Bengt H. Fellenius

FOUNDATIONS FOR THE NEW INTERNATIONAL AIRPORT IN BANGKOK, THAILAND

ASCE Seattle Section

Geotechnical Group

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FOUNDATIONS FOR THE NEW INTERNATIONAL AIRPORT IN BANGKOK, THAILAND

Bengt H. Fellenius

A presentation to the ASCE Seattle Section 2009 Spring Seminar on

Recent Developments and Case Histories in Deep Foundations.

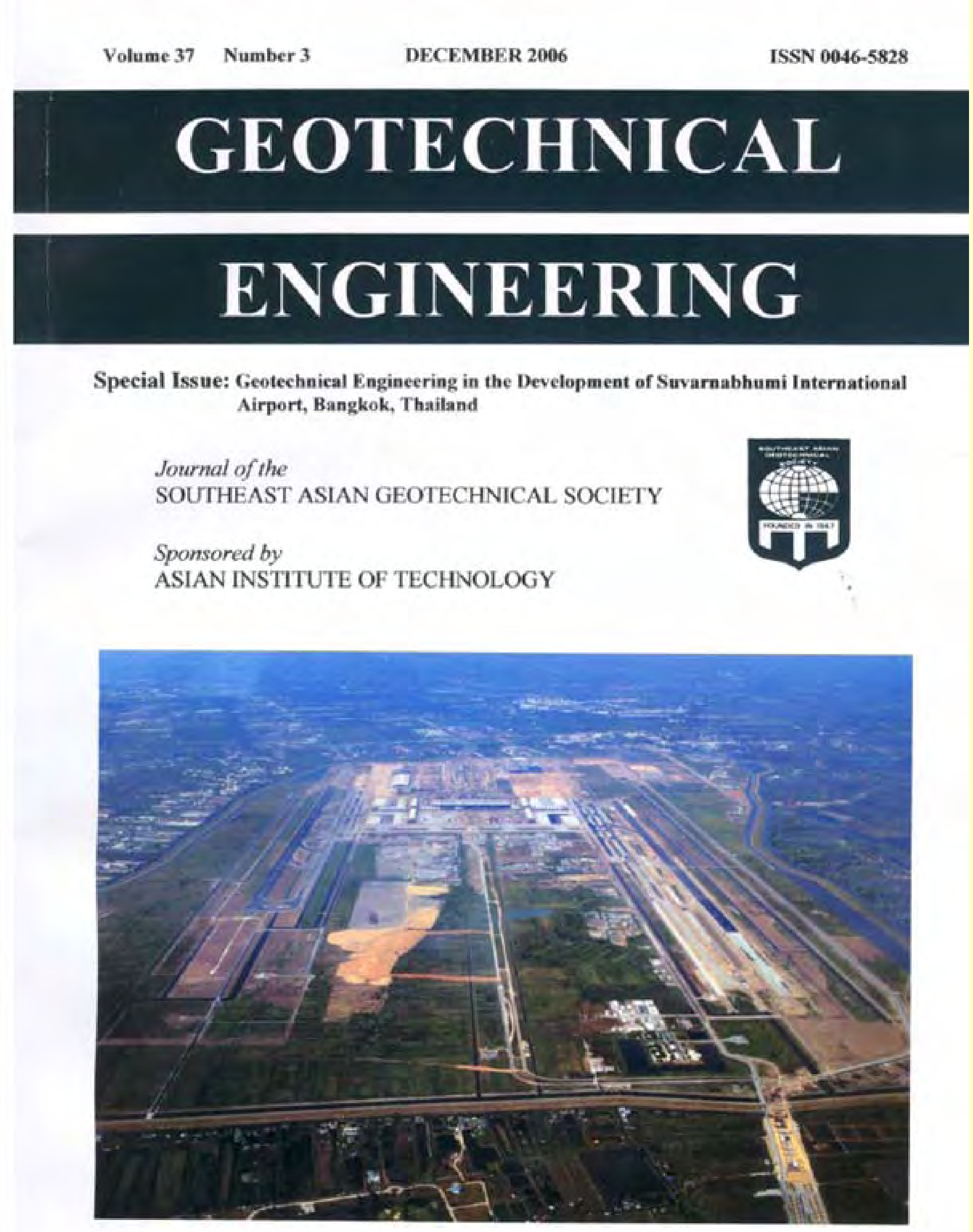
Abstract

The Suvarnabhhumi airport, the new international airport in Bangkok, Thailand, covering an area of 8 Km by 4 Km (8,000 acres), is located in a former swamp, a flat marine delta about 30 Km outside Bangkok. The soil profile consists of a thin weathered crust on typical soft to stiff, compressible Bangkok clay deposited on a sand layer at a depth of about 25 m extending to about 47 m. Below the sand lies an about 10 m thick layer of hard silty clay followed by very dense sand to large depth. Most of the area is devoted to runways, roadways, and parking, which required extensive ground improvement to minimize settlement. The structures consist of several units sharing footprints**:** terminal building, conourse, trellis structure, parking garage, and elevated roadways. The structures are founded on three types of piles installed to the sand below the clay layer, 1,000 mm bored pile, 600 mm diameter bored piles, and 600 mm driven cylinder piles. The