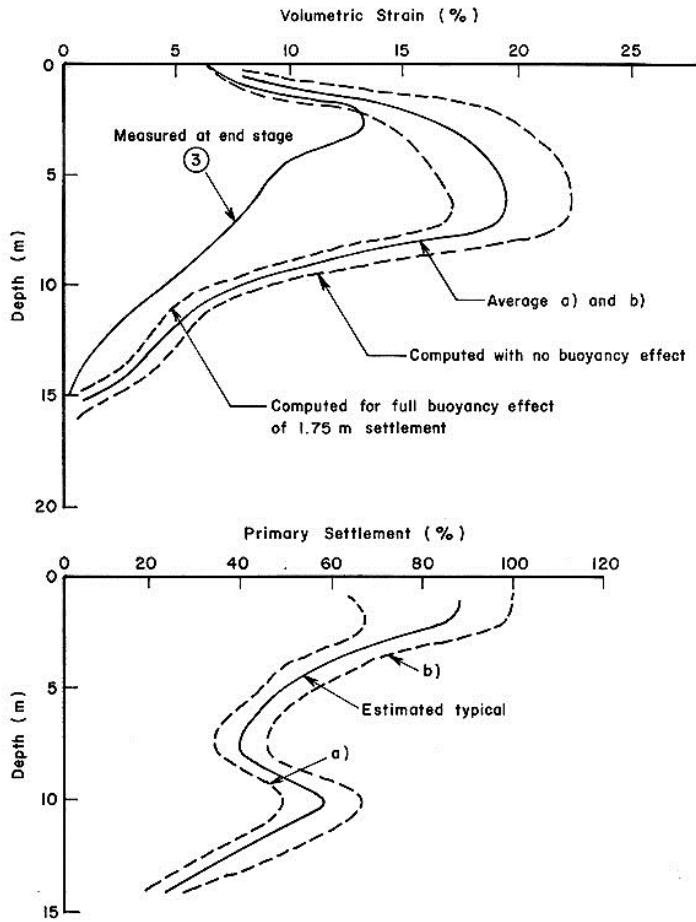


Fig. 5(c) Variation of Void Ratio with Depth

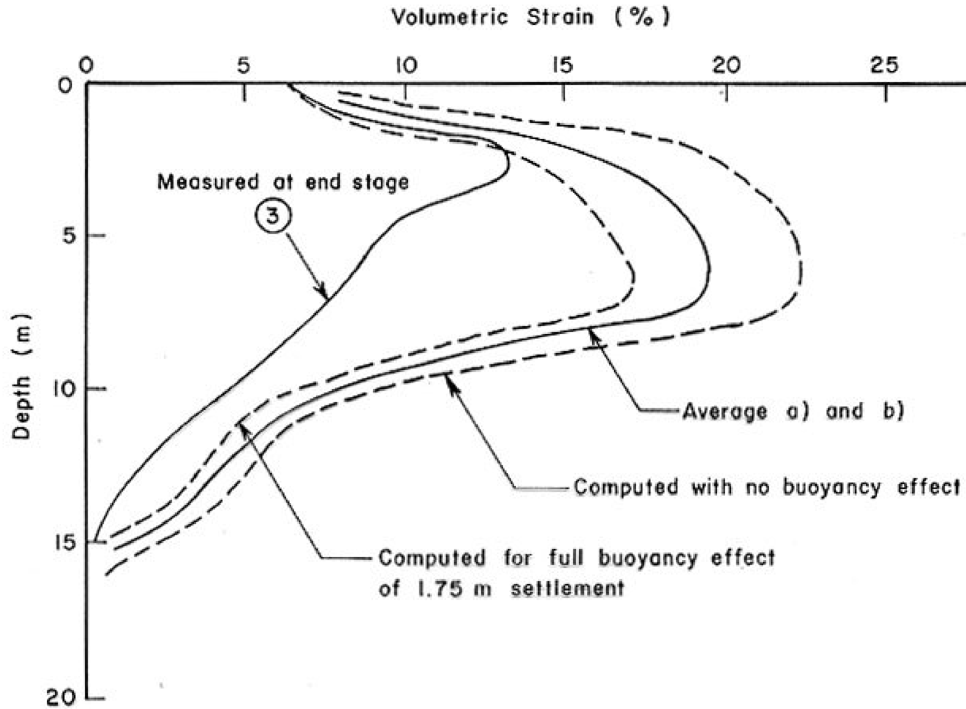


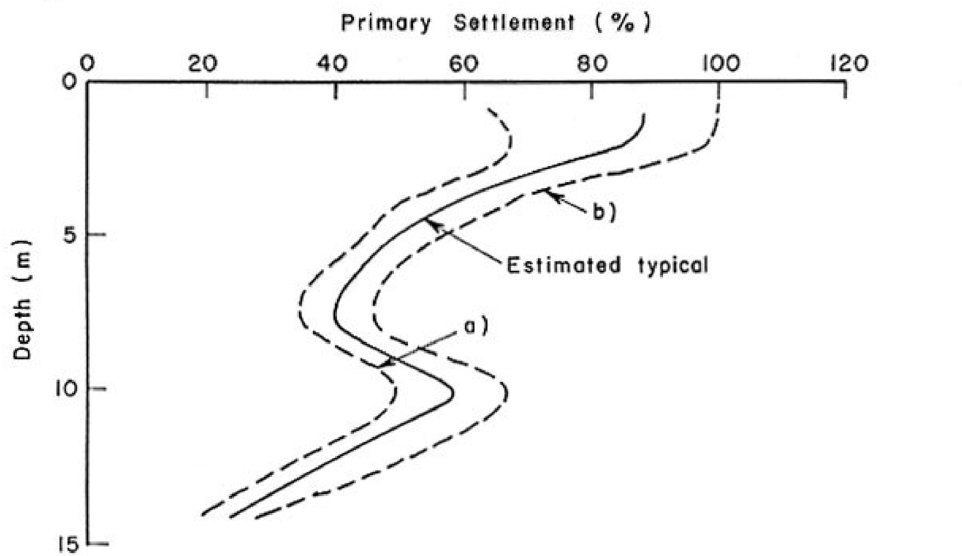
(1) This is based on correlation between OCR, (s_u/σ'_{vo}) vane and I_p according to Bjerrum (1973) and Aas et al. (1985)

Fig. 5(f) Variation of Critical Pressure and Apparent Overconsolidation Ratio with Depth



Volumetric Strain (%)
0
5
10
15
20
25
30
Depth (m)
0
5
10
15
20
Measured at end stage
3
Average a) and b)
Computed with no buoyancy effect
Computed for full buoyancy effect
of 1.75 m settlement
Primary Settlement (%)
0
20
40
60
80
100
120
Depth (m)
0
5
10
15
b)
Estimated typical
a)


Volumetric Strain (%)
0
5
10
15
20
25
30
Depth (m)
0
5
10
15
20
Measured at end stage
3
Average a) and b)
Computed with no buoyancy effect
Computed for full buoyancy effect
of 1.75 m settlement


Primary Settlement (%)
0
20
40
60
80
100
120
Depth (m)
0
5
10
15
b)
Estimated typical
a)

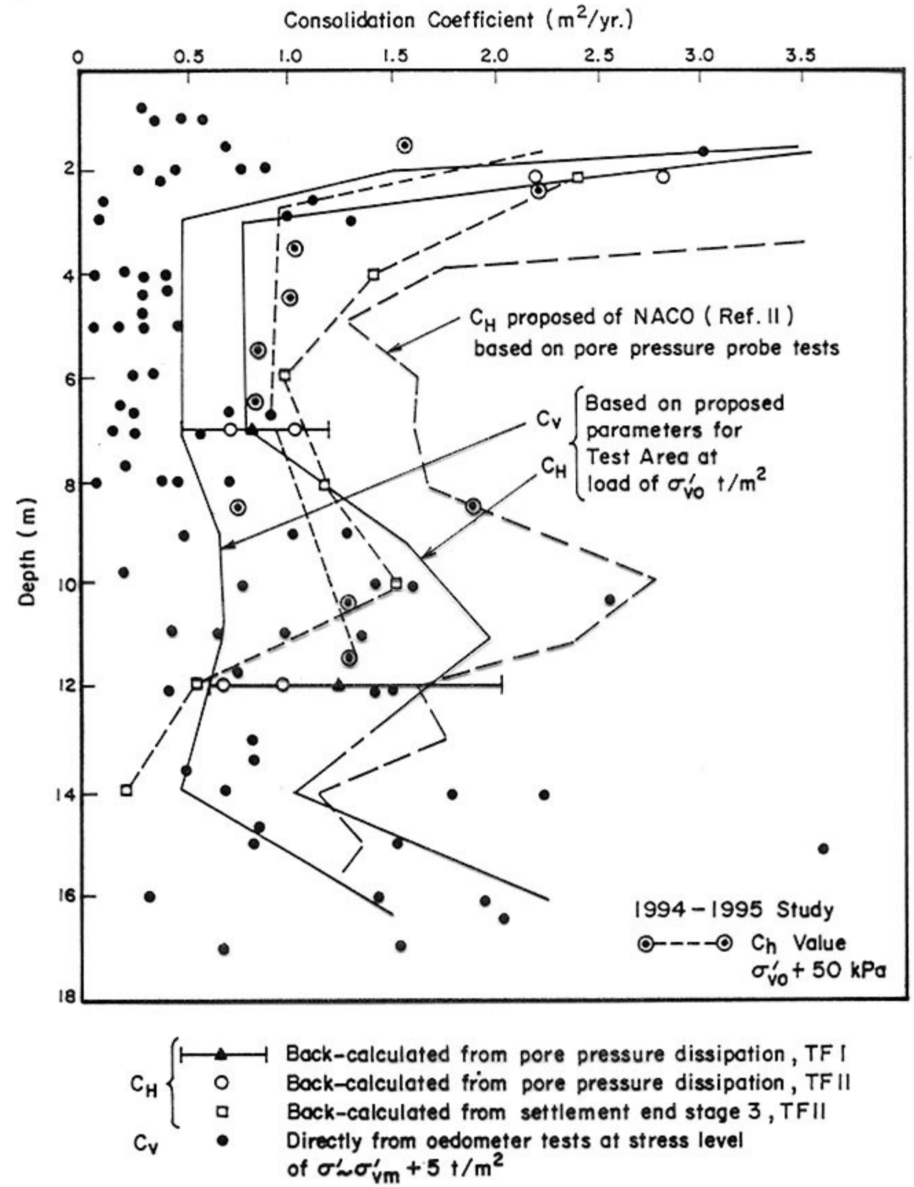


Fig. 5(h) Variation of Coefficient of Consolidation - c_h values

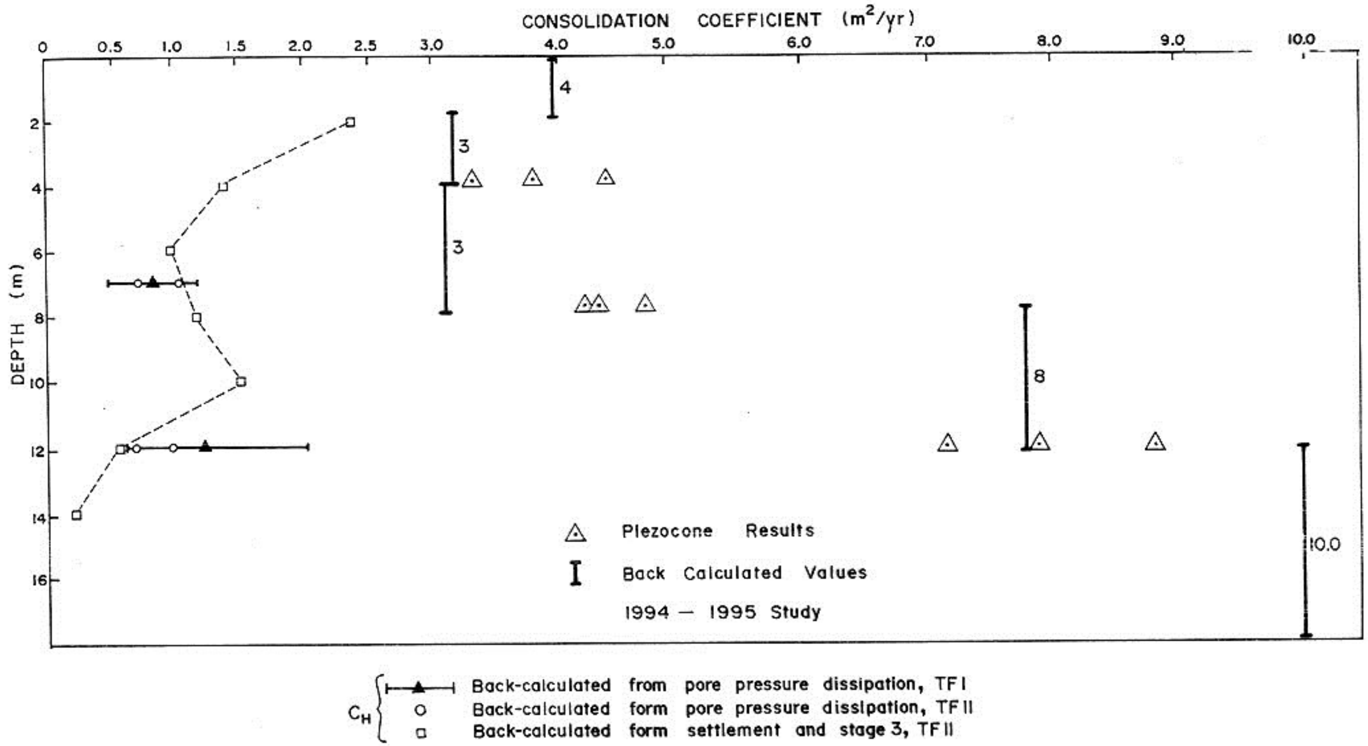


Fig. 2.17 Variation of Horizontal Coefficient of Consolidation (c_h)

Summary of settlement Parameters

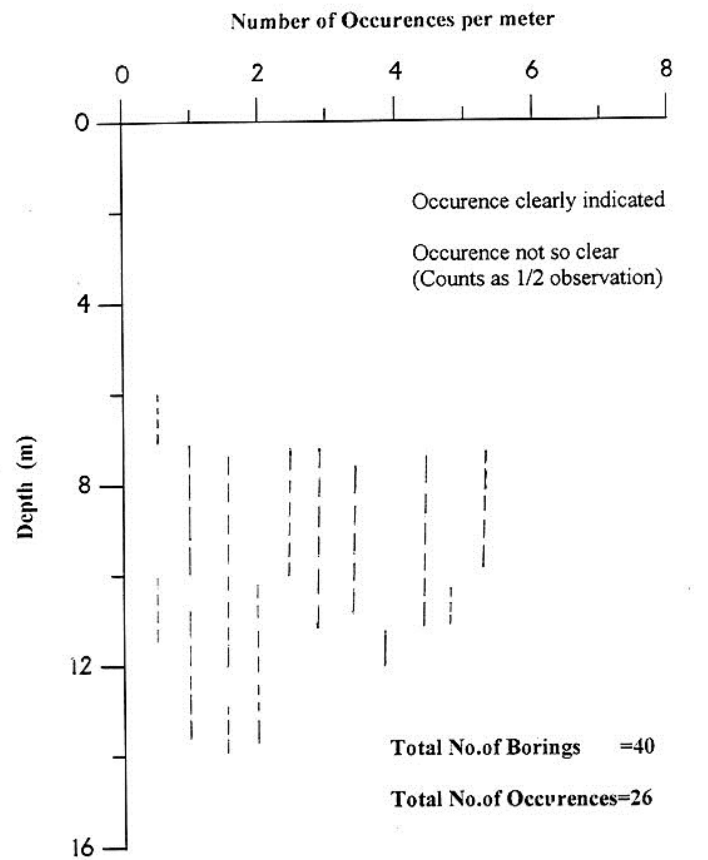
Depth (m)	σ'_{vo} (t/m^2)	P'_c (t/m^2)	e_o	$\frac{C_c}{1 + e_o}$	Permeability ($x\ 10^{-7}$ cm/sec)			Typ. Consol. Coeff. (m^2/yr.) (at p' σ'_{vo}+ 5 t/m^2)	
					k_{vo}	k_{ho}	C_k	C_v	C_h
1	1.15	5.00	2.44	0.48	10.0	10.0	1.17	6.00	6.00
2	1.70	5.00	3.38	0.67	4.0	6.8	1.52	1.50	2.50
4	2.62	5.00	3.38	0.67	1.5	2.55	1.52	0.45	0.77
7	4.00	5.80	3.38	0.67	1.5	2.55	1.52	0.45	0.77
11	7.80	8.40	2.17 (1.63)	0.42 (0.30)	0.5 (0.24)	1.45 (0.7)	0.98 (0.73)	0.65 (0.67)	1.88 (1.94)
14	11.60	11.6	1.90 (1.63)	0.36 (0.30)	0.35 (0.17)	0.82 (0.4)	0.86 (0.73)	0.75 (0.43)	1.75 (1.00)
16	14.80	14.8	1.50 (0.81)	0.27 (0.13)	0.18 (0.11)	0.32 (0.2)	0.68 (0.36)	0.75 (1.27)	1.36 (2.30)

Summary of Main Settlement Parameters in the Soft Clay Layer

Depth (m)	σ'_{vo}	P'_c	CR	Permeability (x 10^{-7} cm/sec)		Typ. Consol. Coeff. (m^2/yr.) (at p' σ'_{vo} + 5 t/m^2)	
				k_{vo}	k_{ho}	C_v	C_h
1	1.15	5.00	0.48	10.0	10.0	6.00	6.00
2	1.70	5.00	0.67	4.0	6.8	1.50	2.50
4	2.62	5.00	0.67	1.5	2.55	0.45	0.77
7	4.00	5.80	0.67	1.5	2.55	0.45	0.77
11	7.80	8.40	0.42	0.5	1.45	0.65	1.88
14	11.60	11.6	0.32	0.35	0.82	0.75	1.75
16	14.80	14.8	0.27	0.18	0.32	0.75	1.36

Calculated Total Primary Consolidation Settlements

Applied Preload (t/m^2)	Total Consolidation Settlements (m)
5.0	1.12
6.0	1.54
6.5	1.72
6.85	1.84
7.0	1.90
7.5	2.06



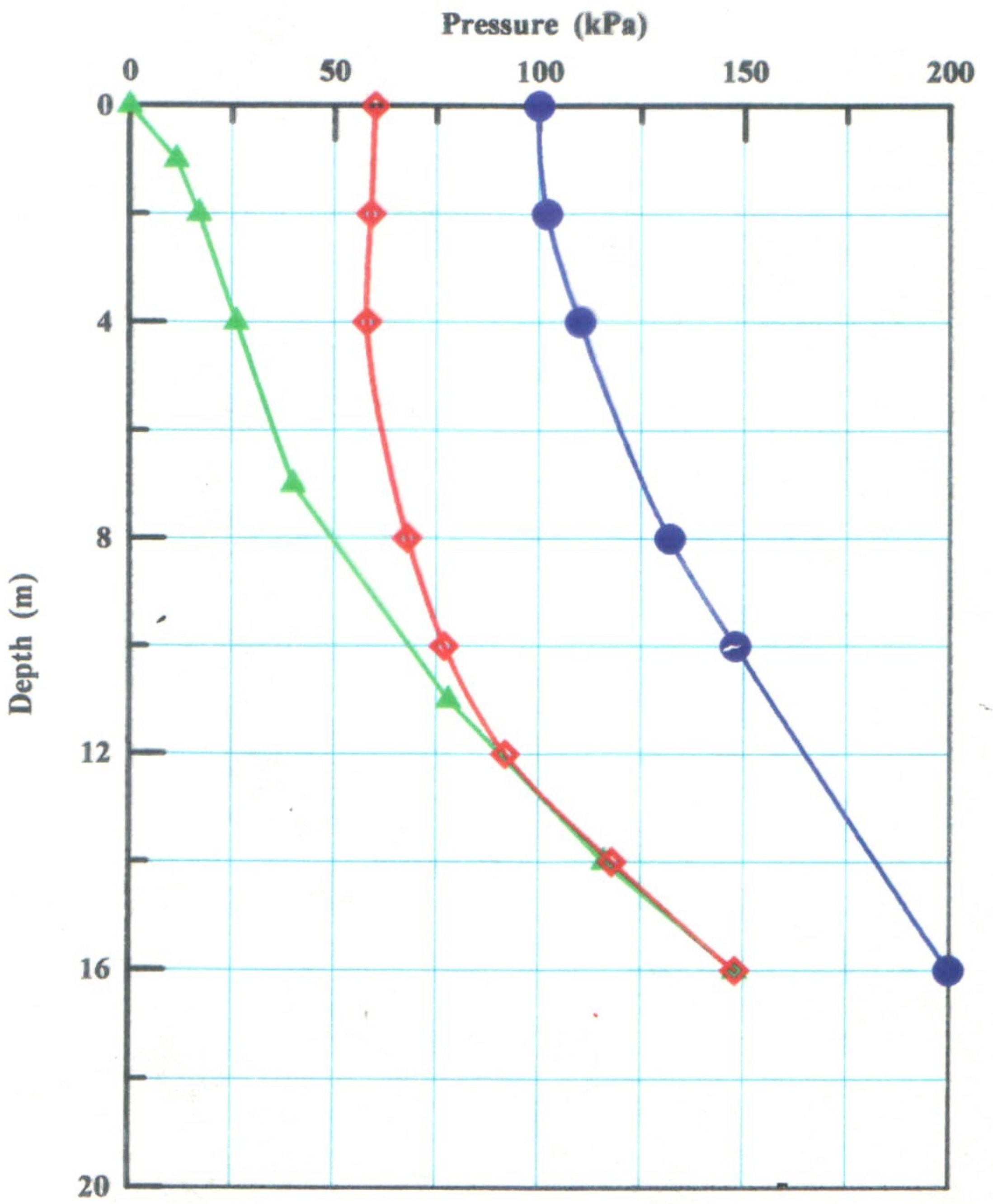
Statistical Distribution of Sand Lenses Based on Pore Pressure Probe Tests