stress-bulbs from the various foundations overlap resulting in a complicated settlement analysis. A total of 25,000+ piles were installed. The design of the airport started in 1995 and construction was completed in 2005 at a total cost of close to us$30 billion. The lecture will present aspects of the soil improvement work, analysis of results from pile tests, and the design of the piled foundations for capacity, settlement, and downdrag. The main part of the presentation consists of information quoted from papers published in Geotechnical Engineering Special Issue, Vol.37, No. 3, December 2006.

The New International Airport, Bangkok Thailand

Foundation Design by
TAMS/Earth Tech, NY
with Dr. Bengt H. Fellenius as
outside consultant

The presentation is primarily based on papers published in the Special Issue of the Journal of South-East Asian Geotechnical Society, "Geotechnical Engineering", December, 2006, as listed in the next slide.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GEOTECHNICAL ENGINEERING

Special Issue: Geotechnical Engineering in the Development of Suvarnabhumi International Airport, Bangkok, Thailand

Journal of the
SOUTHEAST ASIAN GEOTECHNICAL SOCIETY

Sponsored by
ASIAN INSTITUTE OF TECHNOLOGY

Buttling, S. 2006. Bored piles and bi-directional load tests. Special Issue of the Journal of South-East Asian Geotechnical Society, December 2006, 37(3) 207-215.

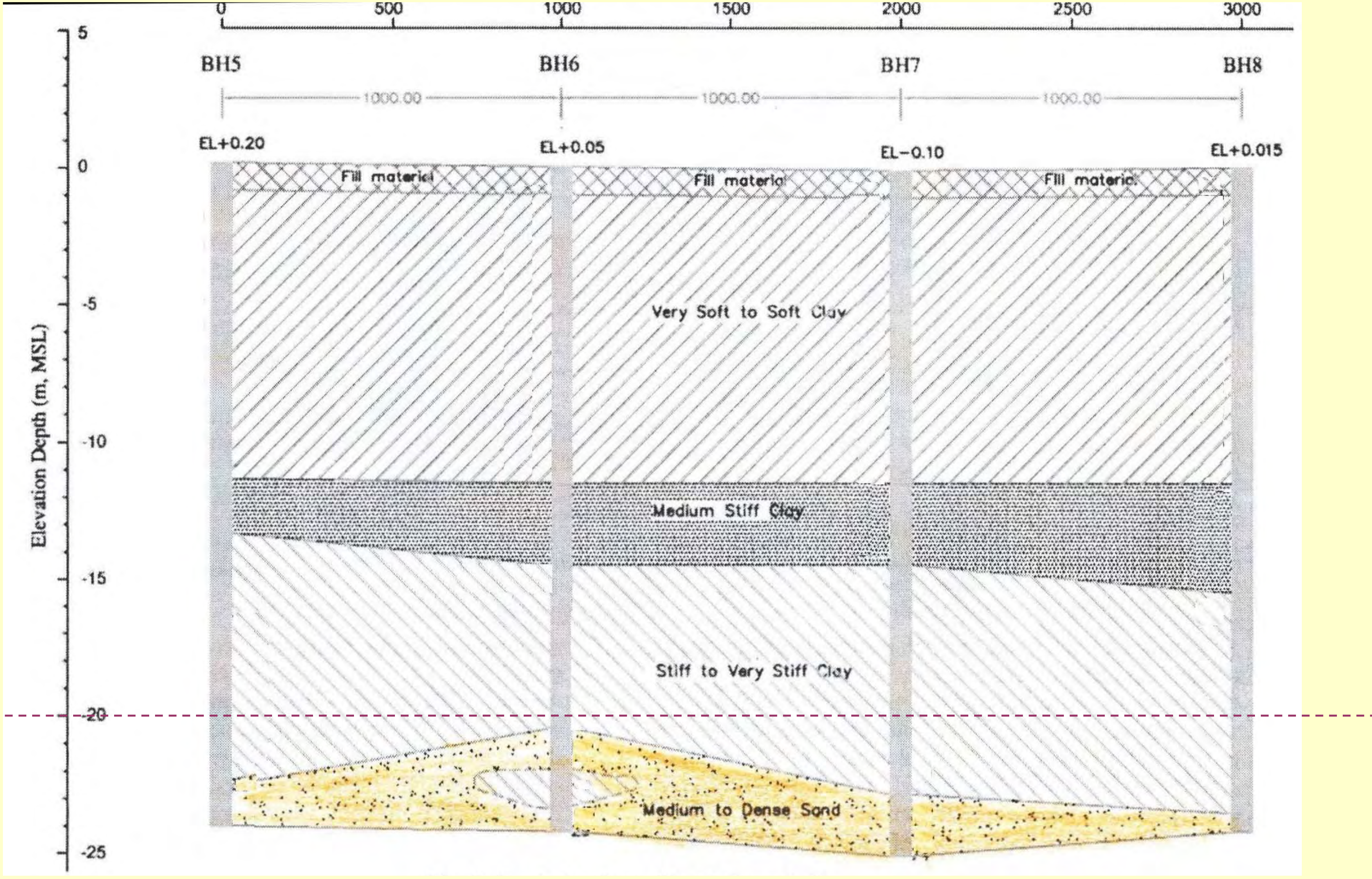
Cortlever, N.G., Visser, G.T, and deZwart, T.P., 2006. Geotechnical History of the development of the Suvarnabhumi International Airport. Special Issue of the Journal of South-East Asian Geotechnical Society, December 2006, 37(3) 189-194.