Soil Parameters used in Settlement and Stability Analysis

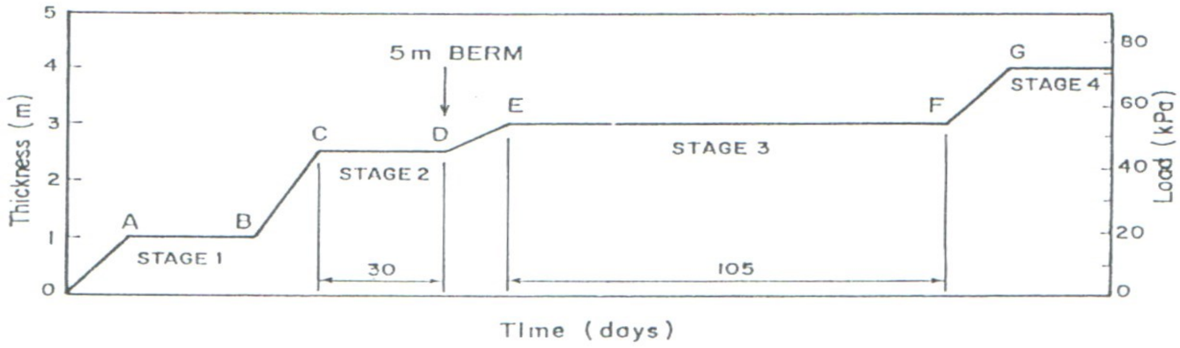
Zone	Depth (m)	Z_i (m)	Γ (kN/m^3)	$\bar{\sigma}_{vo}$ (kPa)	$\bar{\sigma}_p$ (kPa)	OCR	CR	RR	c_h (m^2/yr)	S_u (kPa)
1	0.3-2.0	0.85	16.0	12.1	75	6.20	0.30	0.030	10	12.5
2	2.0-5.0	4.2	14.5	28.5	50	1.75	0.55	0.055	3	10.0
	5.0-7.0									10.5
3	7.0-9.0	8.7	14.5	48.7	65	1.35	0.045	0.045	4	14.0
	9.0-11.0									17.5
4	11.0-13.0	11.7	16.0	64.7	87	1.35	0.035	0.035	4	23.0
5	13.0-15.0	13.7	16.5	77.2	105	1.35	0.030	0.030	4	30.0

Soil Parameters used in the F. E. M. Analysis

Depth (m)	λ	κ	M	ν	k_v 10^{-4} (m/day)	k_h 10^{-4} (m/day)	e_{cs}
0-2	0.34	0.07	1.2	0.25	25.9	25.9	2.80
2-7	0.90	0.18	0.9	0.30	5.9	10.1	5.90
7-12	0.50	0.10	1.0	0.25	2.6	5.2	4.00
12-15	0.34	0.07	1.2	0.25	1.0	2.1	3.00
15-22	0.10	0.02	1.2	0.20	0.3	0.5	1.30



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



SUMMARY OF THE STABILITY ANALYSIS

POINT	THICKNESS (m)	LOAD (kPa)	DURATION (Days)	FACTOR OF SAFETY with 5 kPa Load	FACTOR OF SAFETY 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

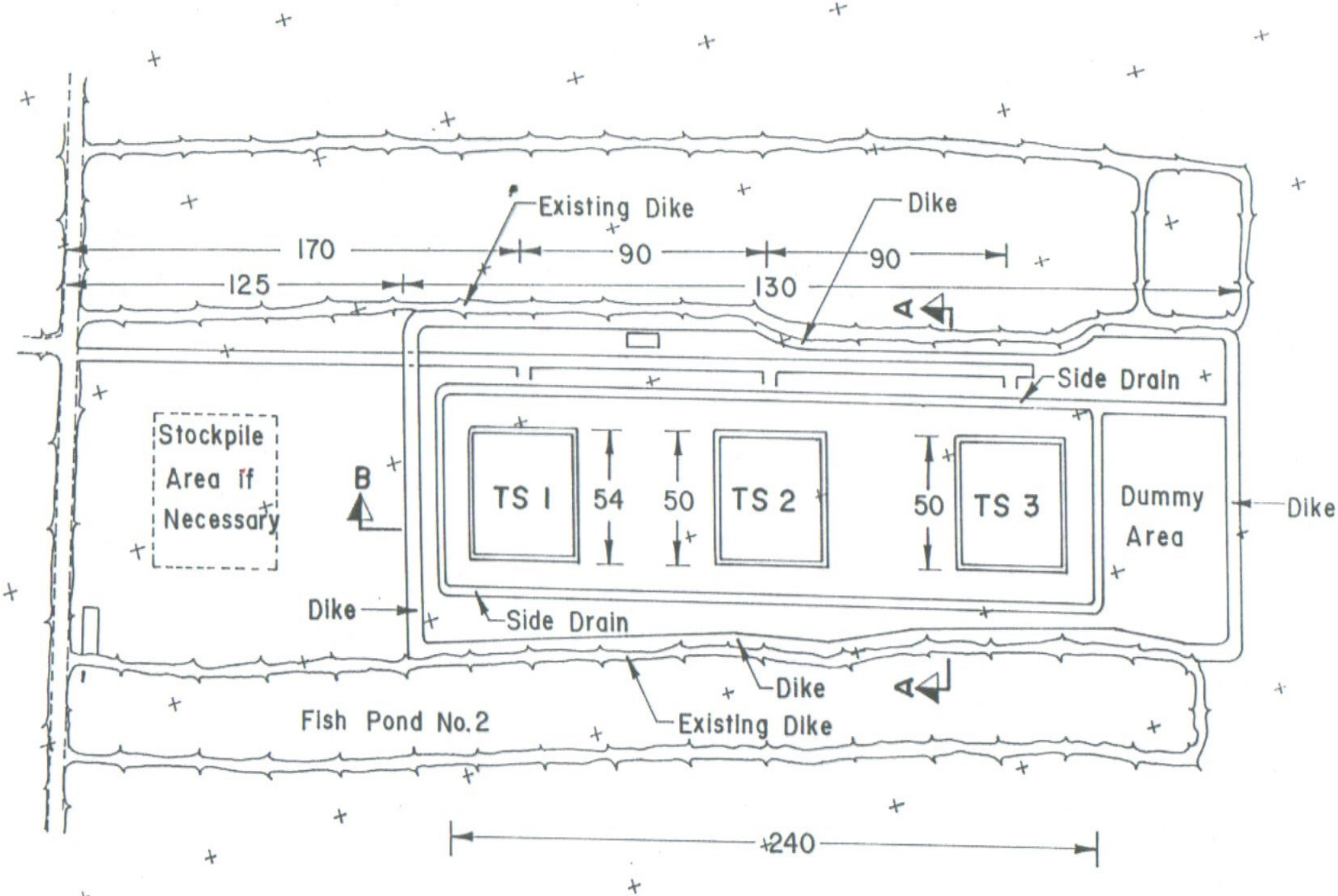
q_c : Embankment load at calculated time

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

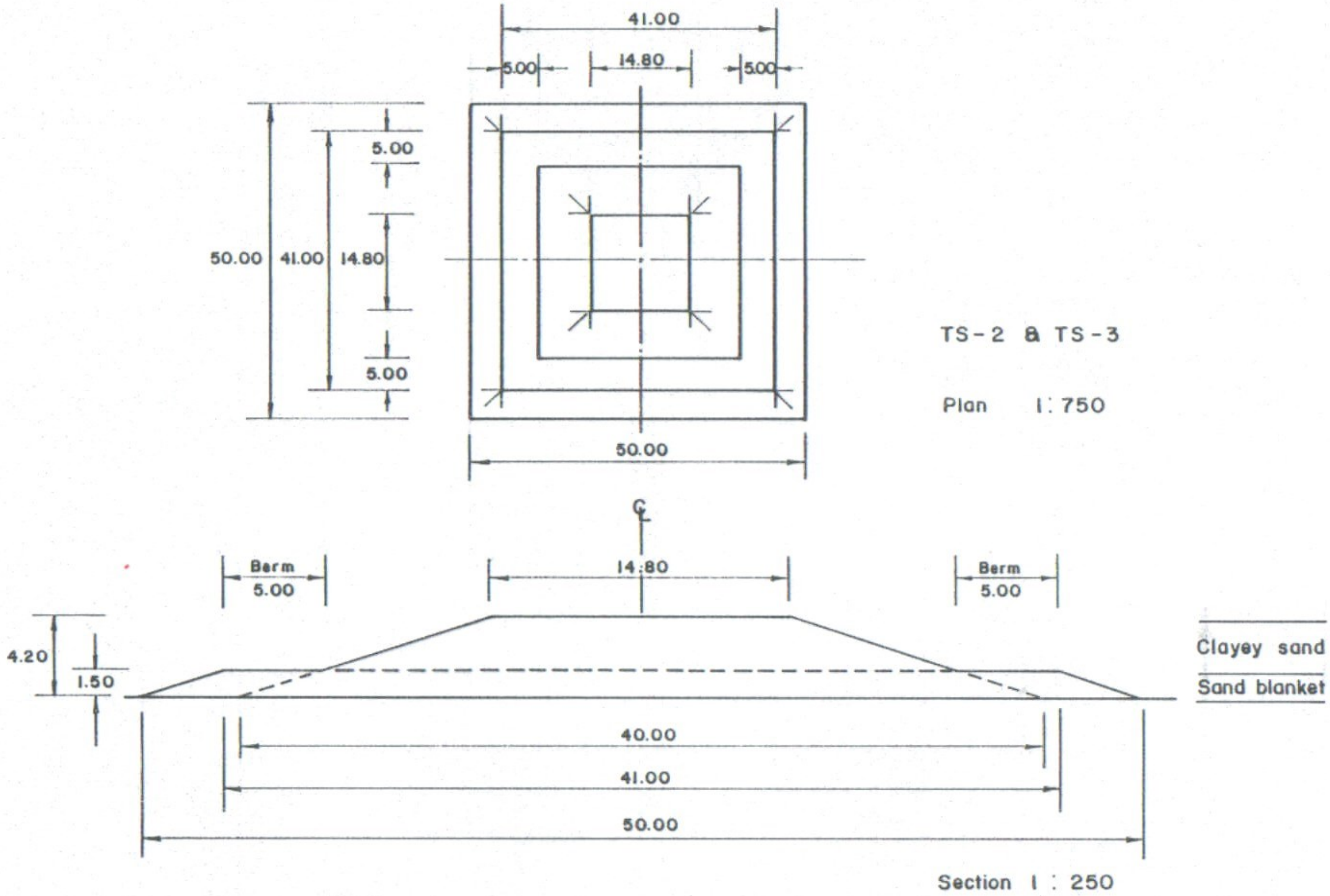
Fig. 4.18 Summary of Stability and Settlement Analyses for Embankment TS3 (with 1.0 m Drain Spacing)



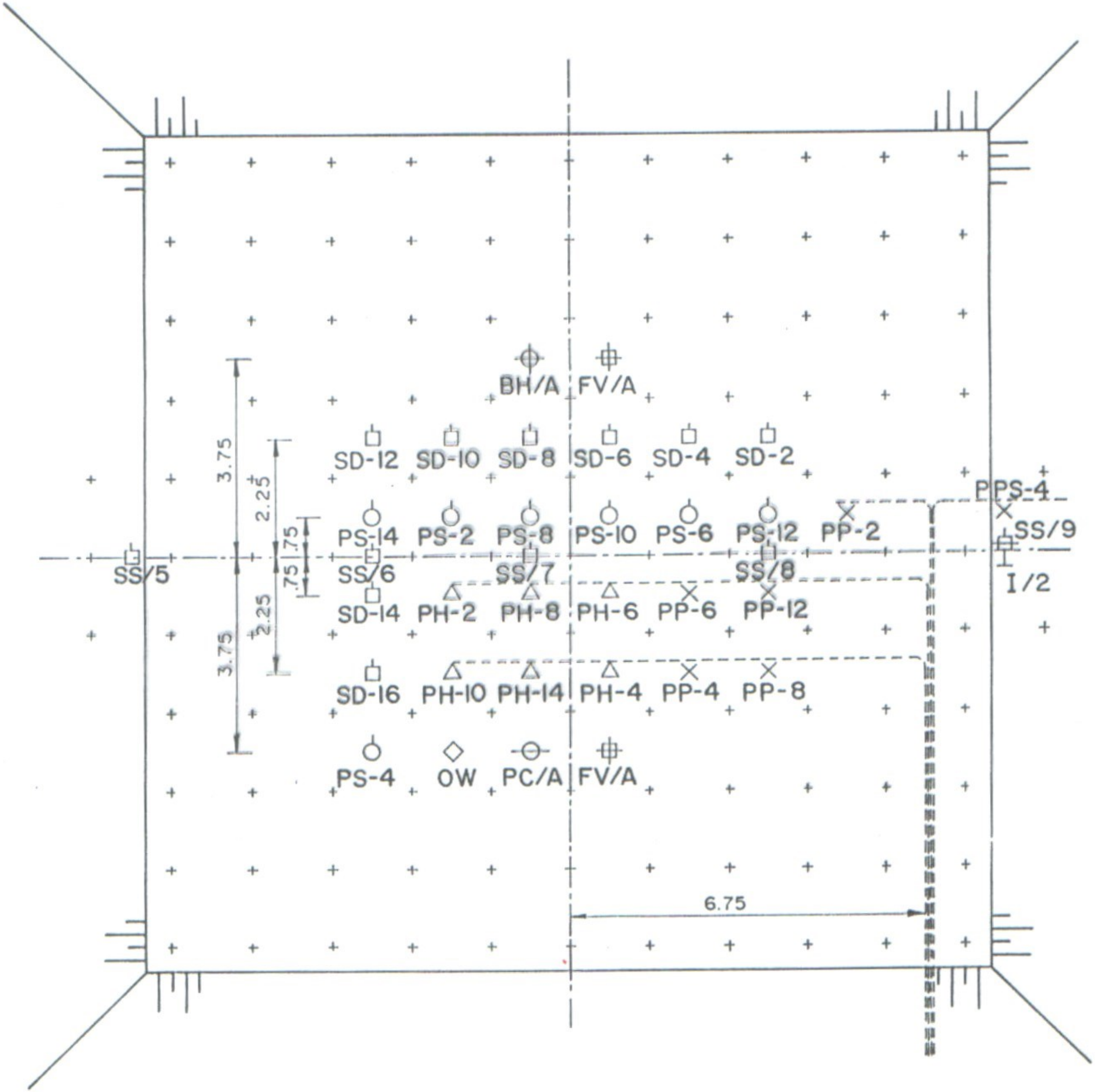
PLAN

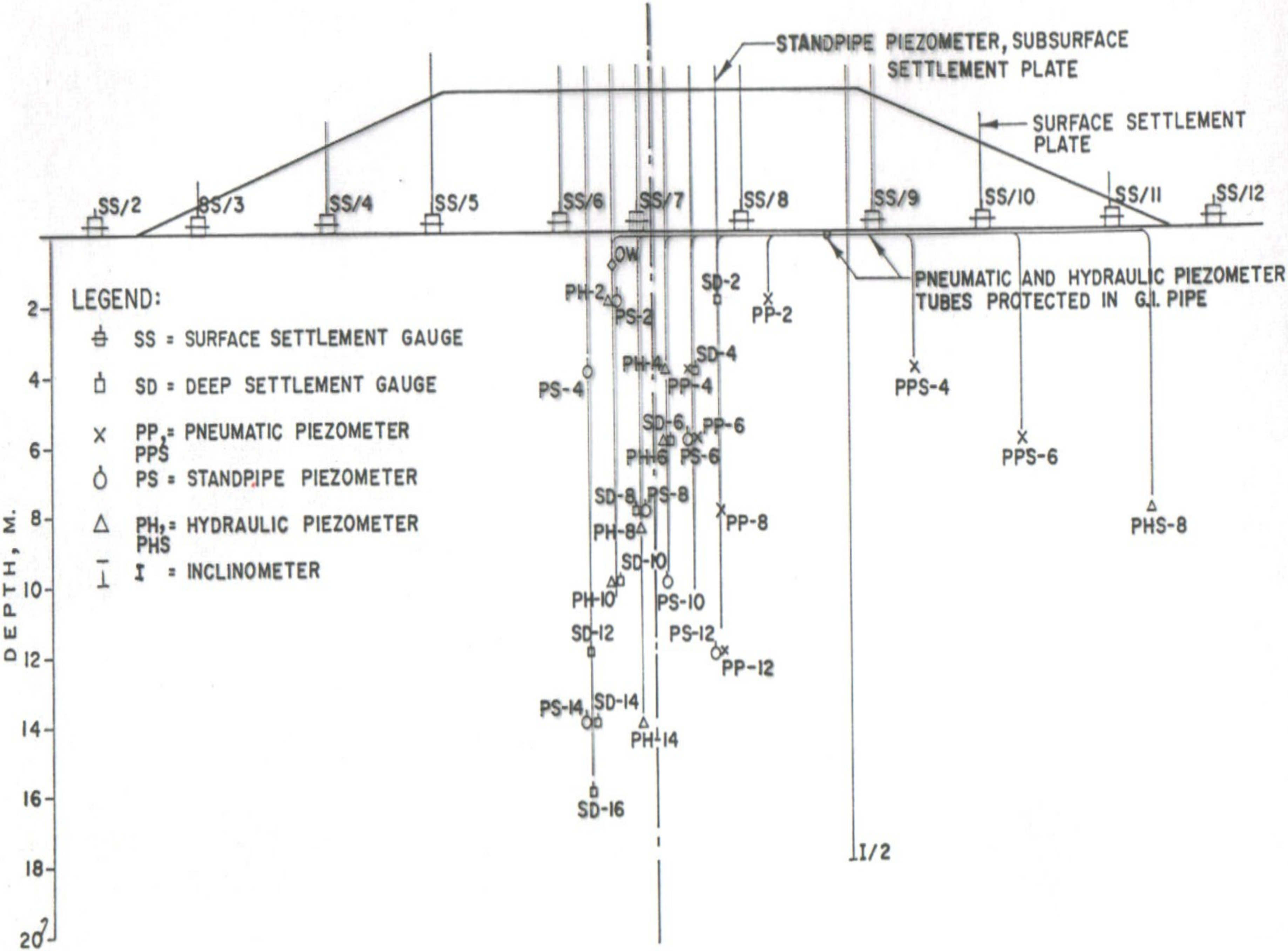
0 50 m.

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
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
DEPTH, M.
2
4
6
8
10
12
14
16
18
20

Fig. A-19 PVD Installation in Field

Fig. A-20 PVD Installation in Field

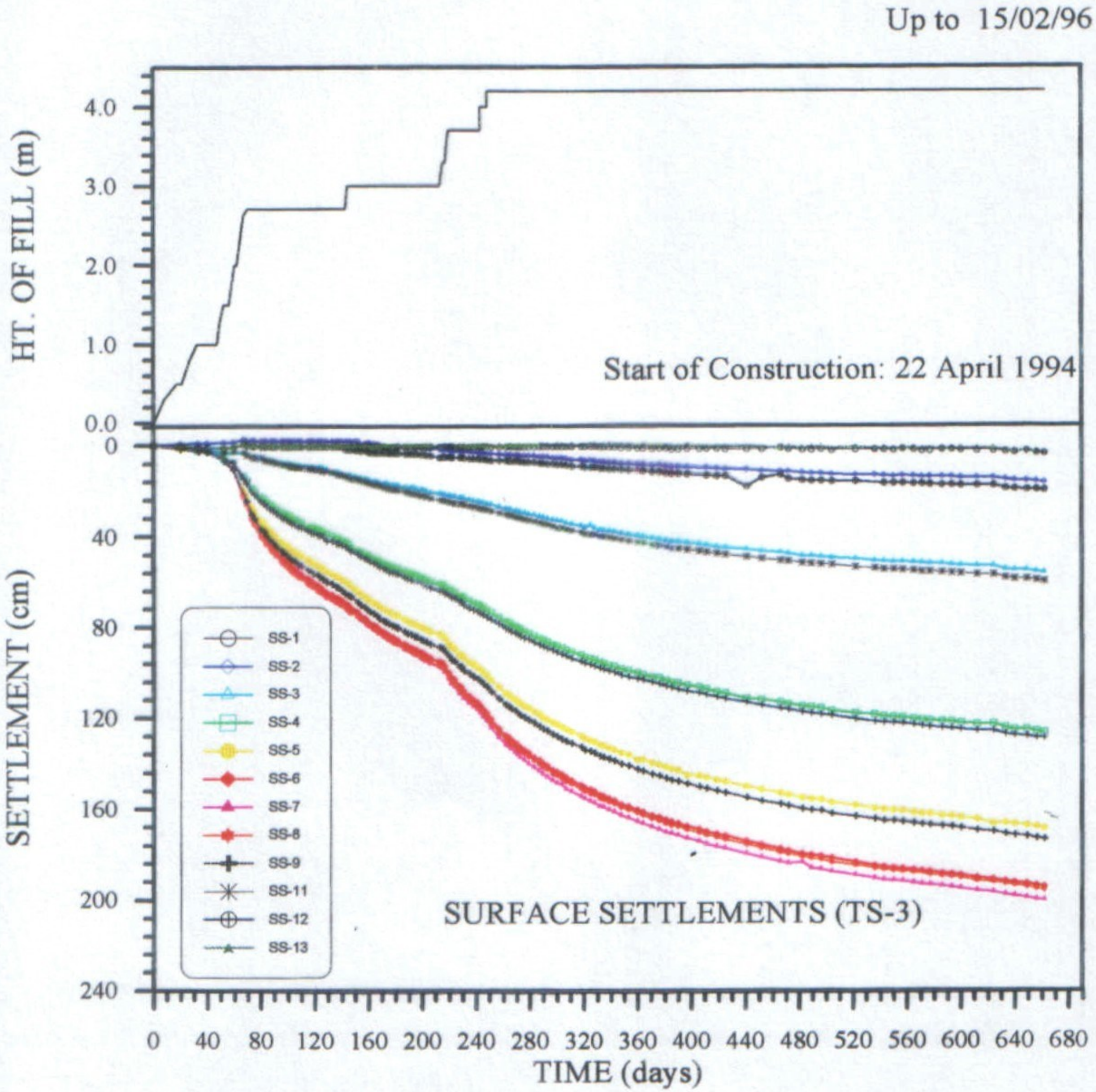
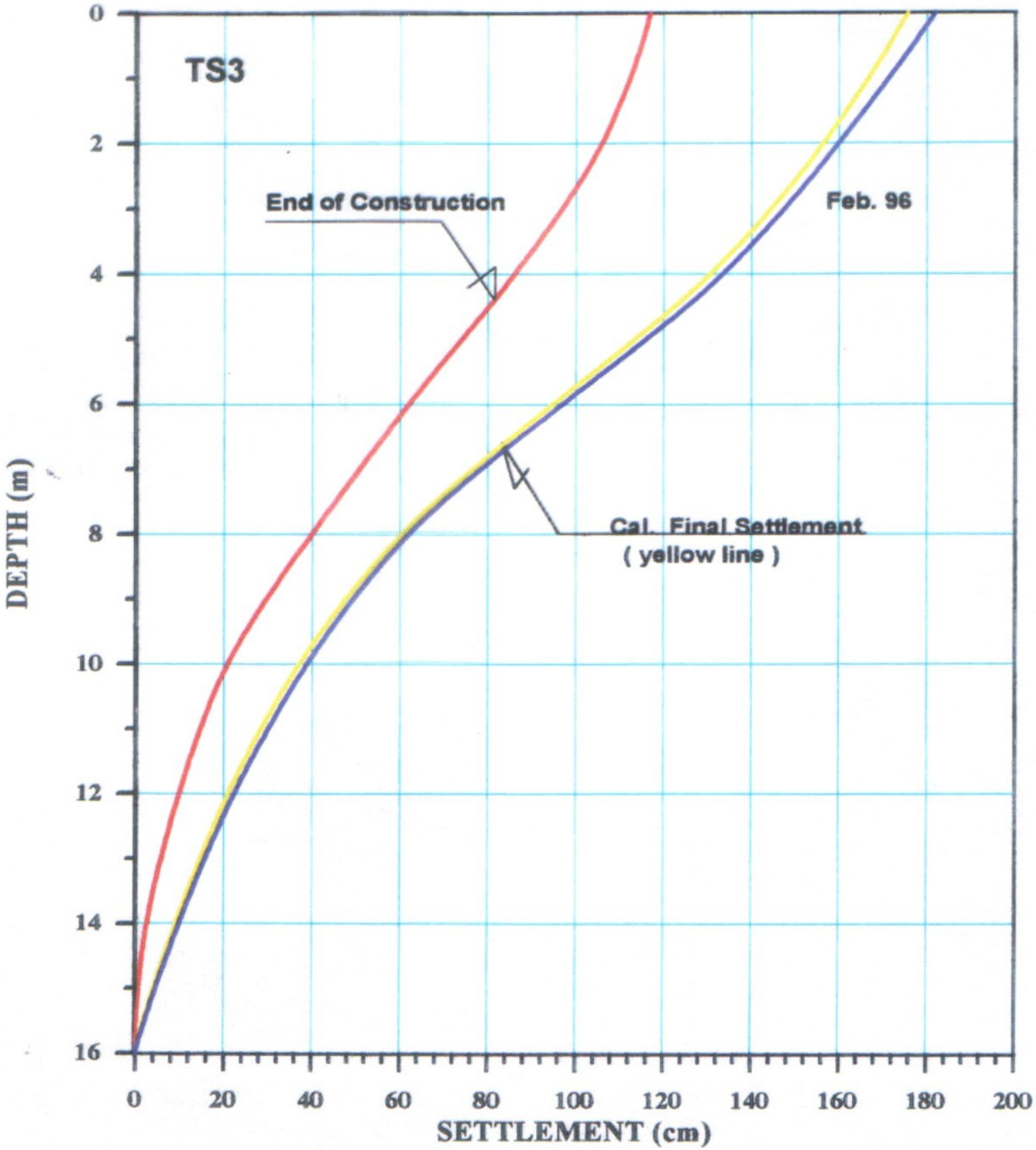


Fig. 3 Time-Settlement Plot (TS3) with Loading Schedule (Surface Settlement Gauge Measurements)



SETTLEMENT PROFILES OF TS3

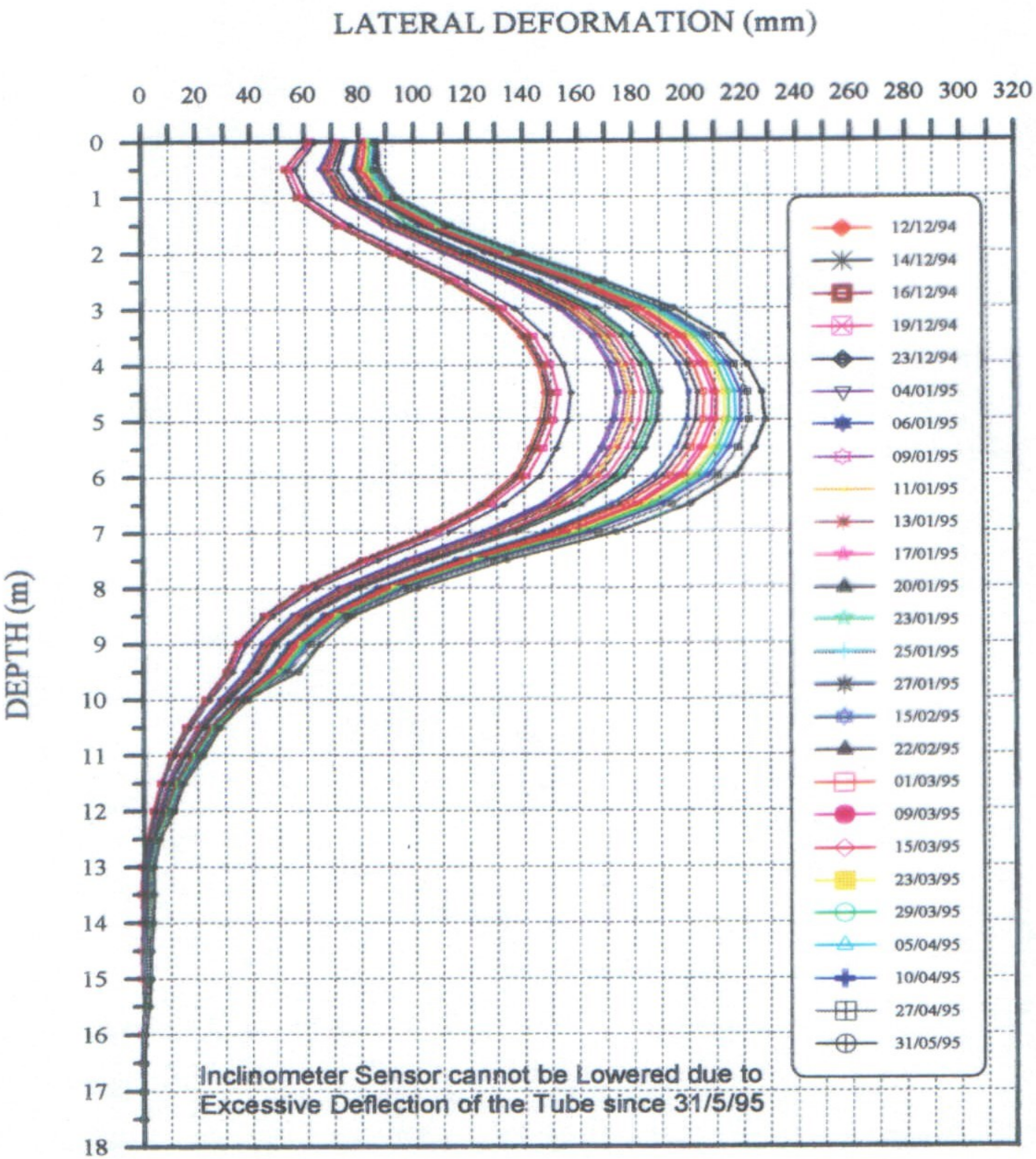


Fig. 12 Lateral Deformations with Depth (TS3 - I2)

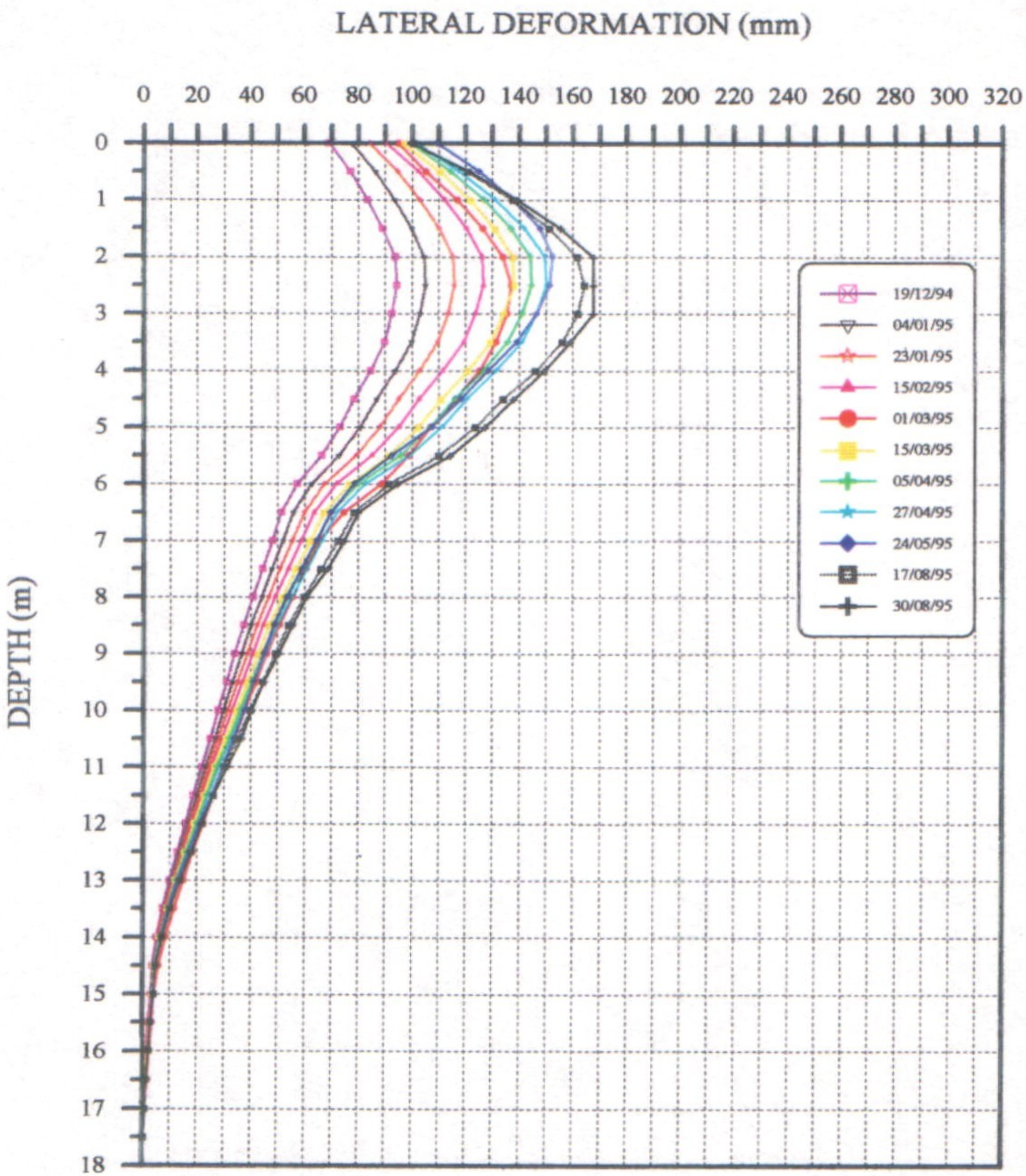


Fig. 9 Lateral Deformations with Depth (TS3 - I1)

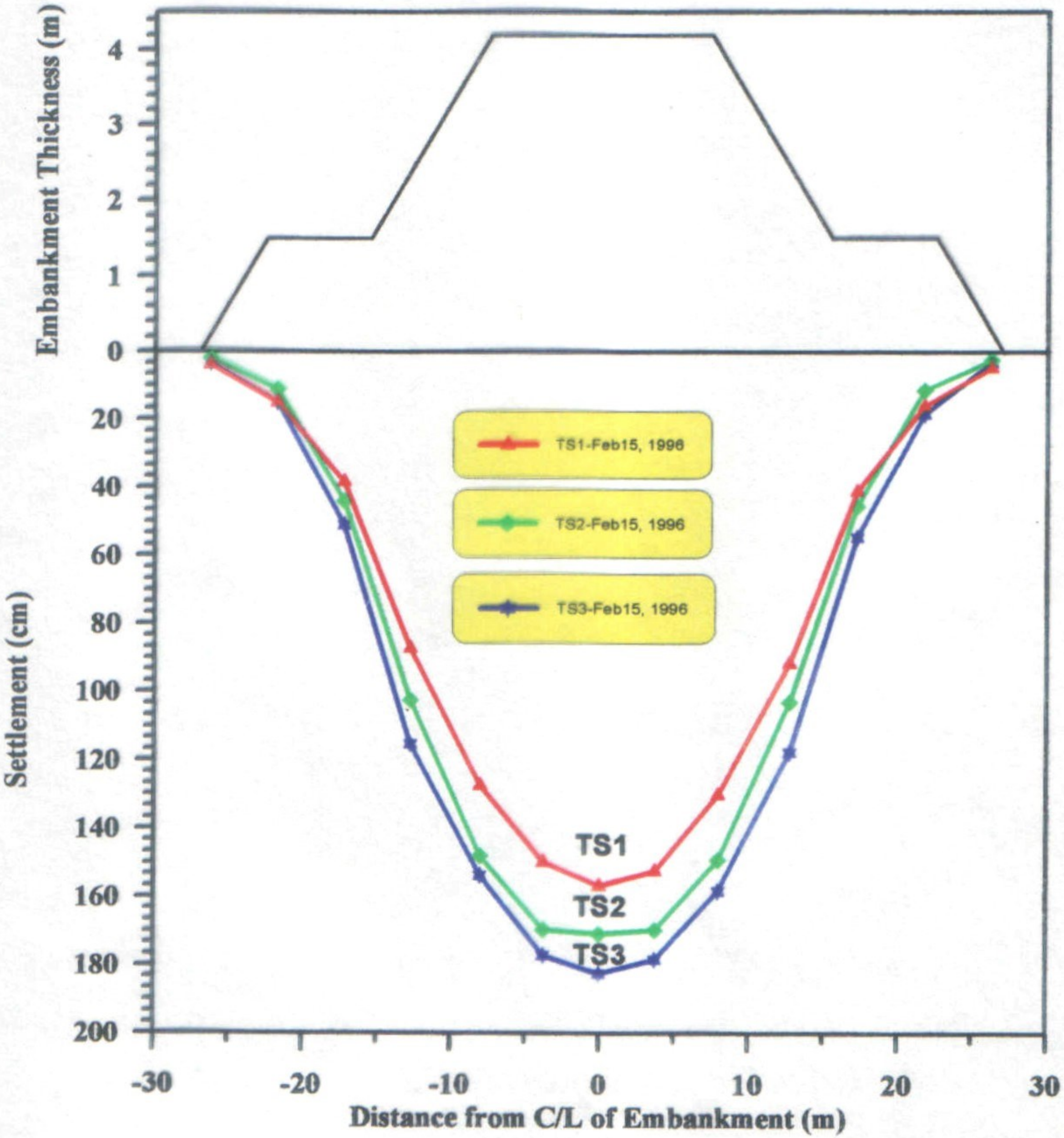
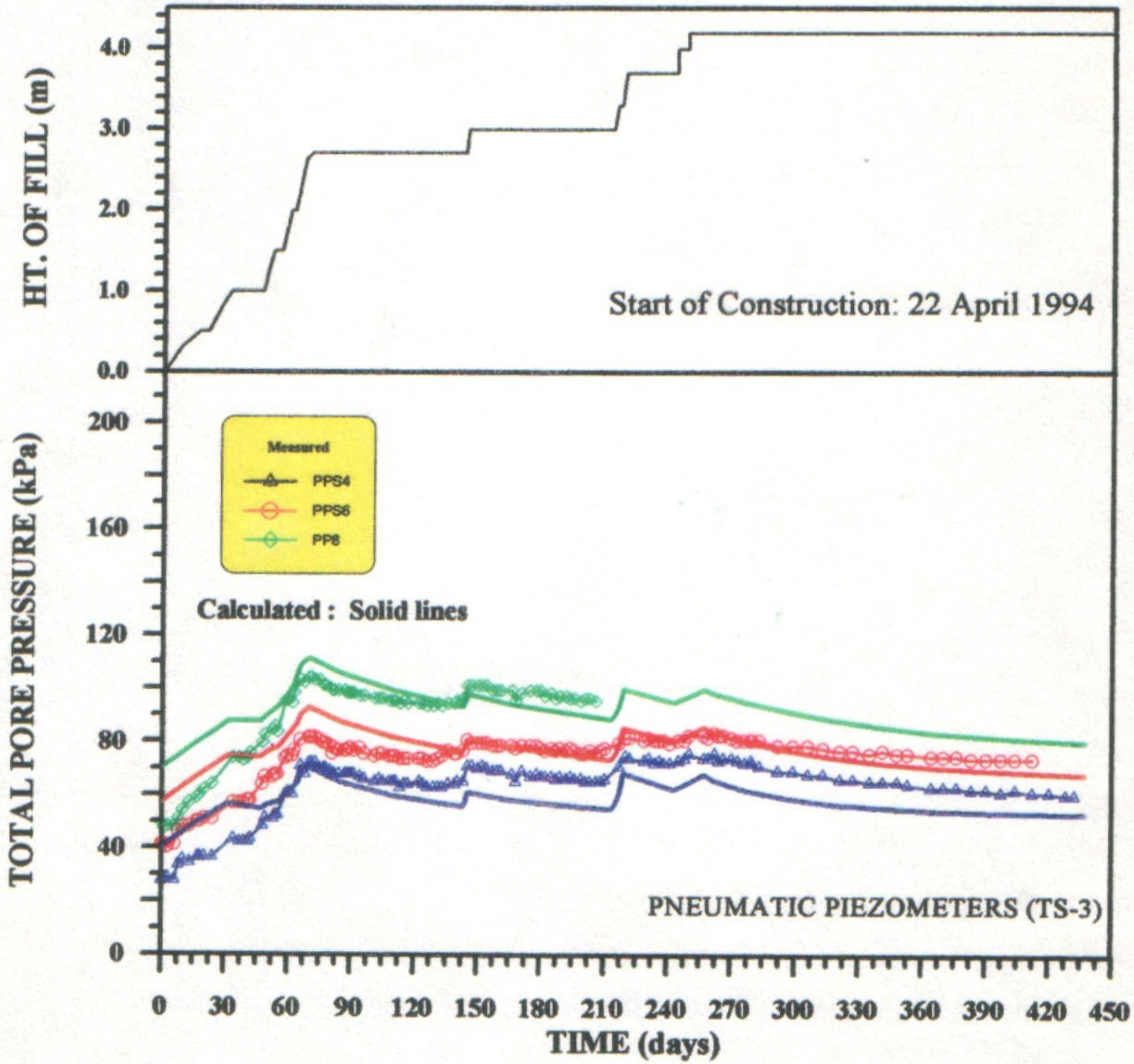


Fig. 6.7 Surface Settlement Profile across the Embankments Cross Sections (TS1, TS2 and TS3)



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

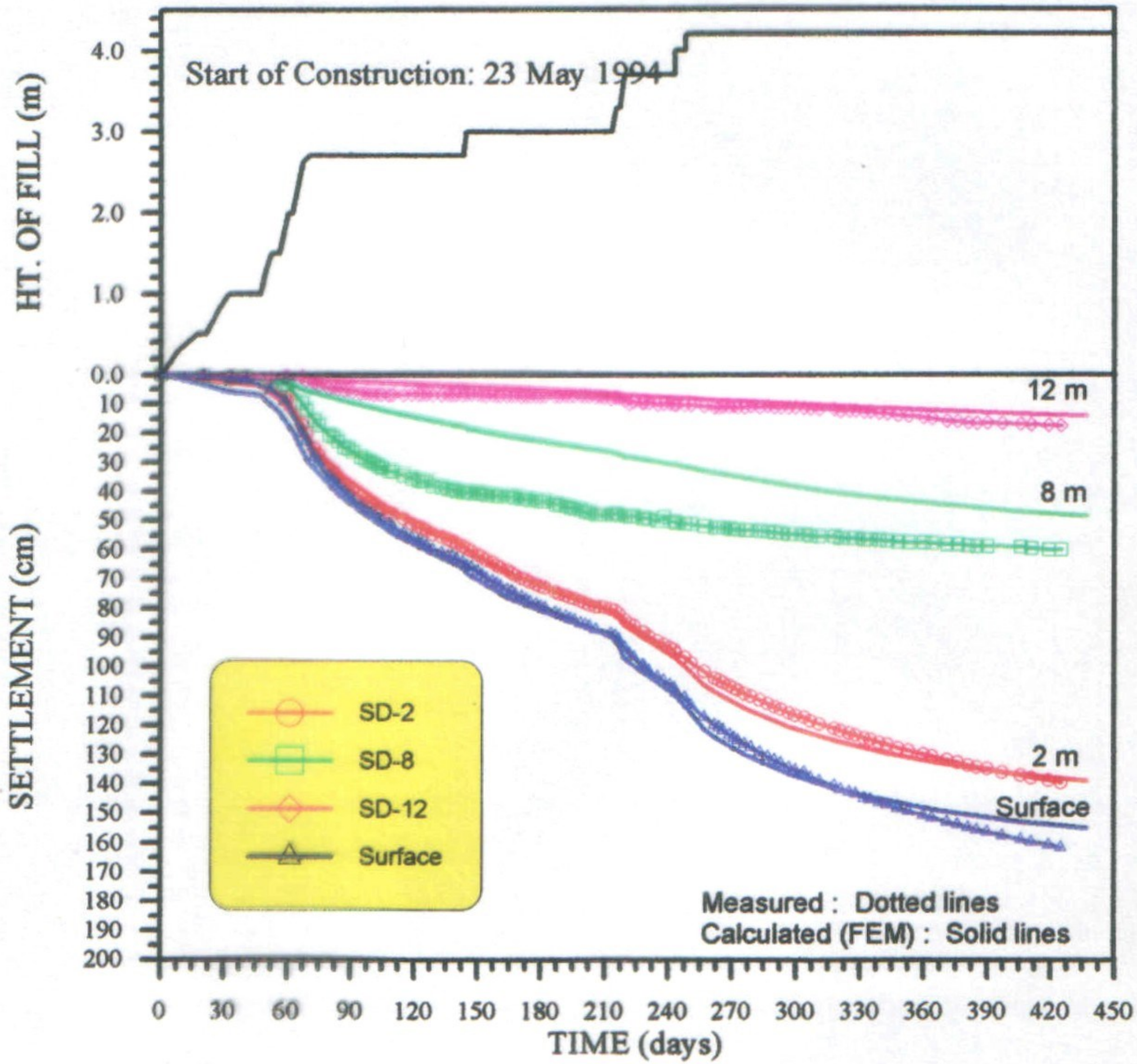


Fig. 6.42 Comparison of Calculated (by FEM) and Measured Settlements at 0 m, 2m, 8m, and 12m Depth for TS3 Test Embankment