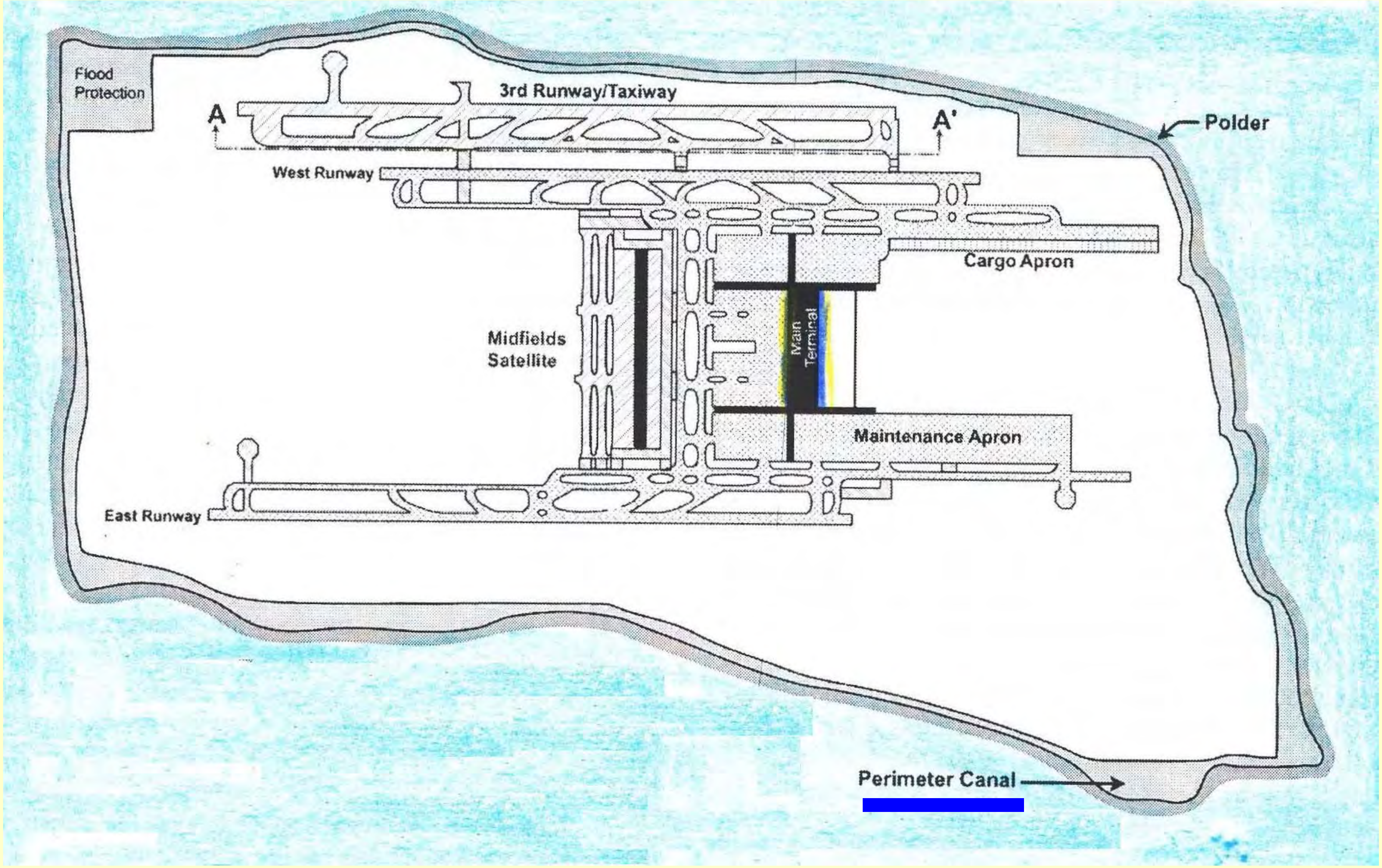
Moh, Z.C. and Lin, P.C., 2006. Geotechnical History of the development of the Suvarnabhumi International Airport. Special Issue of the Journal of South-East Asian Geotechnical Society, December 2006, 37(3) 143-170.

Seah, T.H., 2006. Design and construction of ground improvement works at Suvarnabhumi International Airport. Special Issue of the Journal of South-East Asian Geotechnical Society, December 2006, 37(3) 171-188.

AND

Fox, I., Du, M. and Buttling, S, 2004. Deep Foundations For New International Airport Passenger Terminal Complex in Bangkok. Proceedings of the Fifth International Conference on Case Histories in Geotechnical Engineering, New York, April 13-14, Paper 1.22, 11 p.