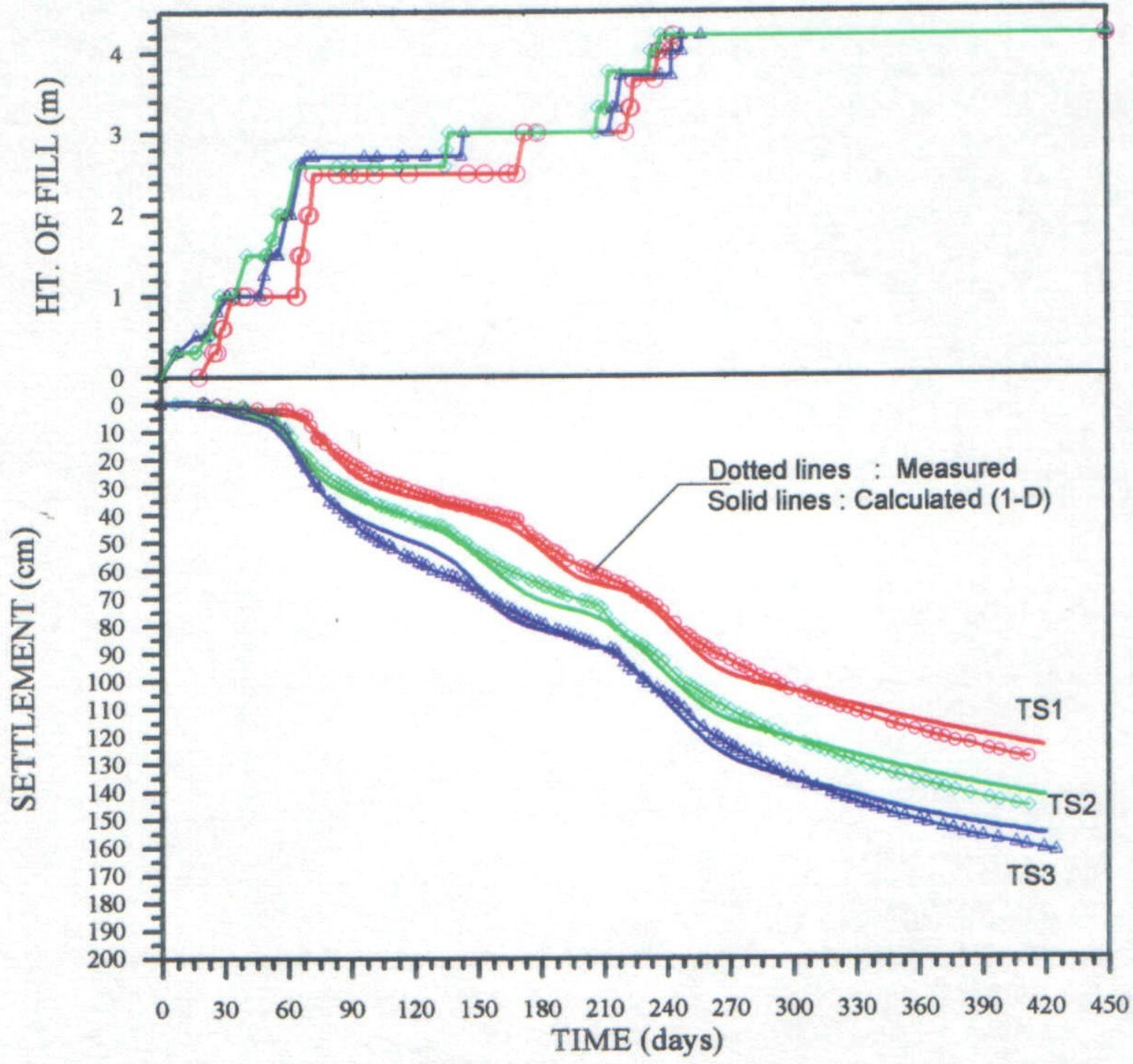
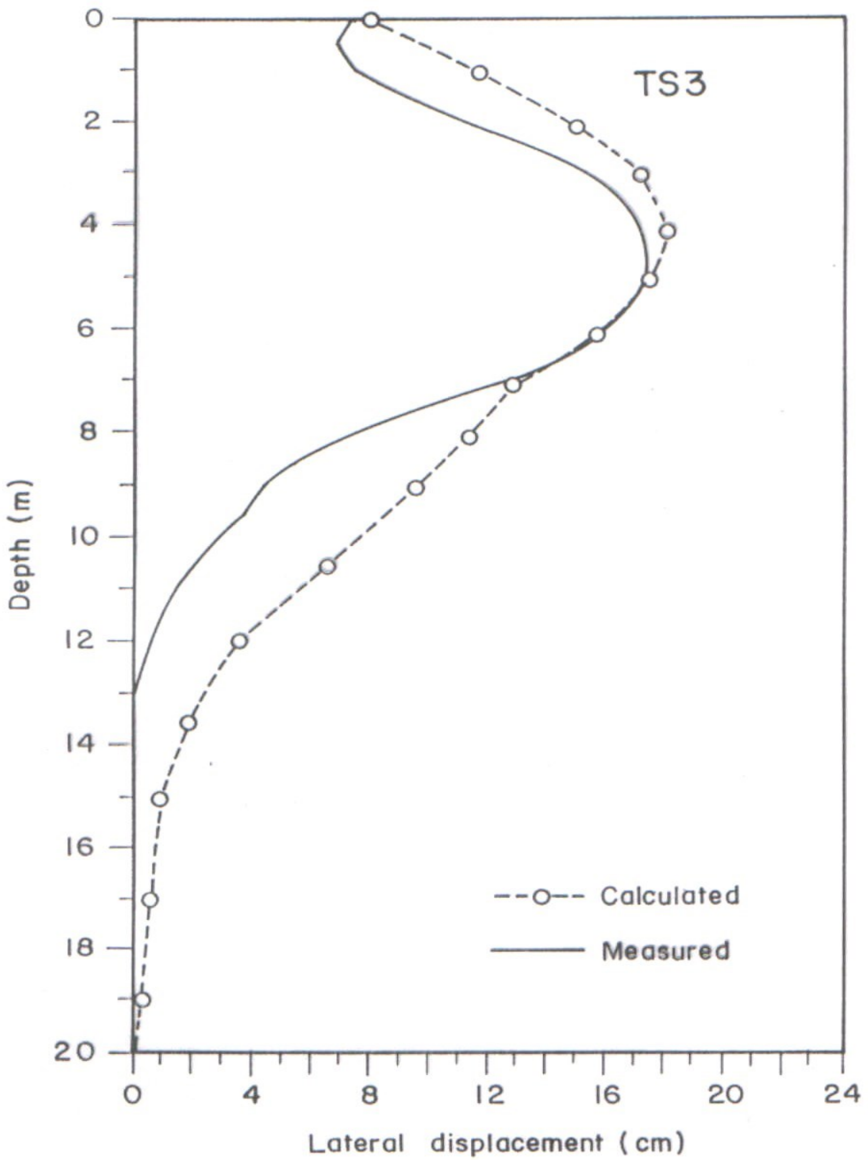


Fig. 6.35 Comparison of Measured and 1-D, Calculated Settlements for Three Test Embankments TS-1, TS-2, and TS-3



Comparison of Computed (FEM) and Measured Lateral Deformations at the end of Construction for Embankment TS3

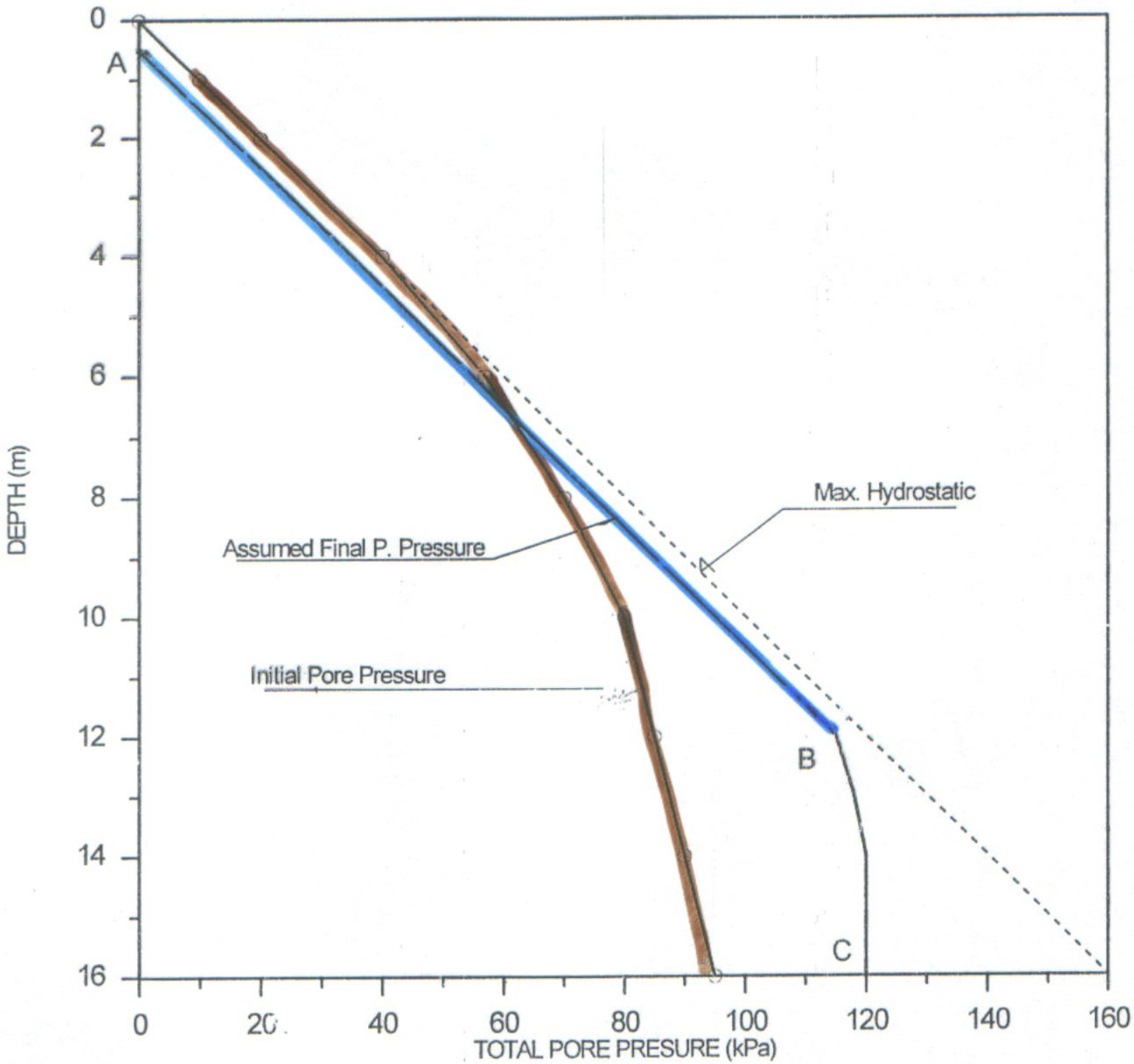
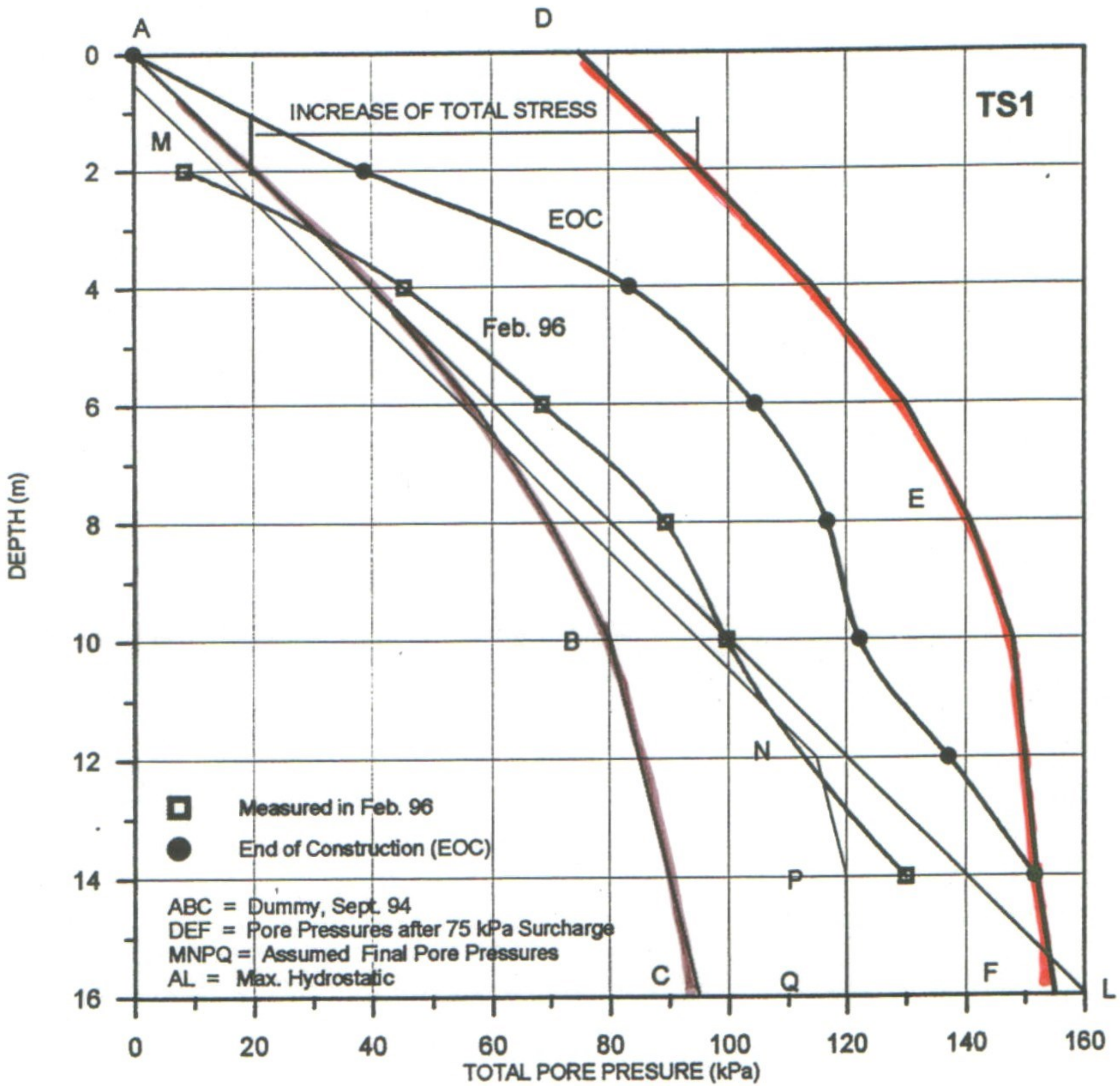
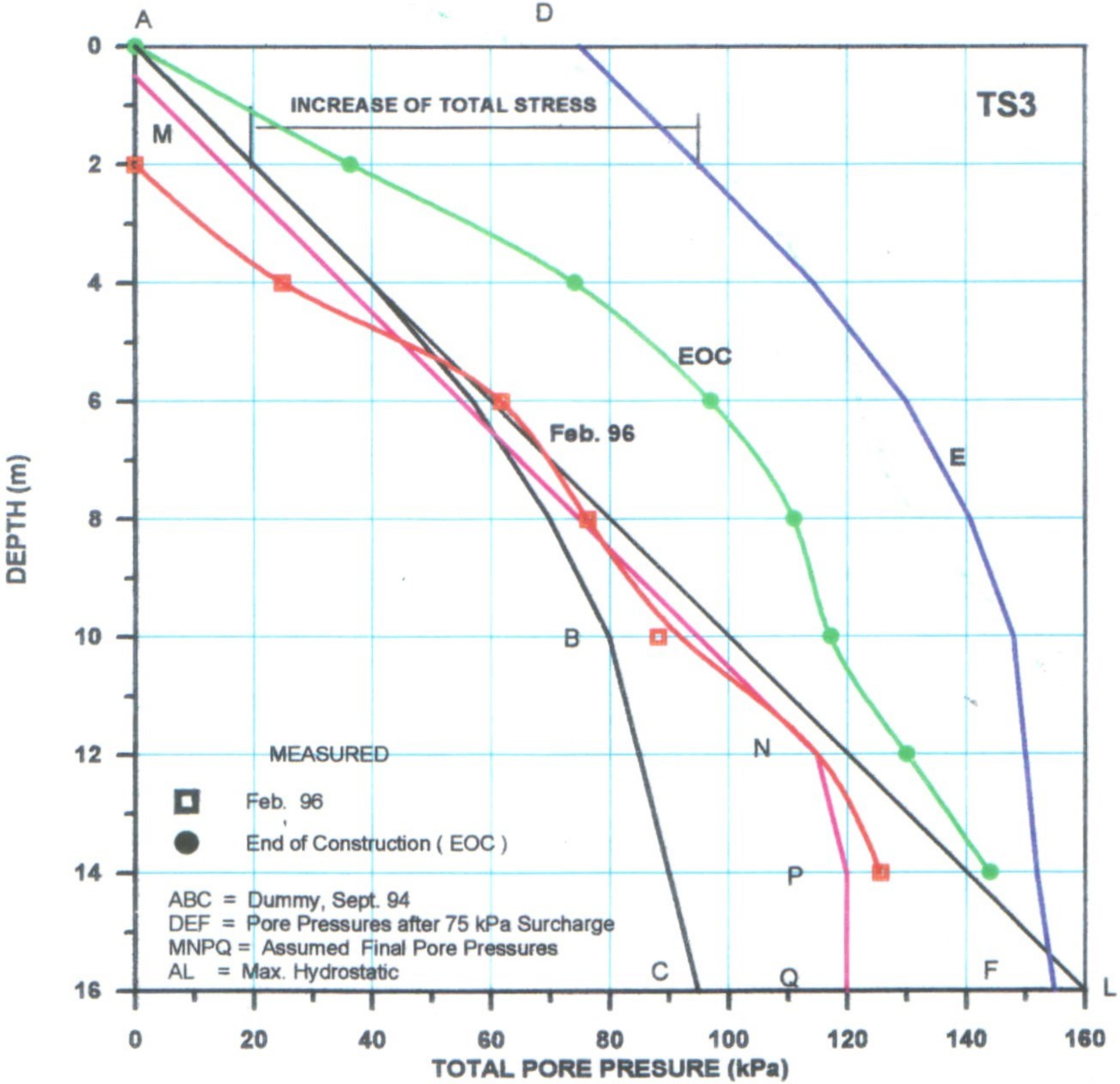


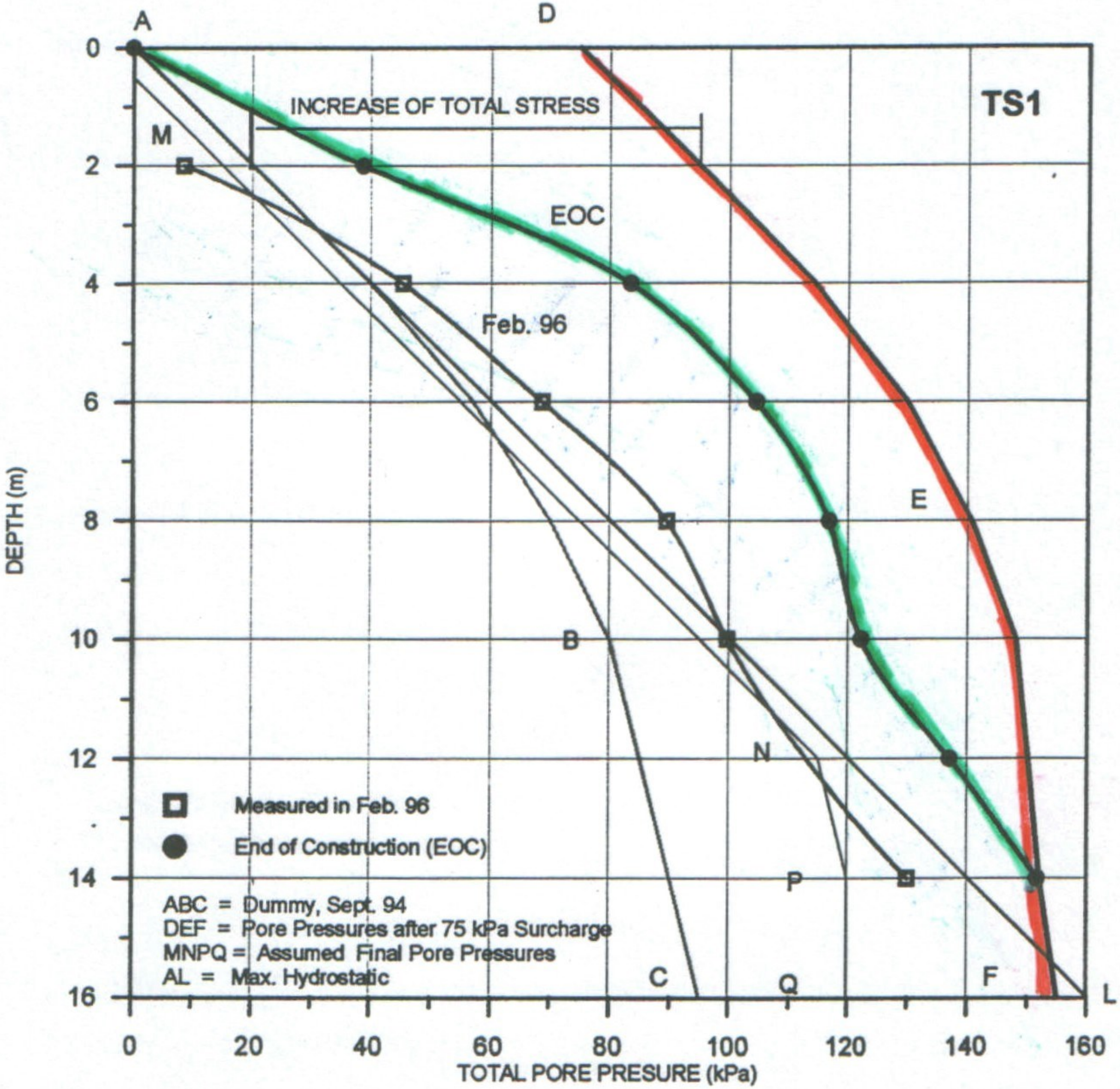
Fig. 6.26: Piezometric Drawdowns (Initial and Assumed Final Values)



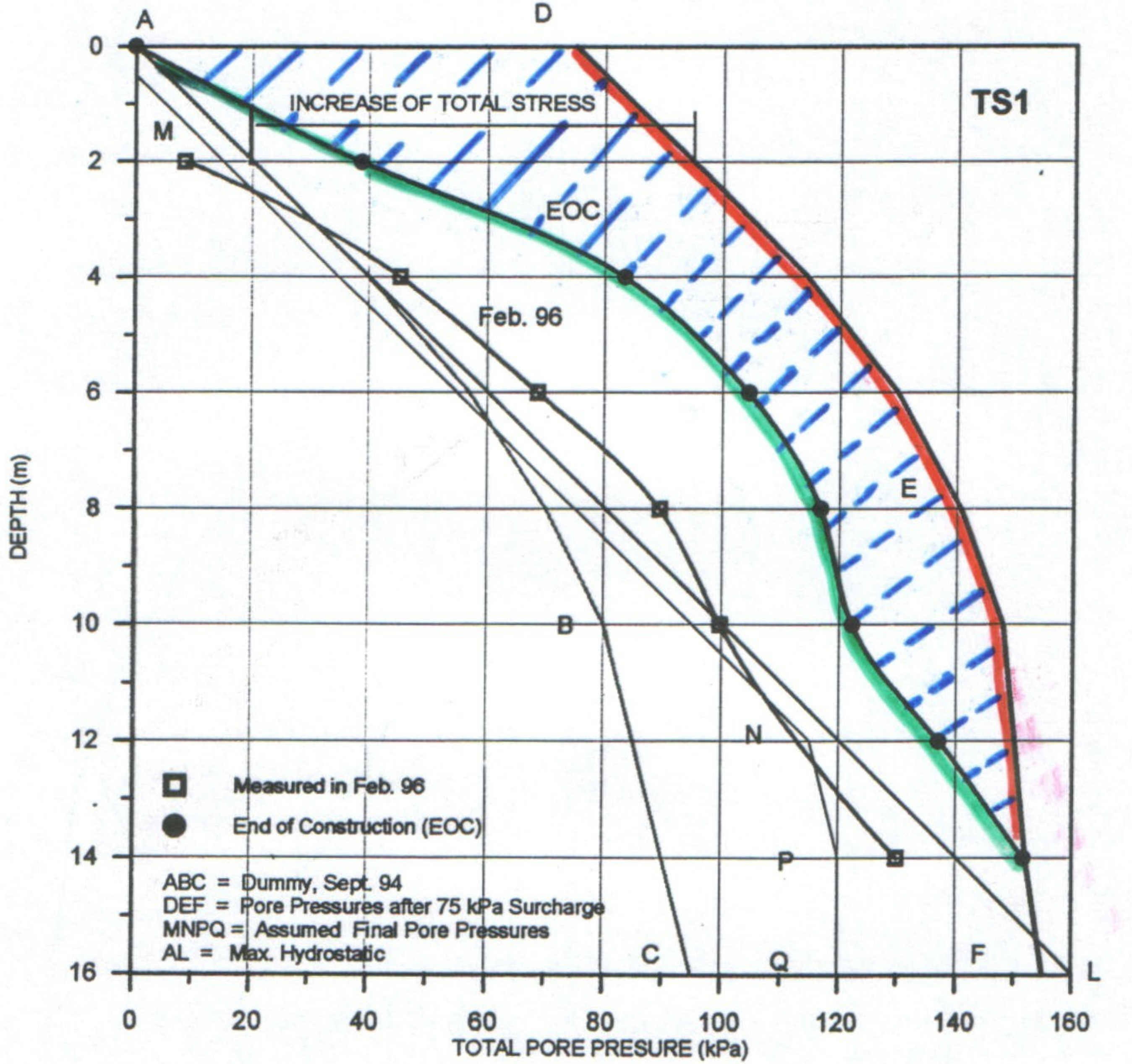
PORE PRESSURE PROFILE IN TS1



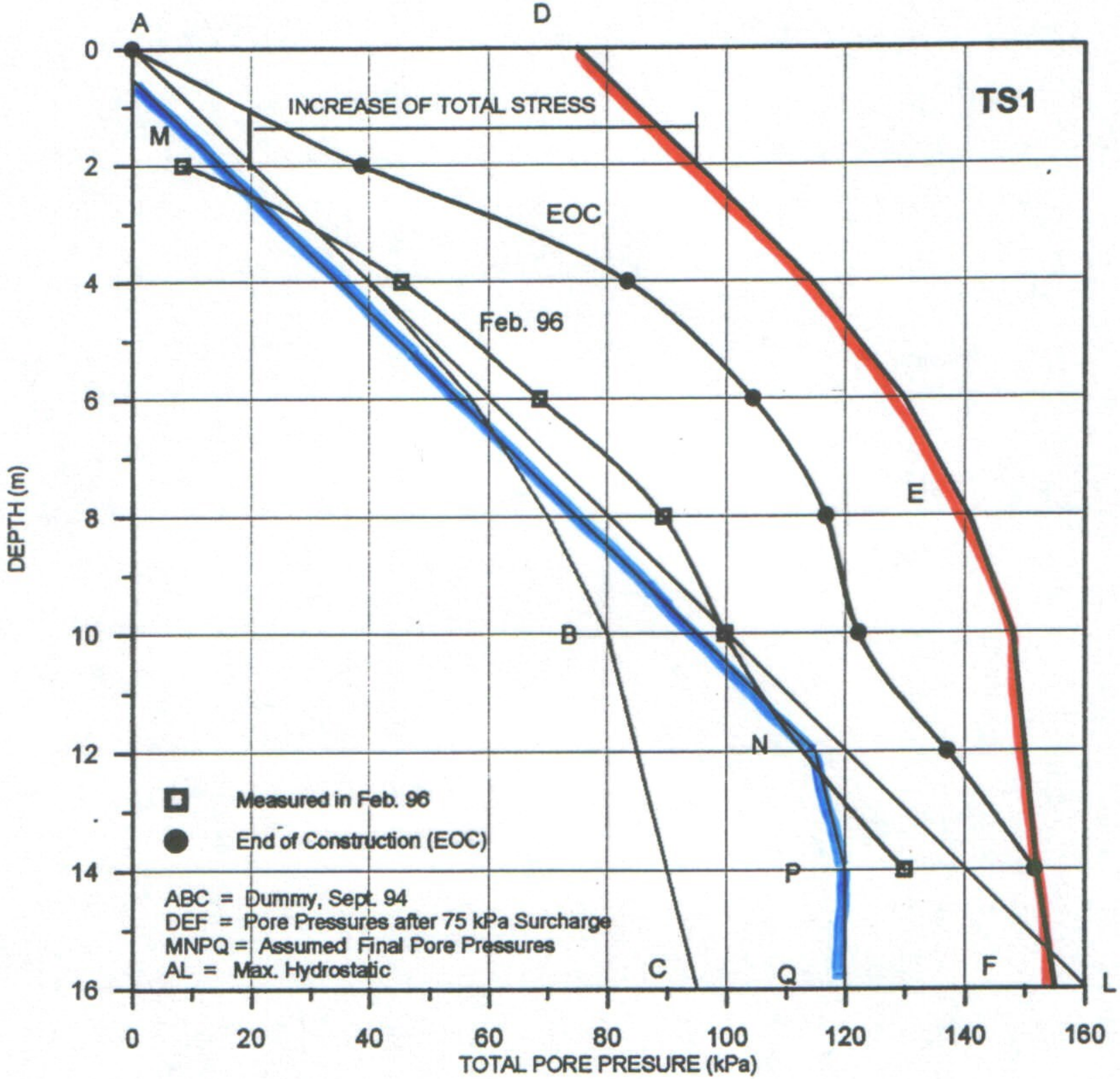
PORE PRESSURE PROFILES IN TS3



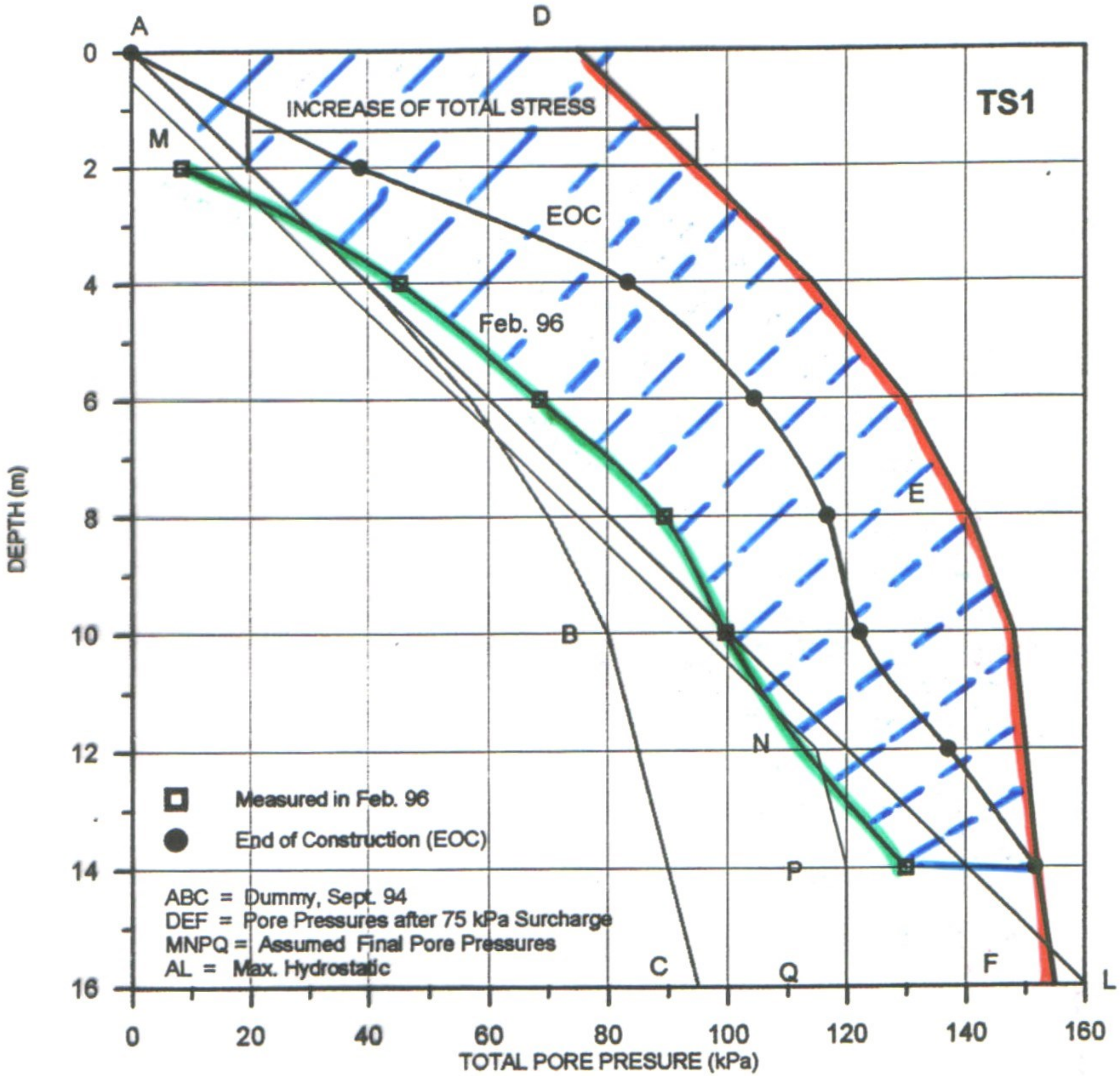
PORE PRESSURE PROFILE IN TS1



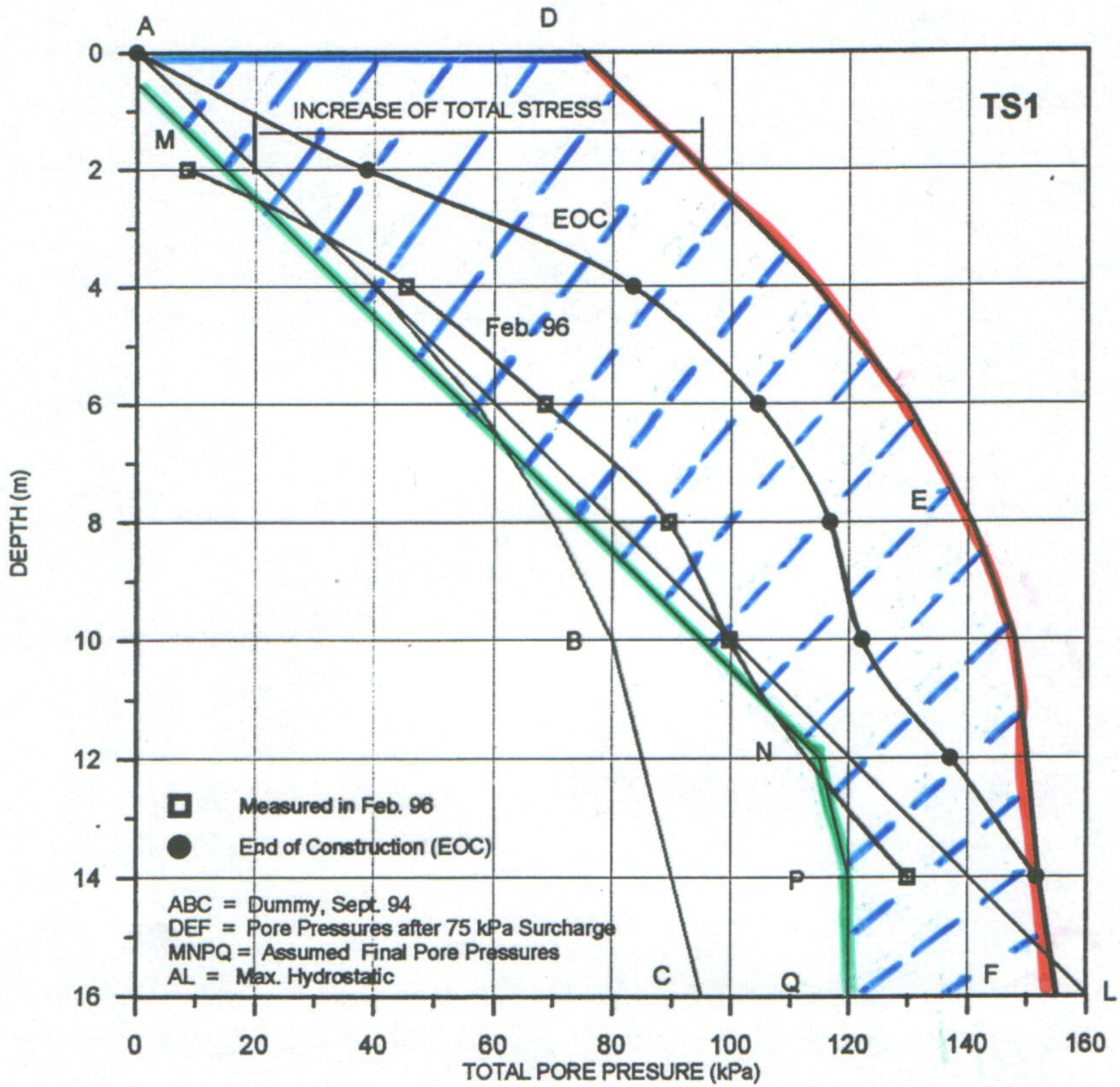
PORE PRESSURE PROFILE IN TS1



PORE PRESSURE PROFILE IN TS1



PORE PRESSURE PROFILE IN TS1



PORE PRESSURE PROFILE IN TS1

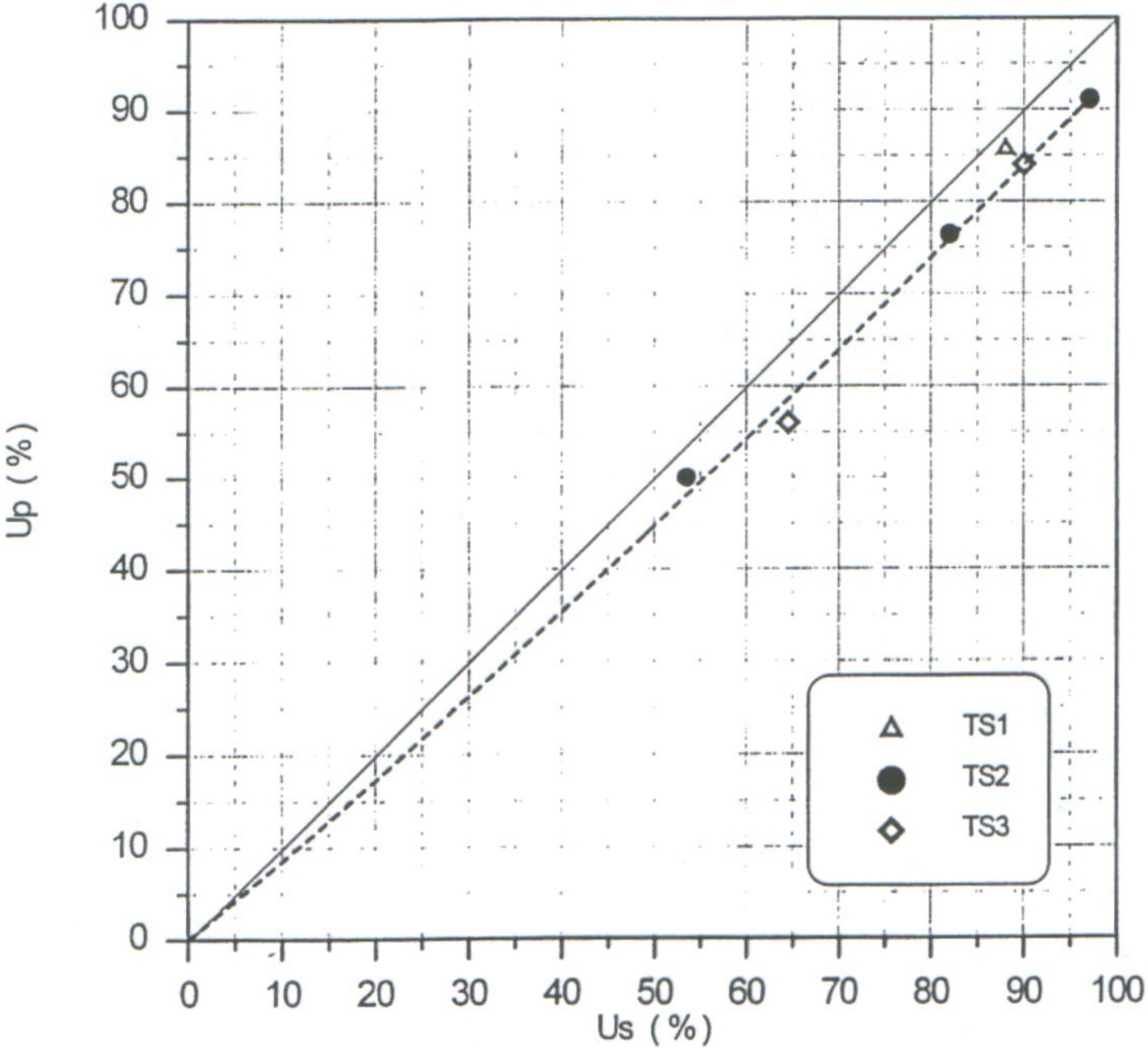
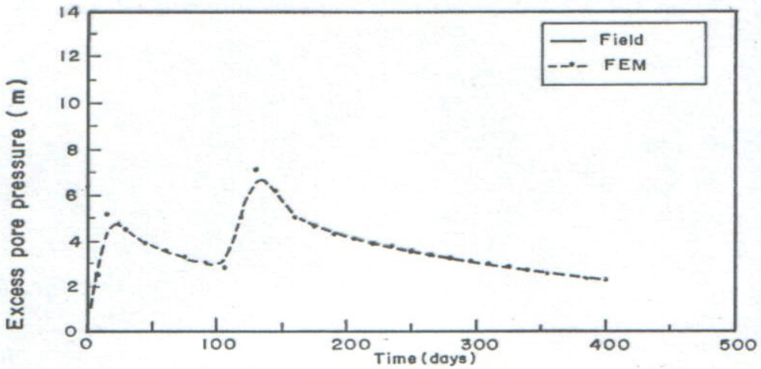
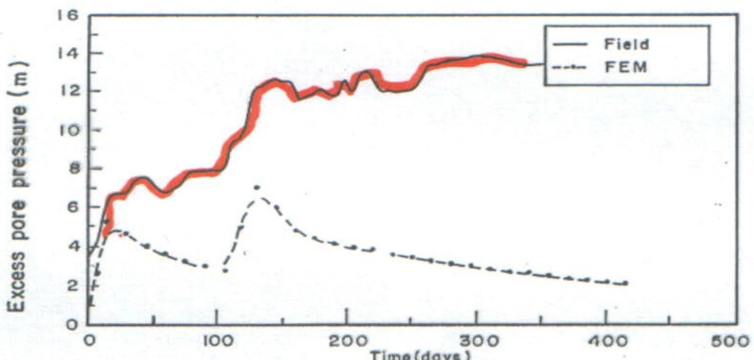


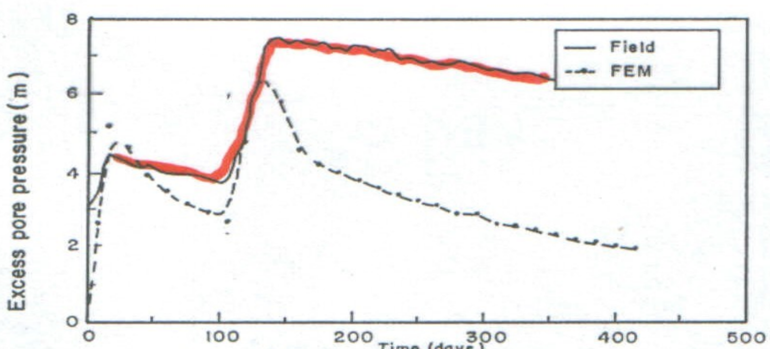
Fig. 14 Relation of Degree of Consolidation from Settlement (Us and Up)



(a) at 4.6 m depth (P4)