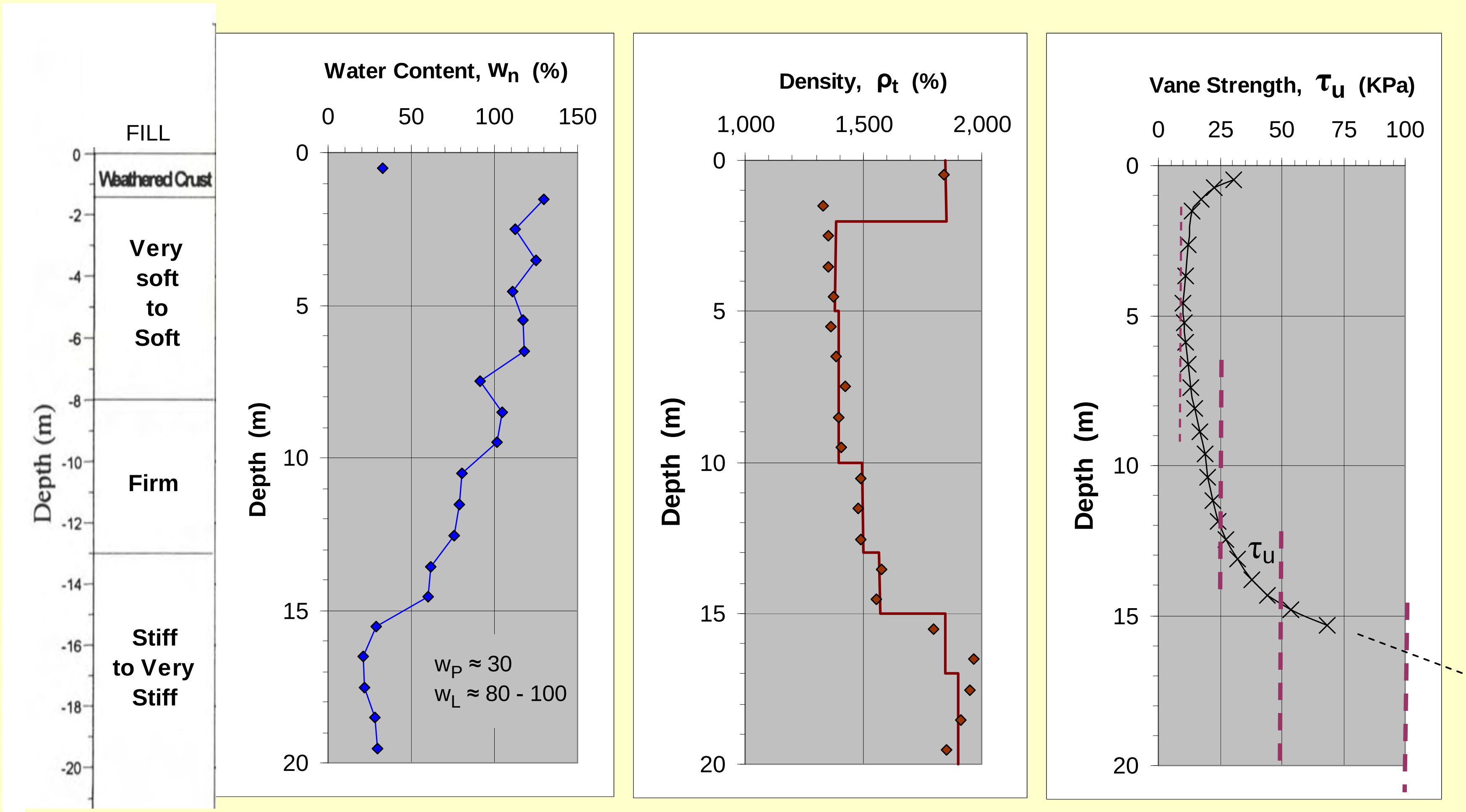


0
500
1000
1500
2000
2500
3000
5
0
-5
-10
-15
-20
-25
Elevation Depth (m, MSL)
BH5
BH6
BH7
BH8
1000.00
1000.00
1000.00
EL+0.20
EL+0.05
EL-0.10
EL+0.015
Fill material
Fill material
Fill material
Very Soft to Soft Clay
Medium Stiff Clay
Stiff to Very Stiff Clay
Medium to Dense Sand