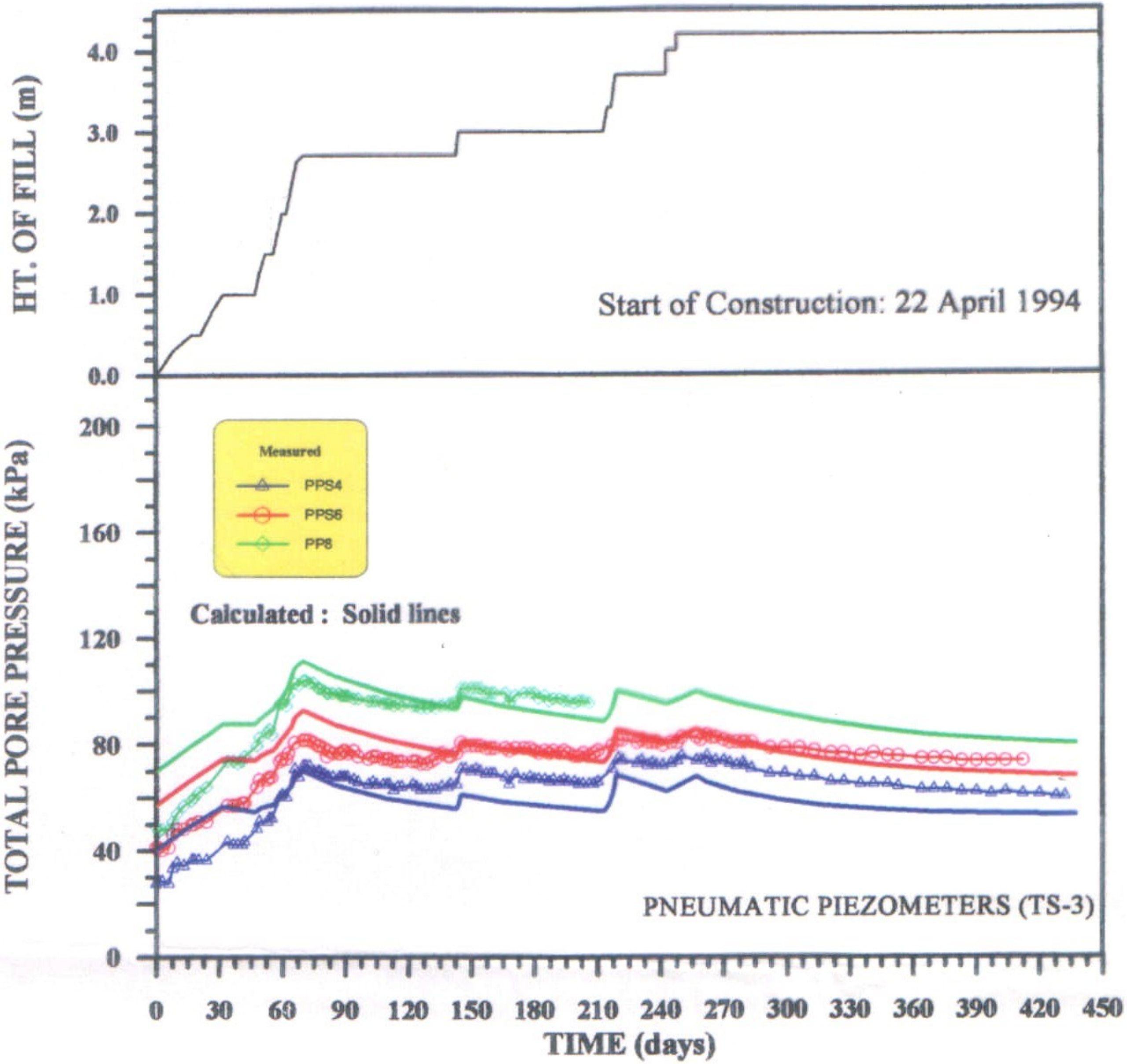


(b) at 9 m depth (P5)



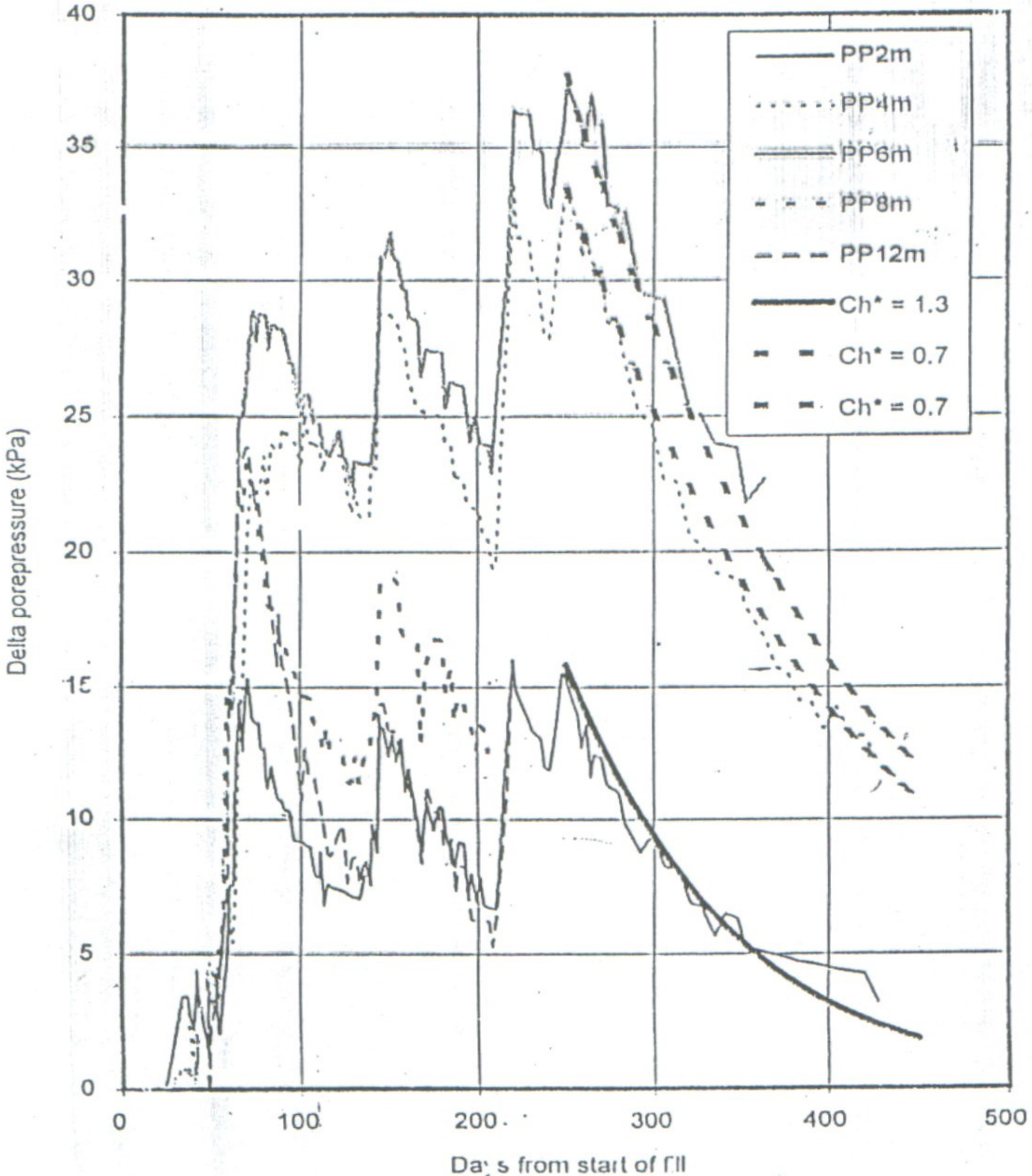
(c) at 13.6 m depth (P6)

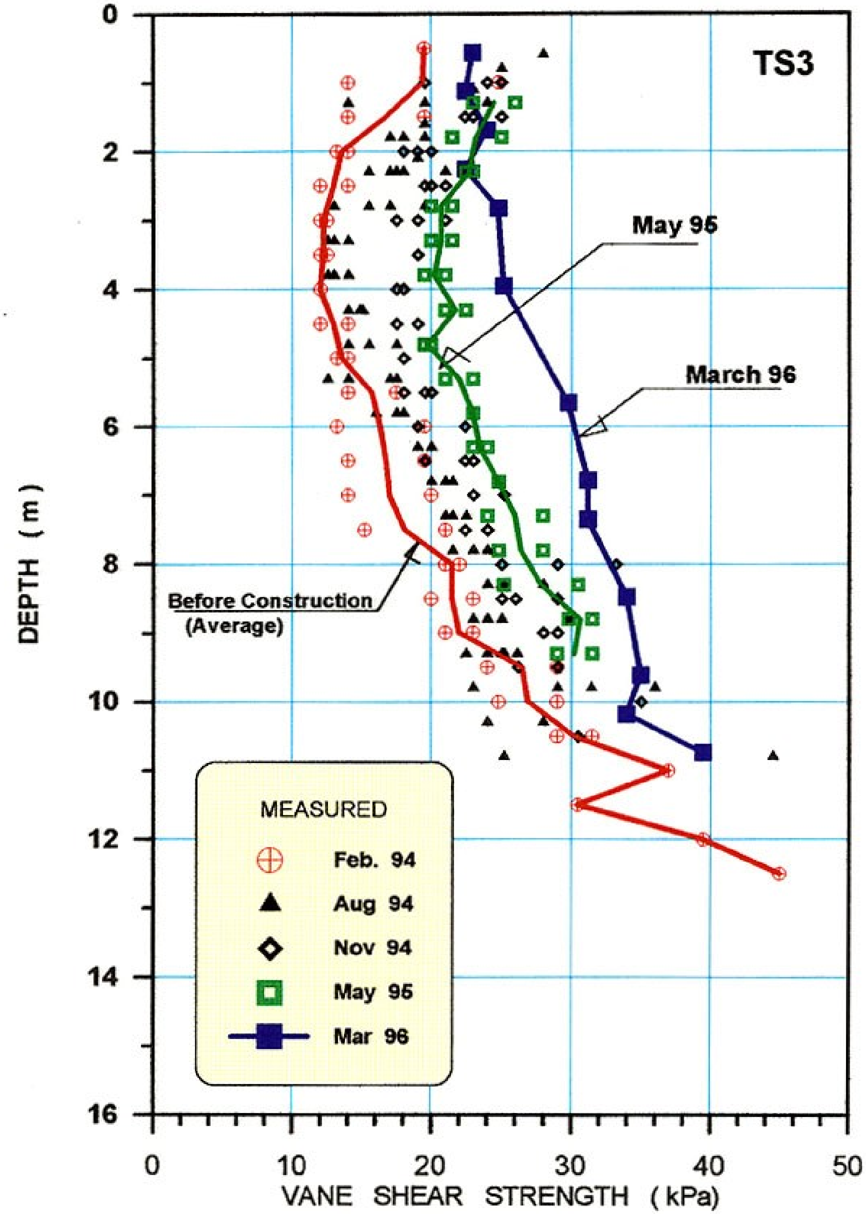
Center Line Excess Pore Pressure – Time Relationships of the Embankment with Vertical Drains



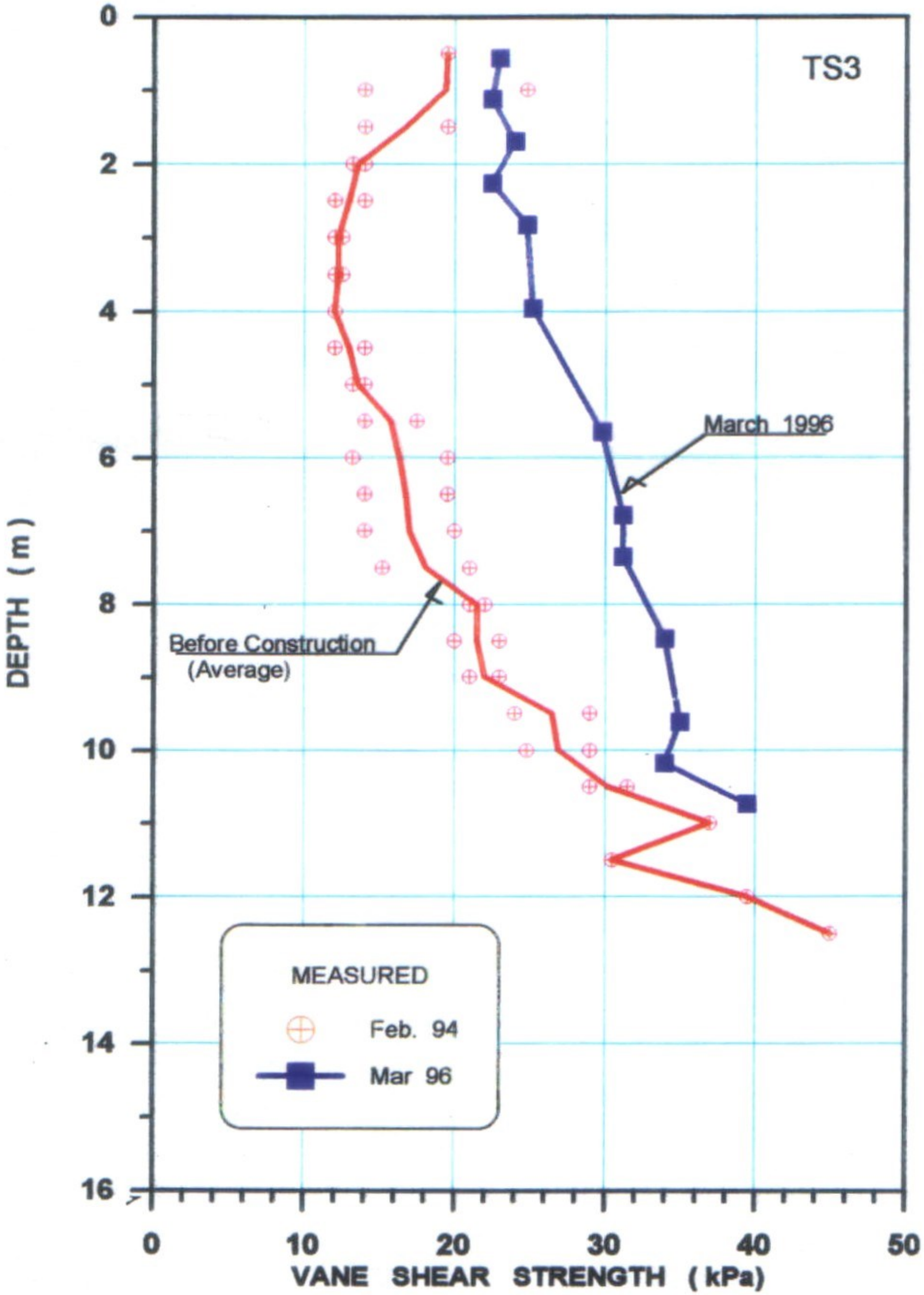
Comparison of FEM Calculated and Measured Total Pore Pressures at Different Depths 4 m, 6 m, and 8 m for TS3 Embankment

Figure 3.1 Back calculation of pore pressure dissipation TS3





Field Vane Shear Strength Measured in Embankment TS3



Field Vane Shear Strength Measured in Embankment TS3

Concluding remarks

This presentation diagnosed the various phases of site investigation at the Bangkok new international airport site over a period of twenty- five years.

The presentation diagnosed the various phases of site investigation at the new Bangkok international airport over a period of twenty-five years

The first phase of investigation should have been designed to obtain design parameters and to enable suitable construction techniques with a view to

1. **Have flood protection with a bund-canal scheme**
2. **Accommodate ground subsidence effects with time**
3. **Use driven piles which are common foundation elements at that time**

The second phase of investigation explores the use of vacuum drainage and large diameter sand drains

1. **Fissures and silt seams in the clay layer made it difficult to maintain the vacuum and the method was not implemented in the final stage. If successful the vacuum method could have eliminated the cost involved in the use of expensive sand as surcharge material**

2. **Sand drains also caused excessive disturbance during the installation and the settlement was non-uniform with large lateral movements.**

The third phase concluded with the following methods to be adopted on the airfield side with the order of priority

1. Pre-consolidation with vertical drains
2. Deep soil improvement
3. Piles supporting a free spanning cocrete slab
4. Relief piles with caps
5. Light weight fill material

The confidence gained from the experience at the Changi site in Singapore, Muar site in Malaysia, the Hong Kong airport site and several other sites in Japan and other countries influenced the selection of pre-consolidation with vertical drains as the suitable ground improvement technique

Pre-consolidation with PVD as the vertical drains was studied in the fourth phase and was used successfully in the construction phase

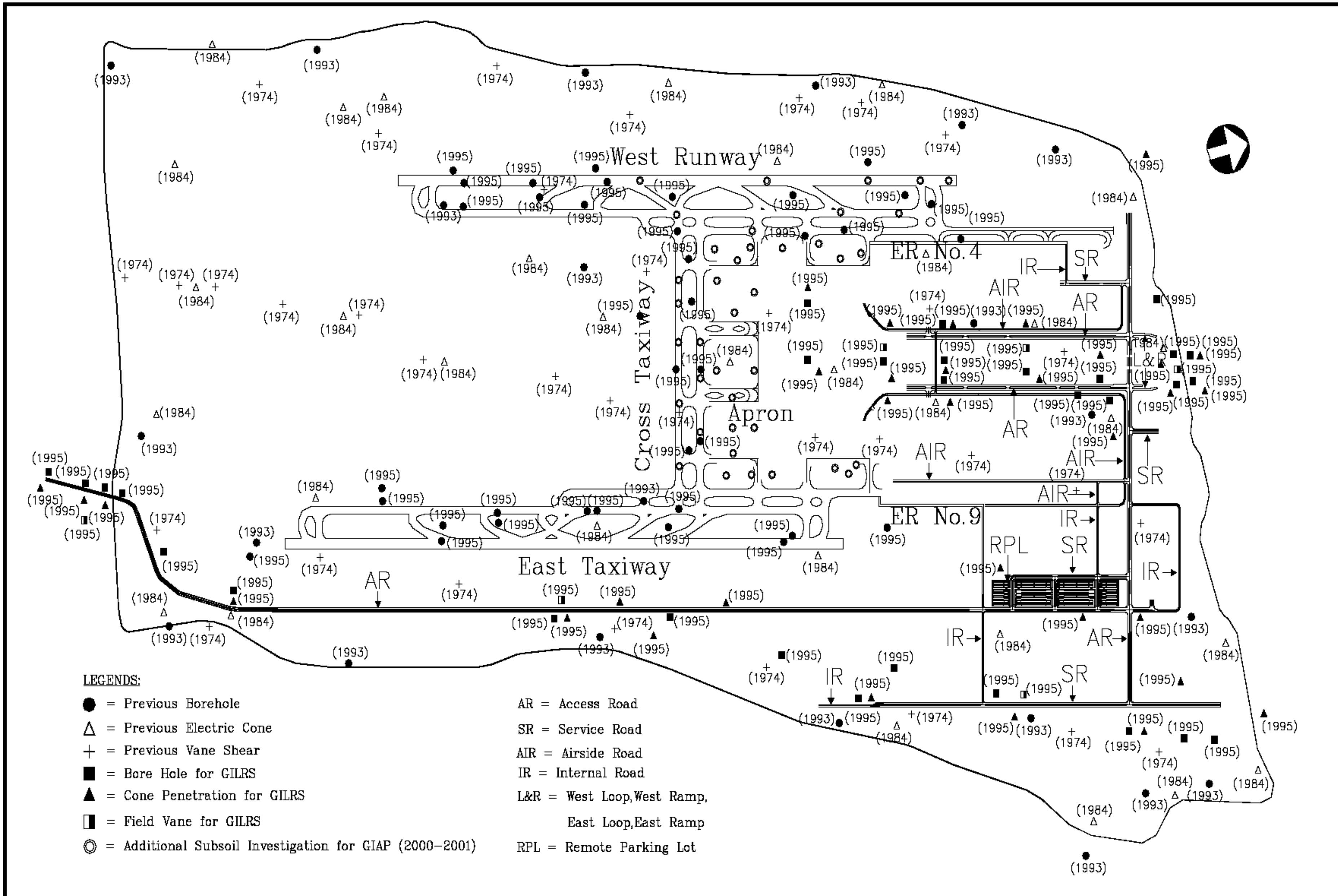
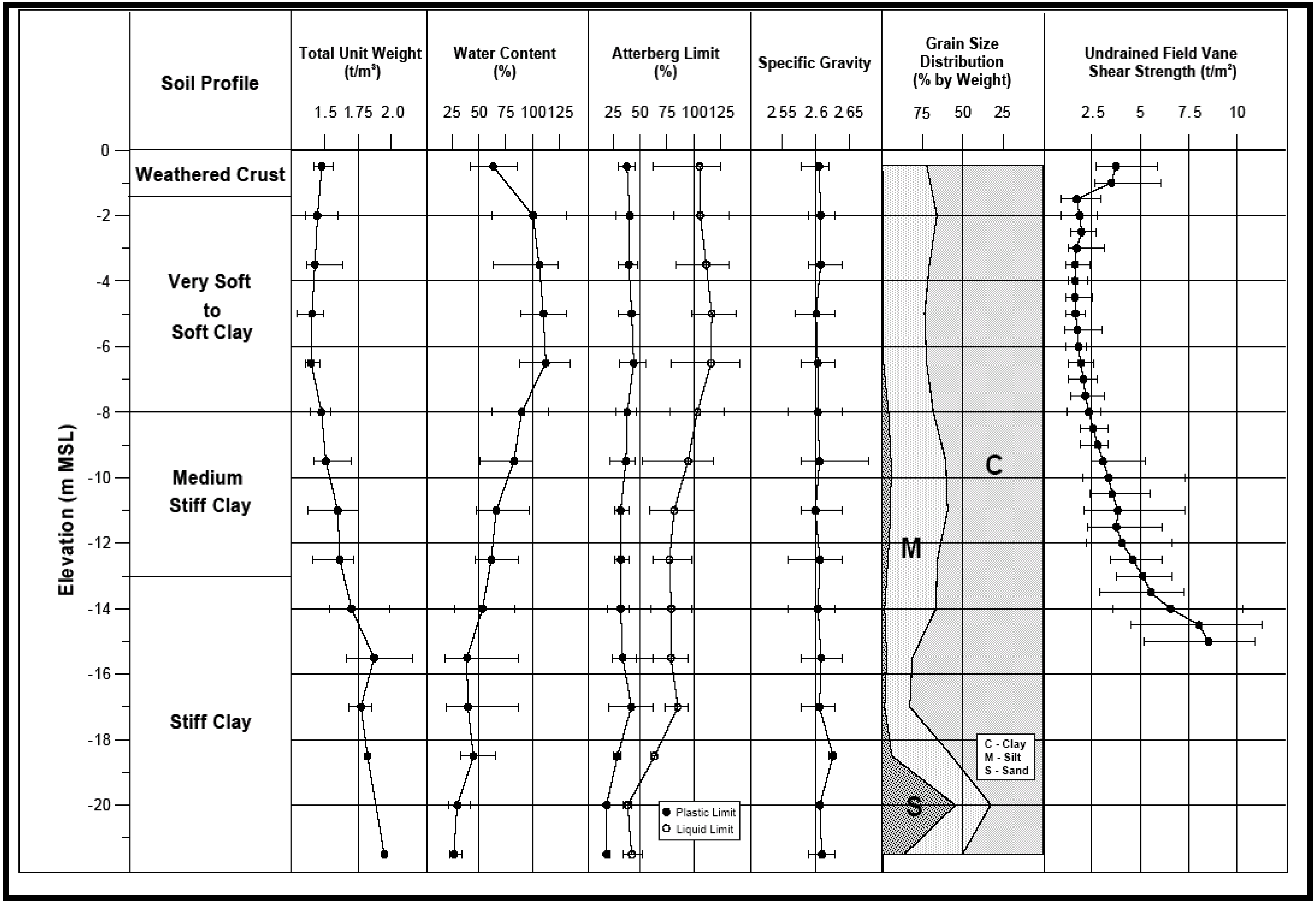