Suvarnabhumi International Airport

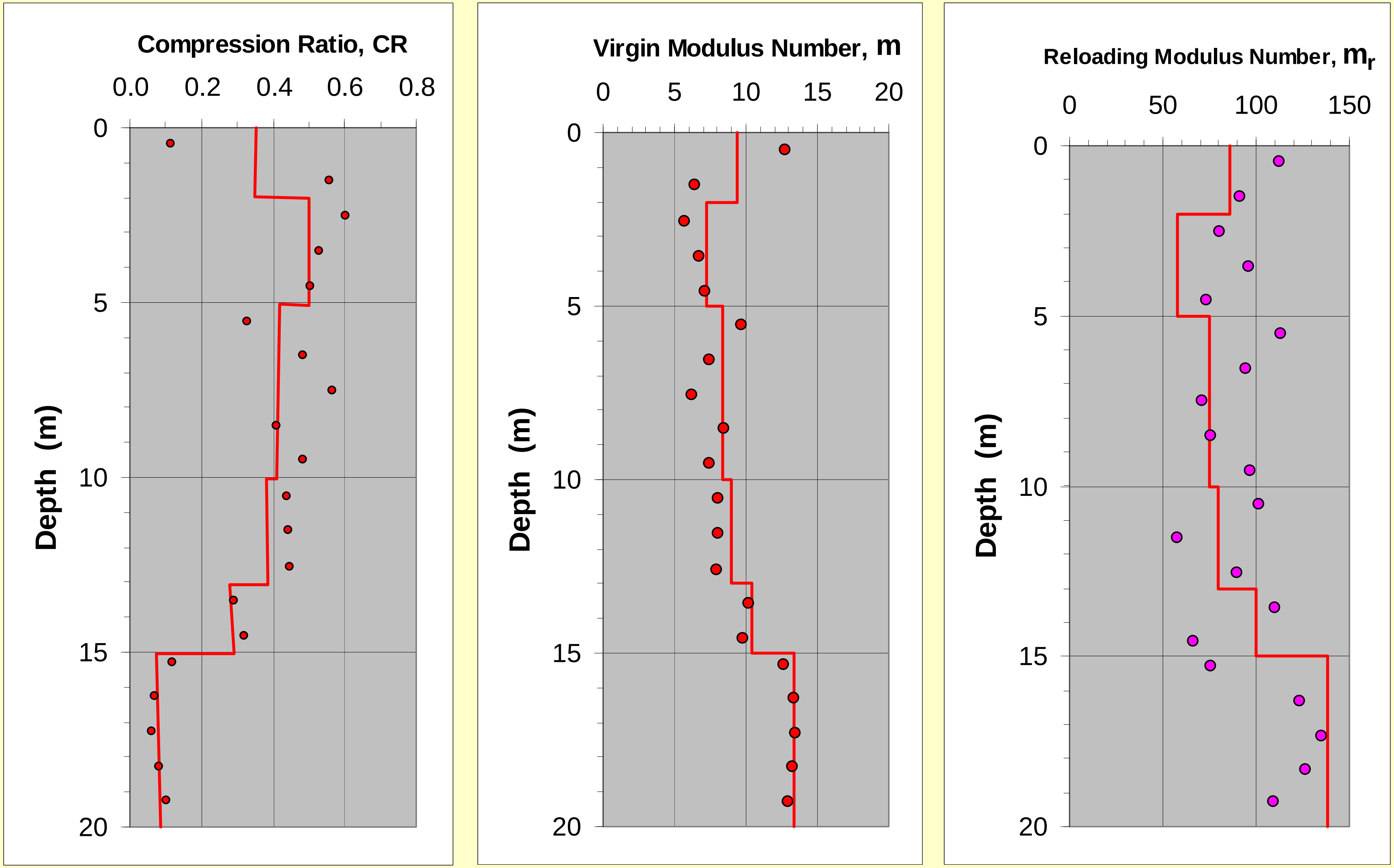
An **8 Km x 4 Km** area close to the Gulf of Thailand

Basic Soil Parameters



Data from: Fox, Du, and Buttling, (2004), Buttling (2006), Moh and Lin (2006), Seah (2006), Cortlever, Visser, and deZwart (2006)