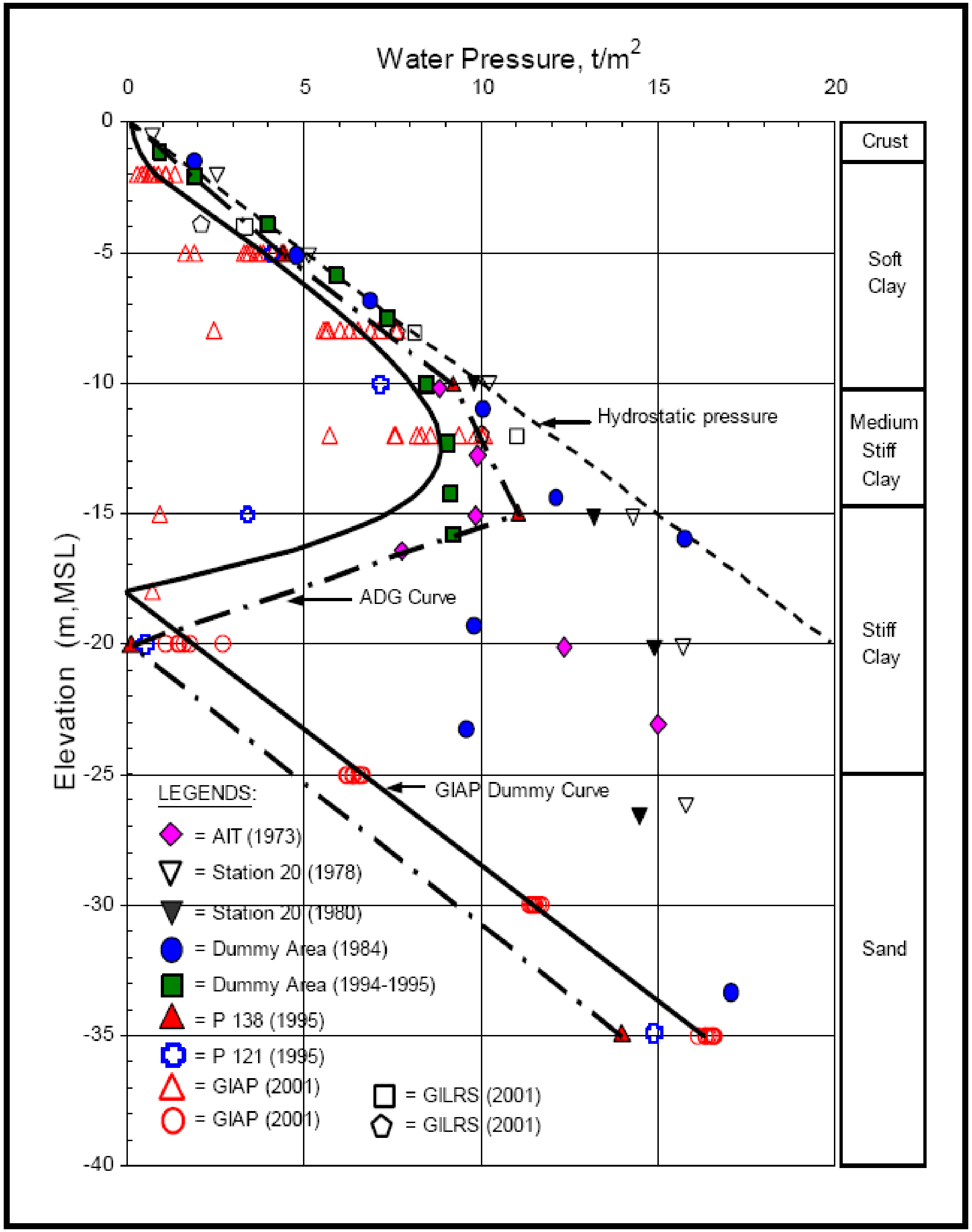
Figure 2 Locations of Soil Investigation at SIA Site

Project Data

	GIAP	GILRS
Total Construction Cost (Thai Baht)	8,419,205,000	1,767,488,000
Financial Source	Government Budget	OECF Loan (Japan)
Construction Period	01/11/97~30/04/02 (54 mos.)	01/12/00~19/04/03 (29 mos.)
Design	Airside Design Group	Moh and Associates
Construction Supervision	TEC/MAA/SIGEC/UIC/MTL	TEC/MAA/NK
G.I. Area (sq. m)	3,080,000	1,320,000
PVD (m)	33,580,000	10,889,600
Sand Blanket (cu. m)	4,550,000	899,600
Preloading Material (cu. m)	2,890,000 (Crushed Rock)	1,722,800 (Sand)



General Soil Properties at SIA Site



Dummy Pore Water Pressures

Comparison of Ground Improvement Design Between GIAP and GILRS

Item \ Project	GIAP	GILRS		
		Type I	Type II	Type III
Design Criteria	A min. 80% of the primary consolidation should be reached.	Rate of consolidation settlement of the subsoil should be less than 0.5 cm/mo. before pavement construction.		
Sand Blanket	150cm	50cm	80cm	130cm
PVD	10m deep with 1.0m spacing in square pattern	10m deep with1.0m spacing in triangular pattern		
Filter Fabric	Below and above sand blanket	None		
Preloading Material	Crushed Rock	Sand		
Stage Loading	Two**	One	Two**	Three**
Embankment Thickness	3.8m & 4.2m	2.2m	3.5m	4.5m
Berm	15m wide & 1.7m high with 1:4 side slope	No berm with 1:3 side slope		
Design Load	75 kPa & 85 kPa	41.8 kPa*	66.5 kPa*	85.5 kPa*
Removing Criteria	Min. 6 (or 11) months waiting period, min. 80% consolidation & 2%~4% settlement ratio	Min. 6 months waiting period with max. 3 cm monthly settlement rate		

*: Embankment unit weight is assumed to be 19 KN/m^3.

**: Three months waiting period is required before next stage loading construction.

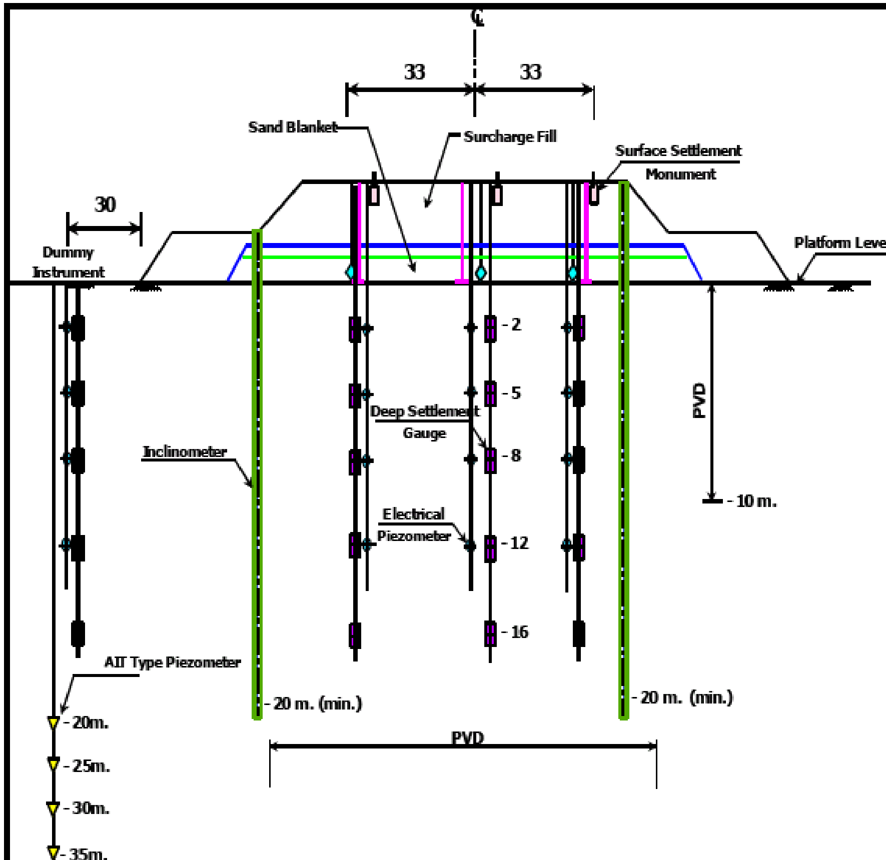
Total Quantity of Instrumentation

Item \ Project	GIAP		GILRS	
	Embankment	Dummy	Embankment	Dummy
Surface Settlement Plate	1,724	-	730	4
Surface Settlement Monument	553	-	-	-
Permanent Benchmark	-	2	-	-
Inclinometer	56	-	88	-
Deep Settlement Gauge	111*	11*	53**	-
Pneumatic Piezometer	-	-	159	6
Electric Piezometer	444	46	-	-
AIT-type Piezometer	-	40	-	-
Observation Well	1,722	-	1236***	-

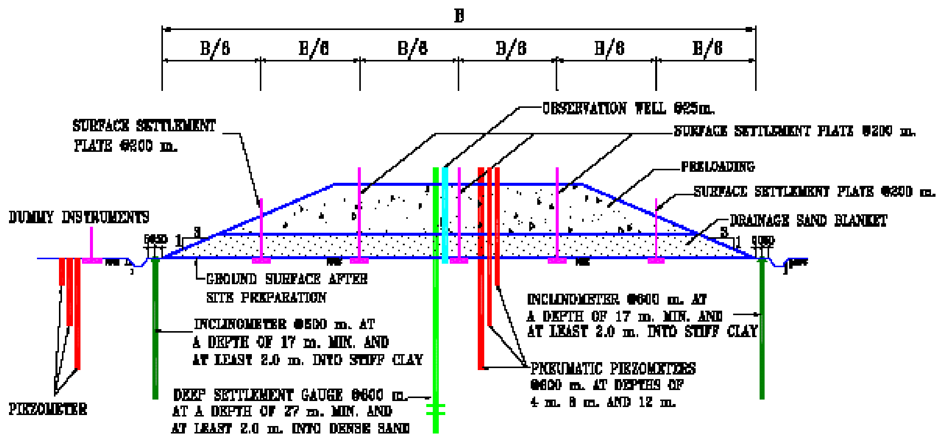
*: One set includes 5 individual deep settlement gauges at 2m, 5m, 8m, 12m & 16m.

**: A deep settlement gauge includes sensor rings at every 2m to min. depth of 27m.

***: Also used as pumping wells

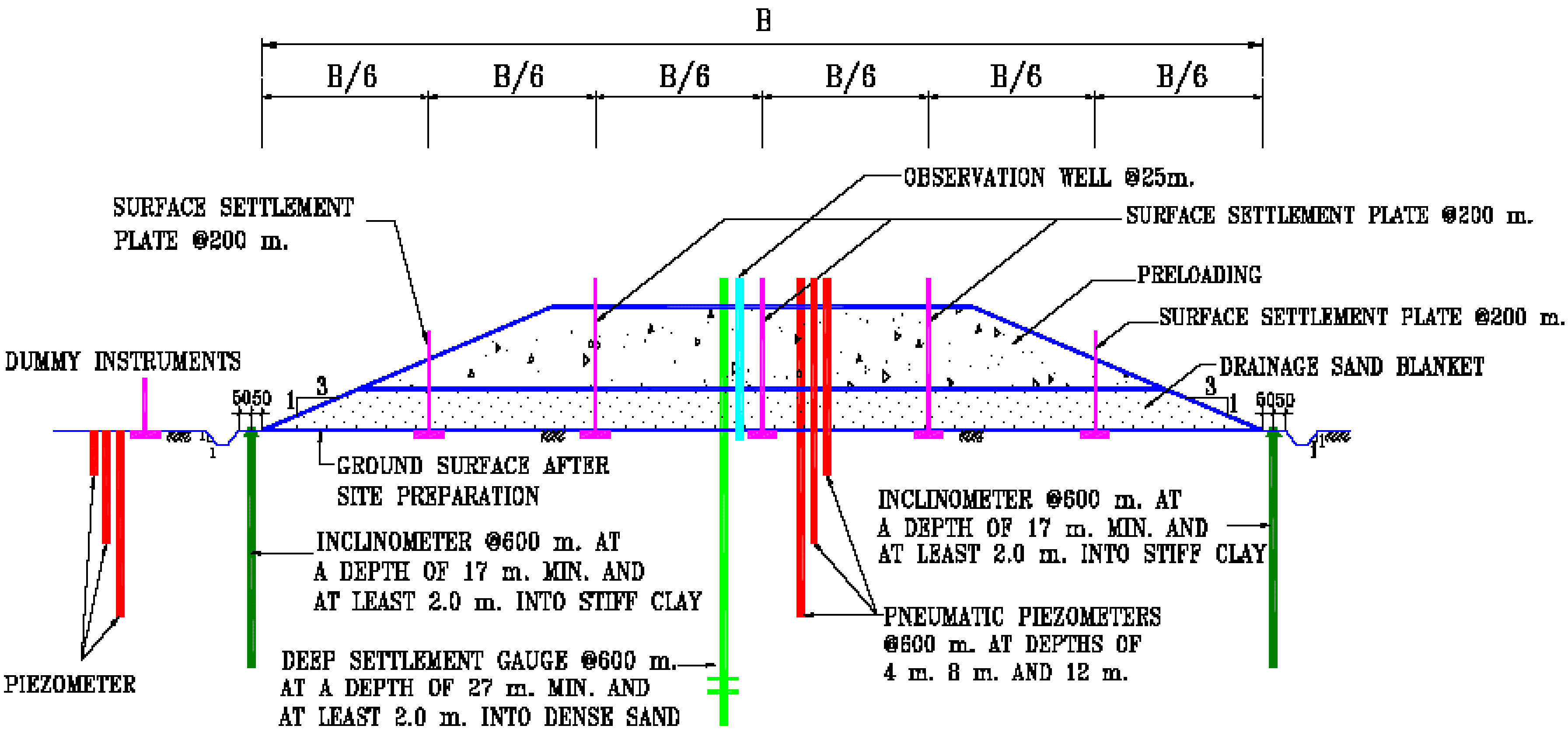


GIAP

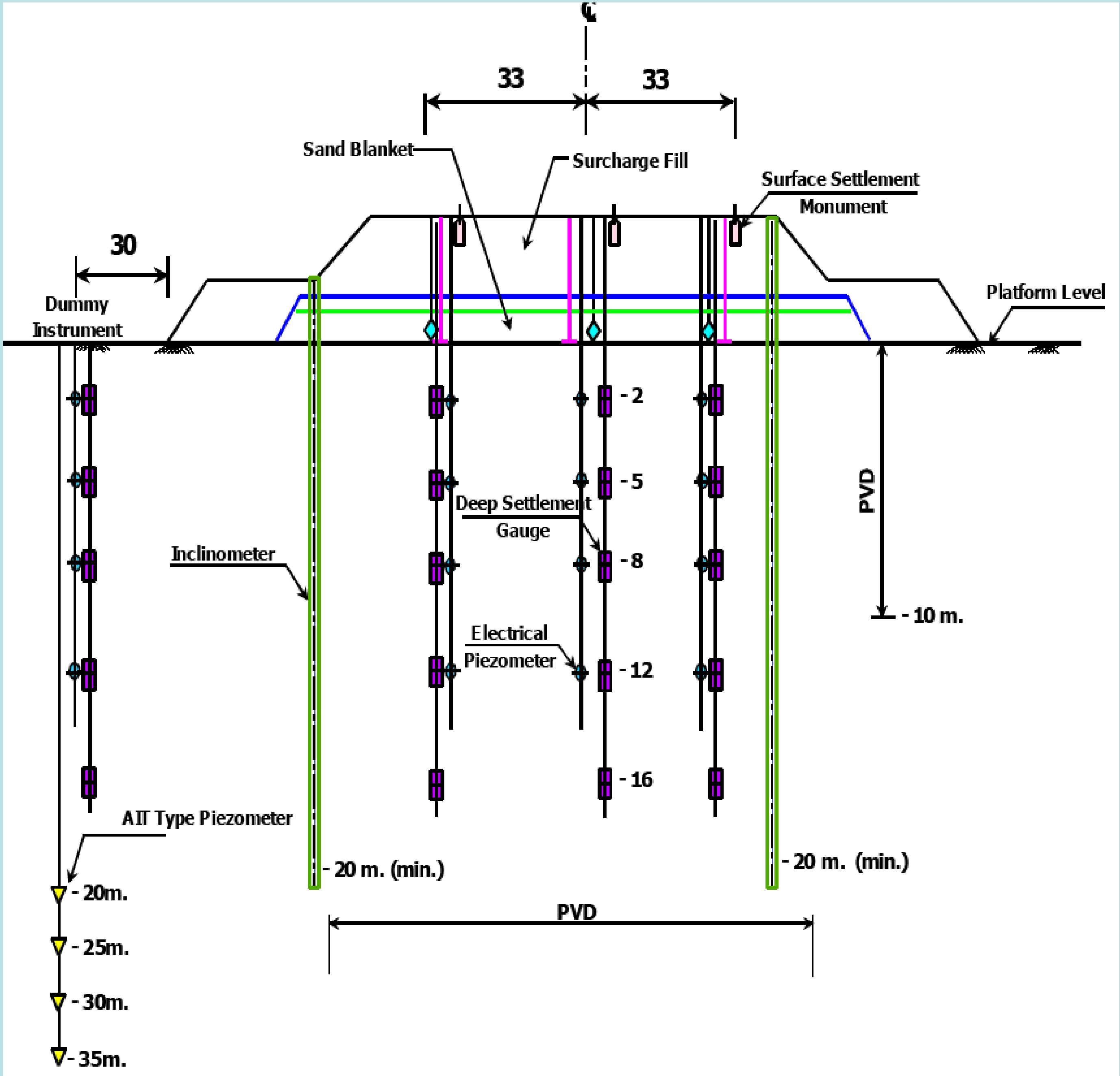


GILRS

Typical Cross Section of Instrumentation

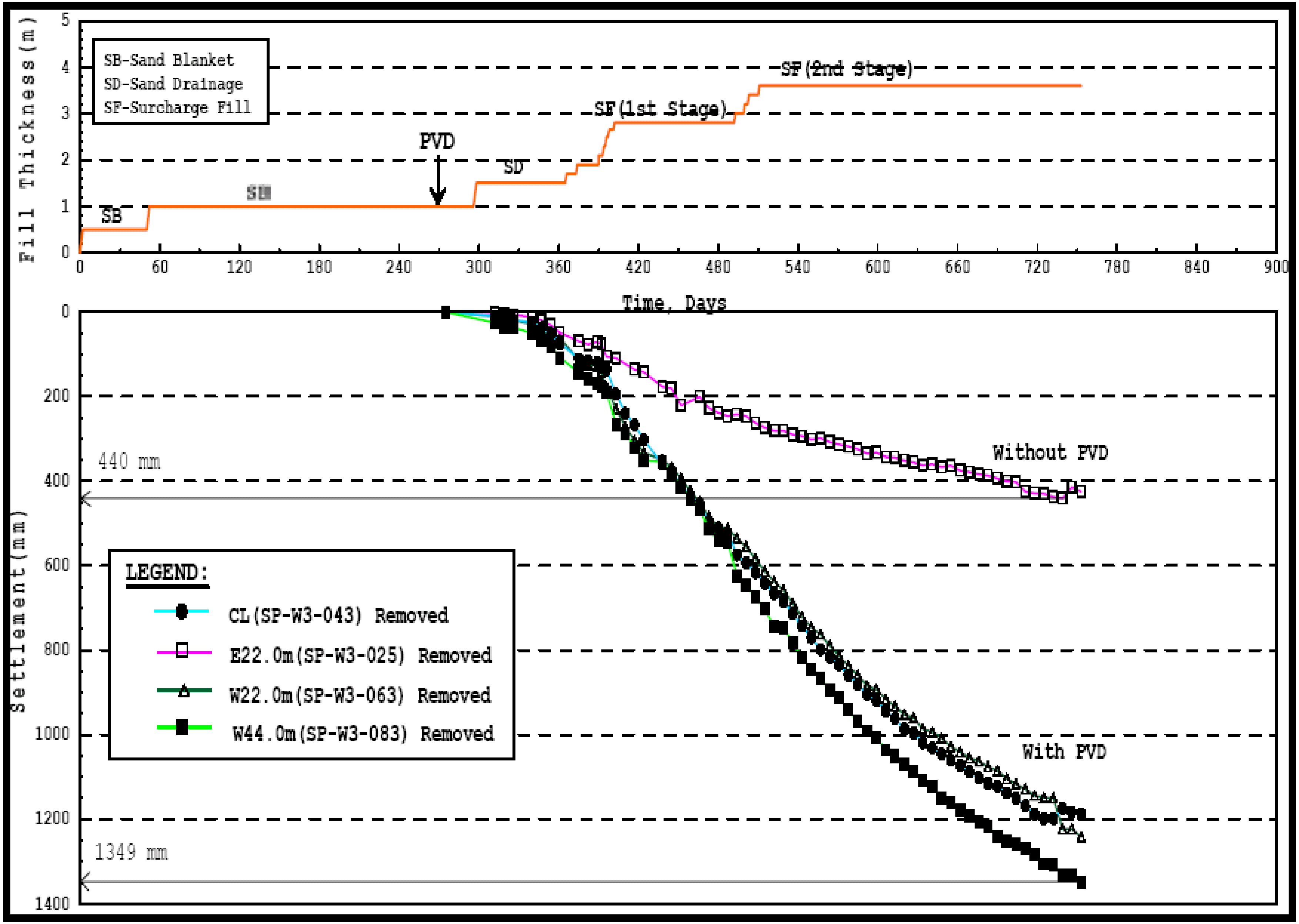


B
B/6
B/6
B/6
B/6
B/6
B/6
OBSERVATION WELL @25m.
SURFACE SETTLEMENT PLATE @200 m.
SURFACE SETTLEMENT PLATE @200 m.
PRELOADING
SURFACE SETTLEMENT PLATE @200 m.
DRAINAGE SAND BLANKET
DUMMY INSTRUMENTS
5050
6050
GROUND SURFACE AFTER SITE PREPARATION
INCLINOMETER @600 m. AT A DEPTH OF 17 m. MIN. AND AT LEAST 2.0 m. INTO STIFF CLAY
INCLINOMETER @600 m. AT A DEPTH OF 17 m. MIN. AND AT LEAST 2.0 m. INTO STIFF CLAY
PNEUMATIC PIEZOMETERS @600 m. AT DEPTHS OF 4 m. 8 m. AND 12 m.
DEEP SETTLEMENT GAUGE @600 m. AT A DEPTH OF 27 m. MIN. AND AT LEAST 2.0 m. INTO DENSE SAND
PIEZOMETER

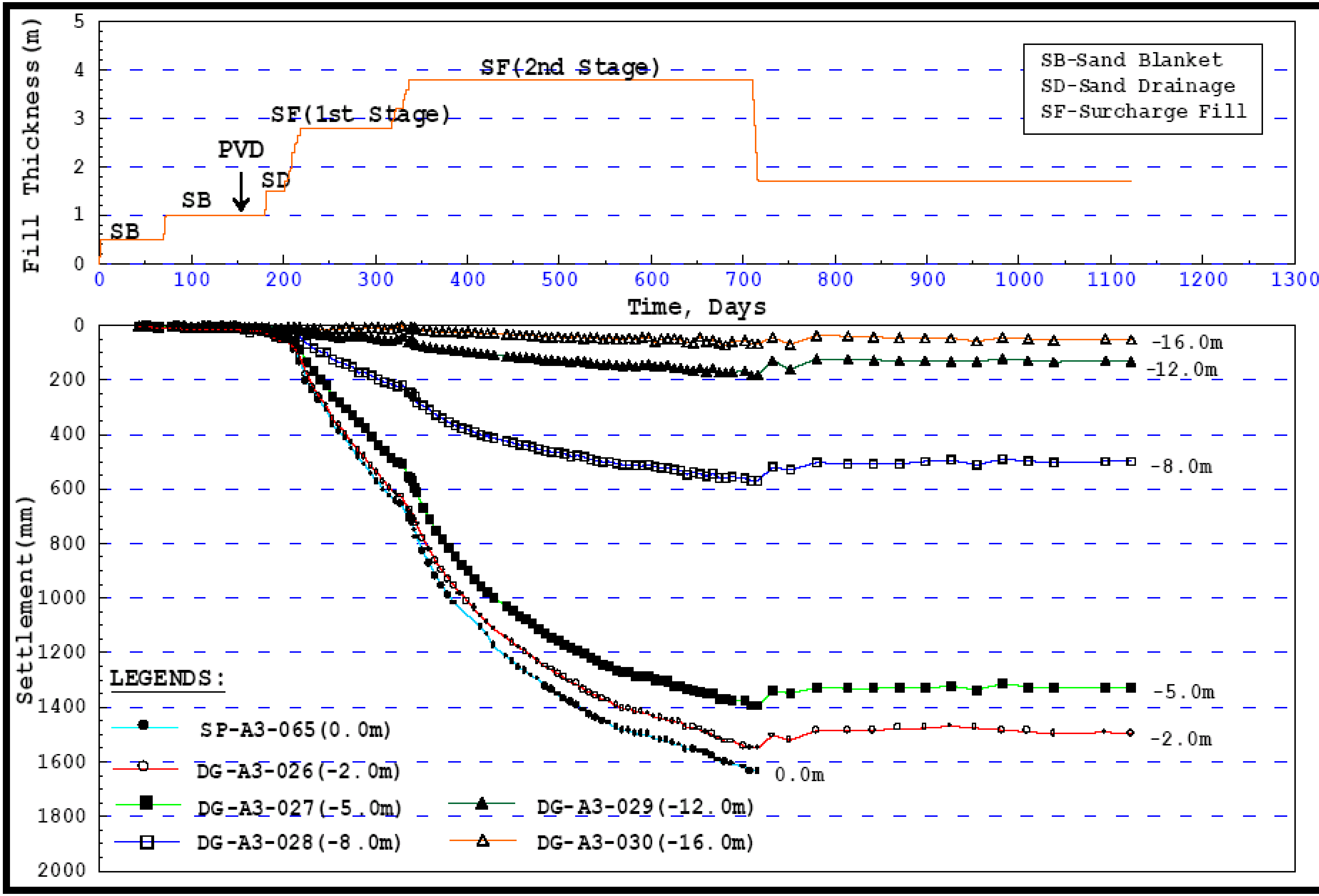


℄
33
33
Sand Blanket
Surcharge Fill
Surface Settlement
Monument
30
Dummy
Instrument
Platform Level
- 2
- 5
Deep Settlement
Gauge
- 8
PVD
- 10 m.
Inclinometer
Electrical
Piezometer
- 12
- 16
AIT Type Piezometer
- 20 m. (min.)
- 20 m. (min.)
- 20m.
- 25m.
- 30m.
- 35m.
PVD

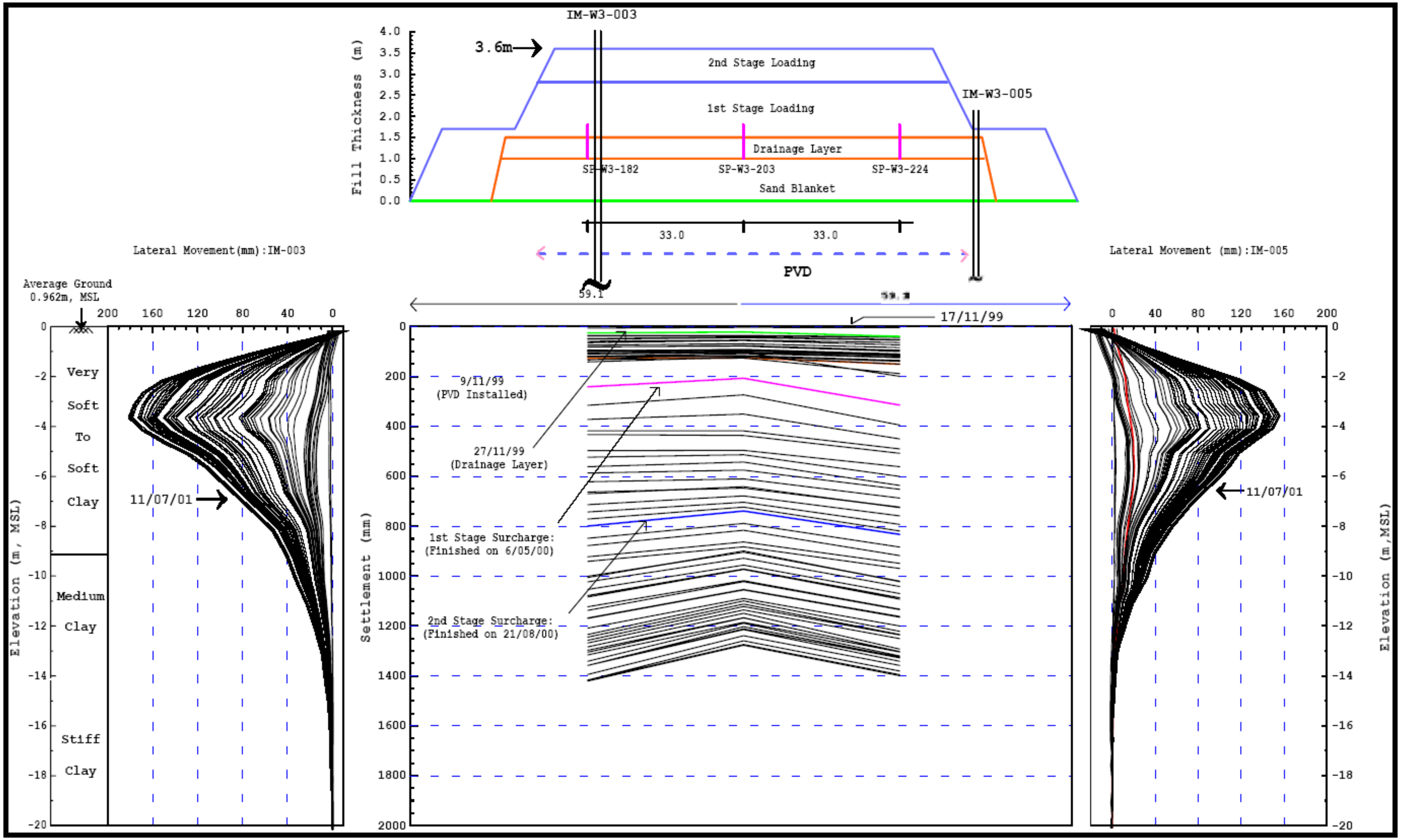
Settlement Contours in the Apron Area



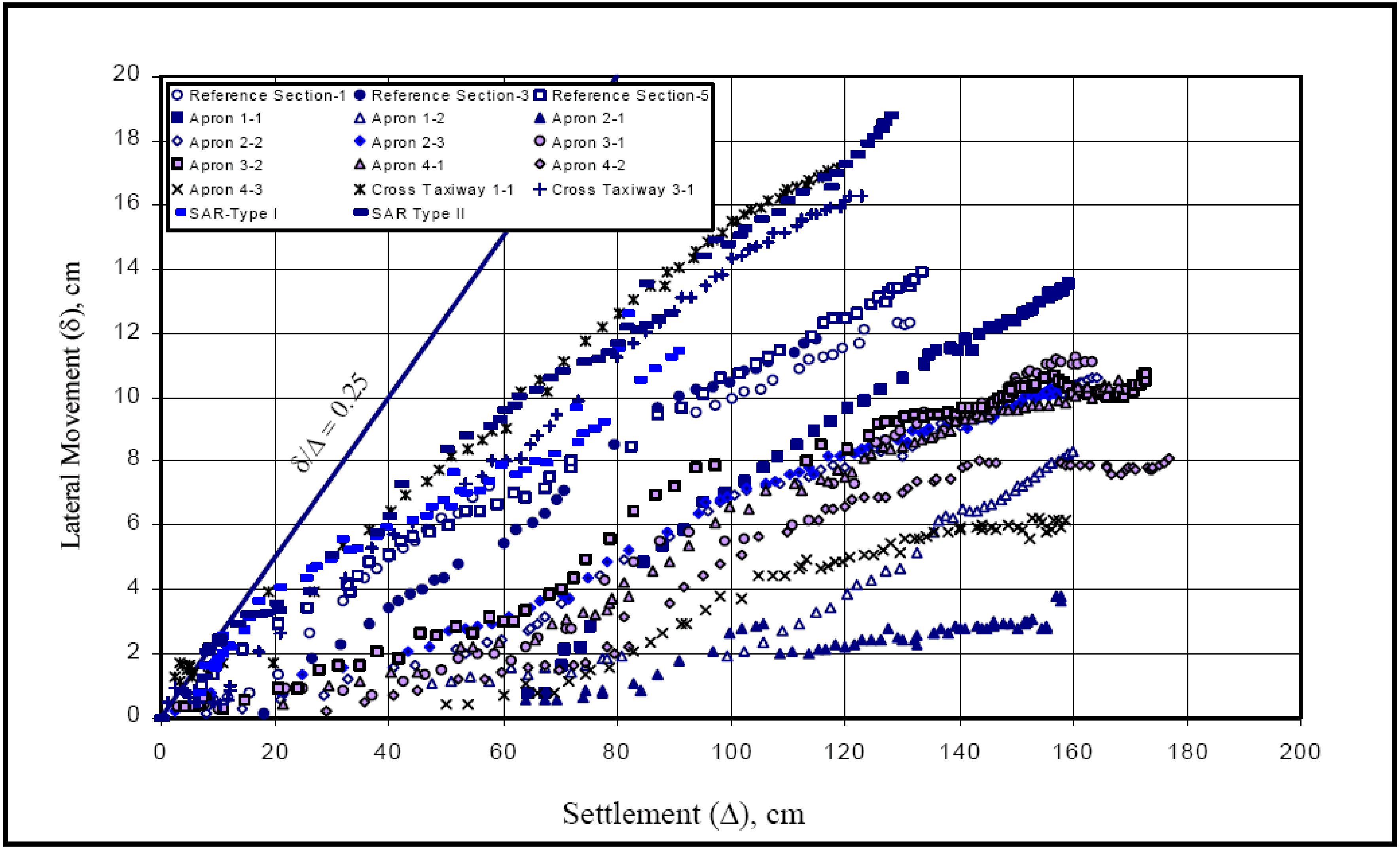
Comparison of Settlement between PVD and Non-PVD Area



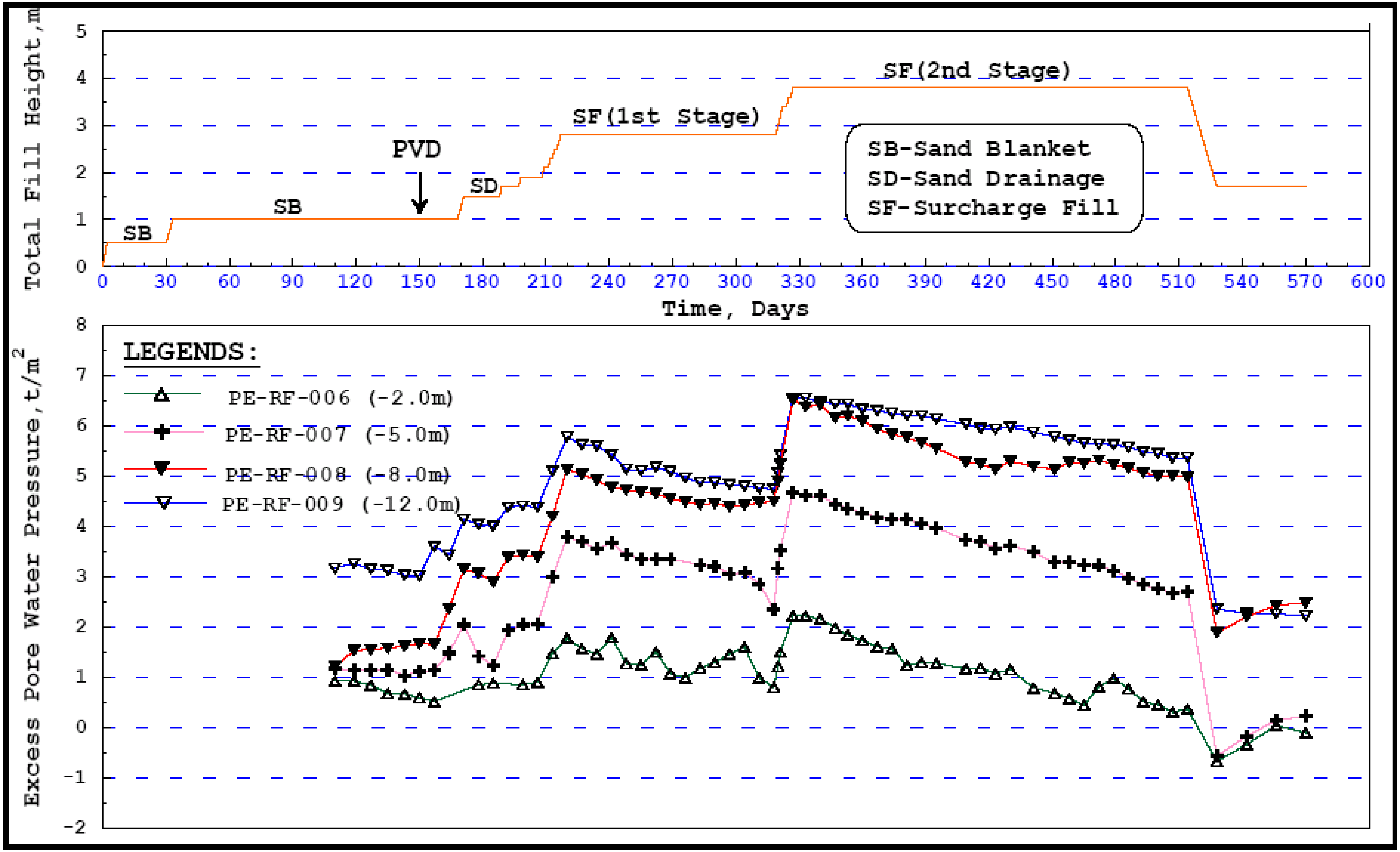
Deep Settlement Profile in the Apron Area



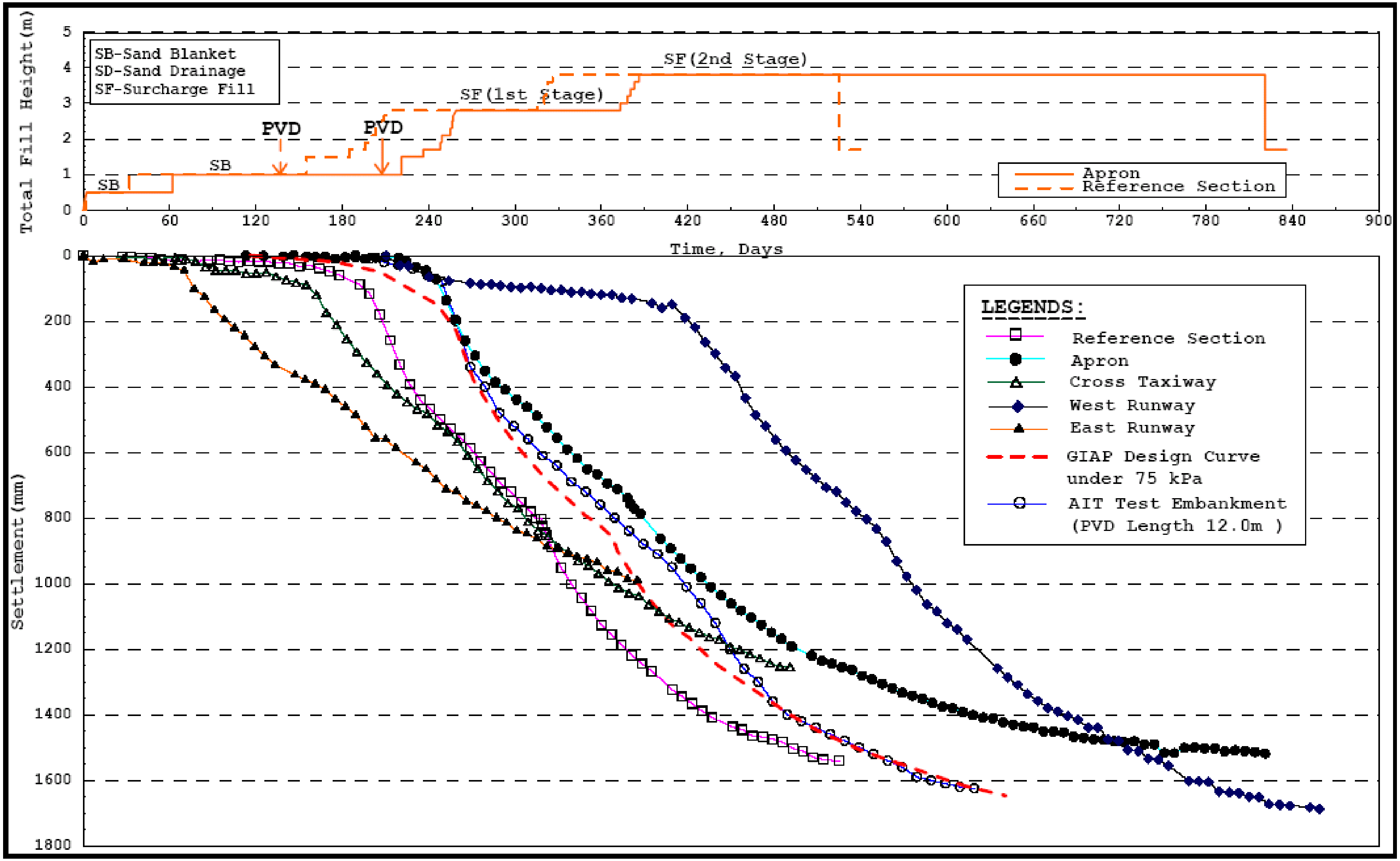
Observed Settlement and Lateral Movement Profile at West Runway



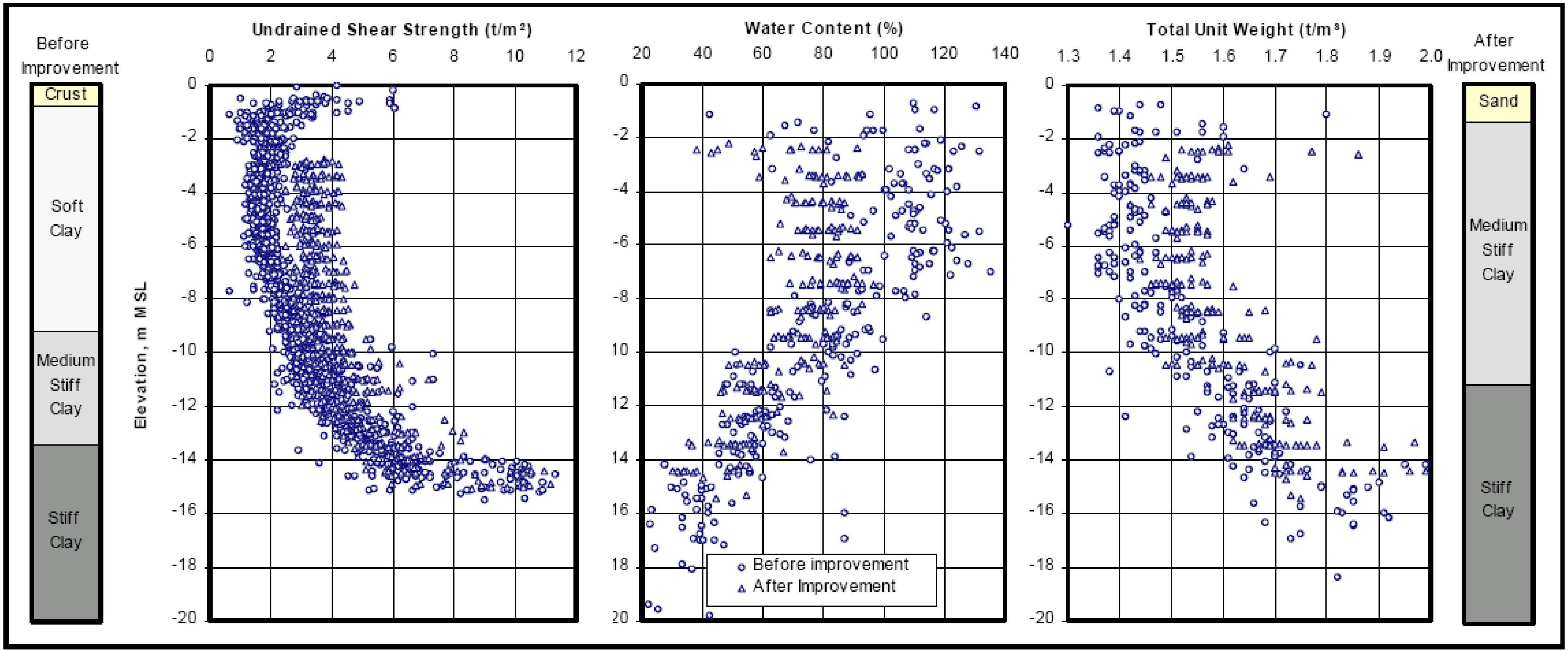
Ratios of Lateral to Vertical Movement at Apron, Cross Taxiway (GIAP) and SAR (GILRS)



Observed Excess Pore Pressure Distribution with Time and Fill Status at the Reference Section



Comparison of Field Settlements with the Design and AIT Test Embankment



Comparison of Soil Properties Before and After Ground Improvement