Compressibility



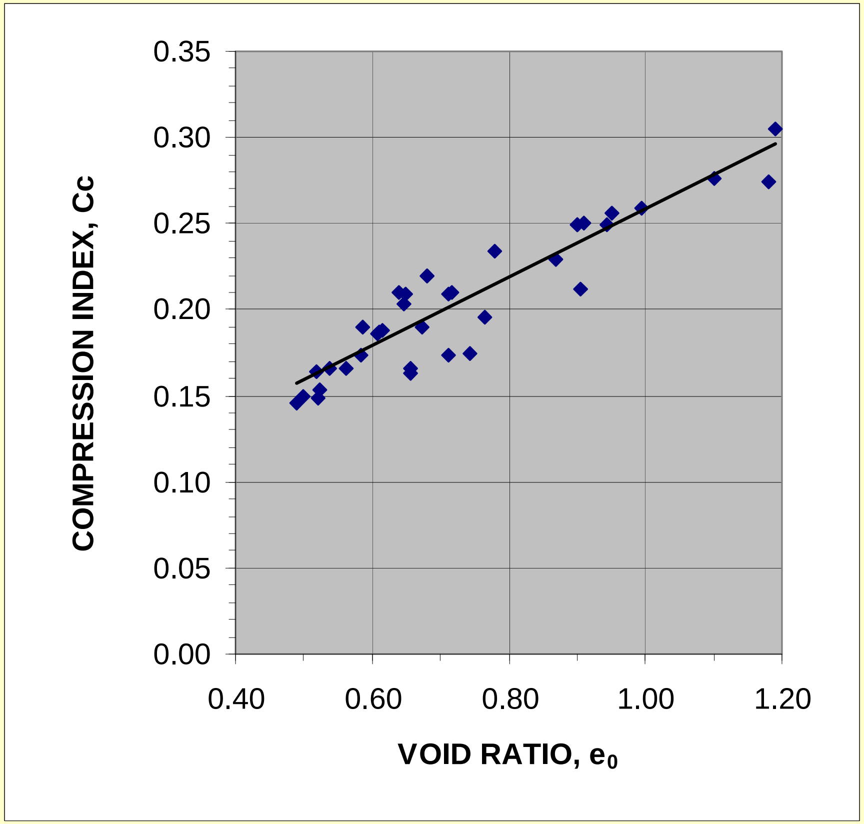
$CR = C_c/(1+ e_0)$

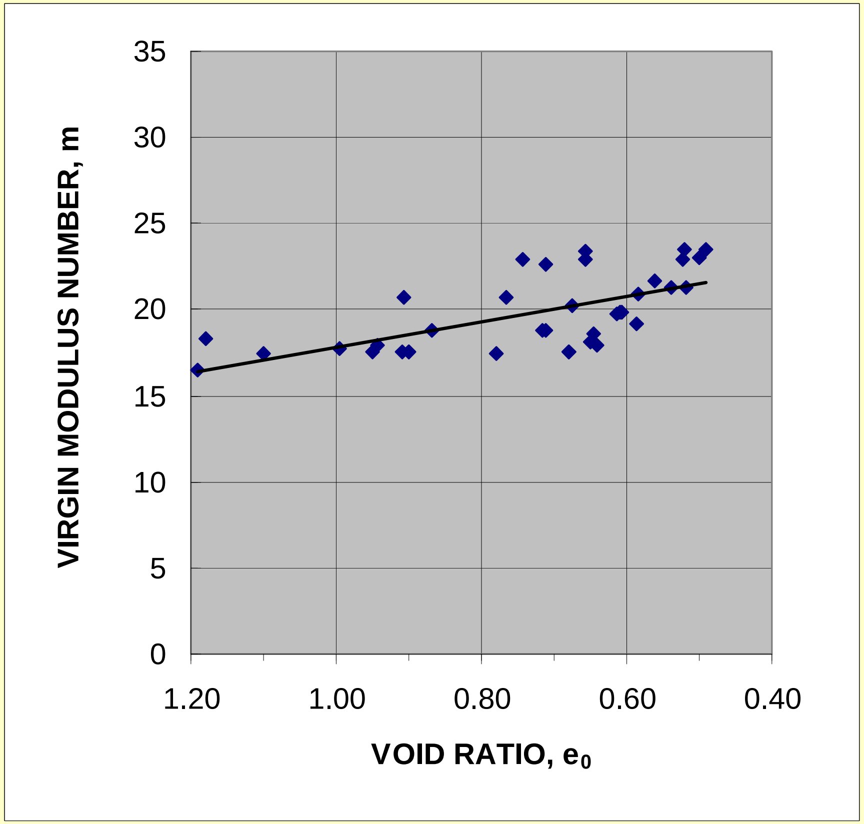
$m = \ln10\ (1 + e_0)/C_c$

Red Lines show distribution for design

Circles are data points

Comparison between the C_c/e_0 approach and the Janbu Modulus Number method





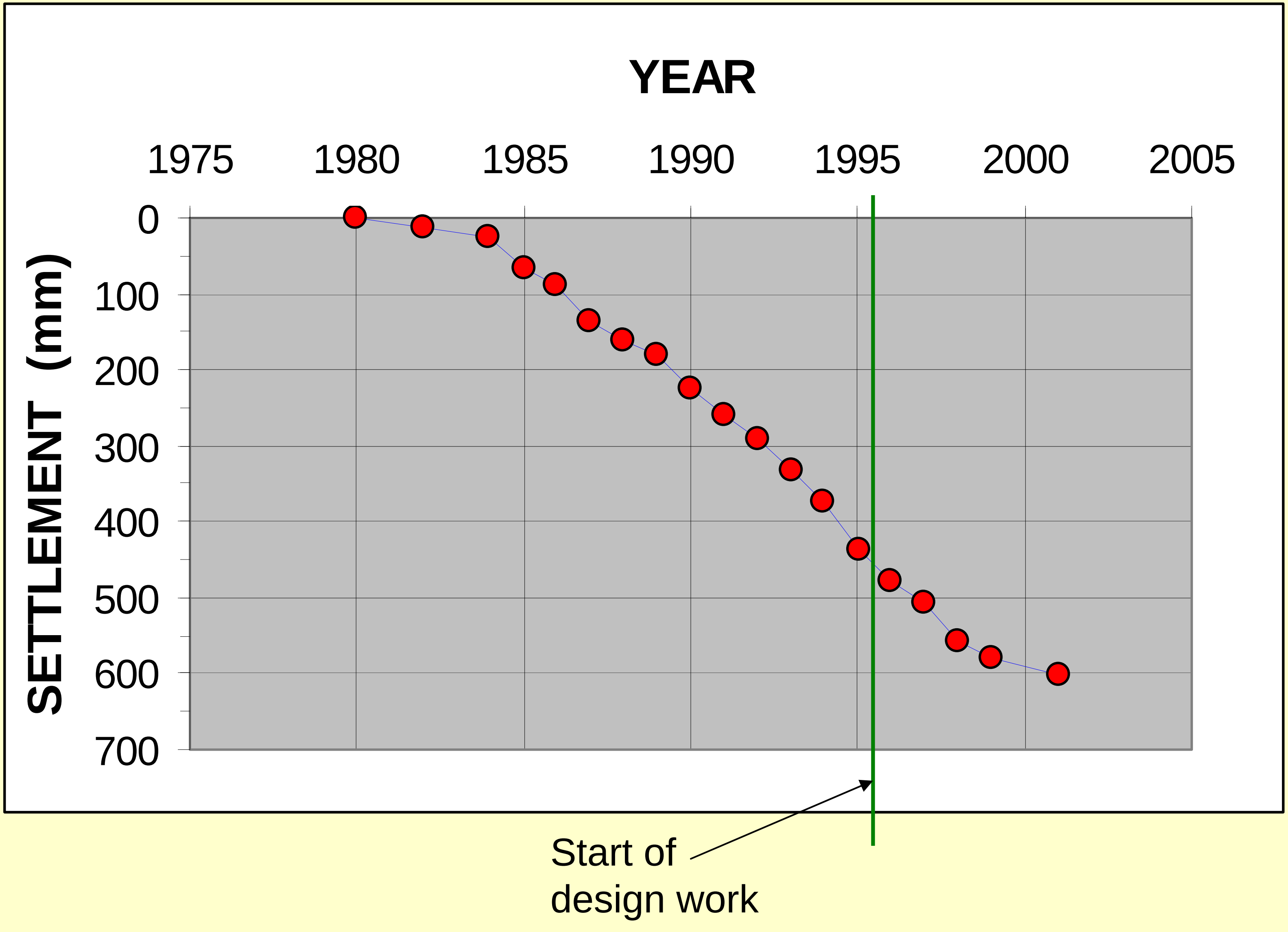
Data from a 20 m thick sedimentary deposit

The C_c-e_0 approach (based on C_c) implies that the the compressibility varies by 30± %.

However, the Janbu methods shows it to vary only by 10± %. The modulus number, m, ranges from 18 through 22; It would be unusual to find a clay with less variation.

Settlement of the Ground Surface Observed at Airport Site

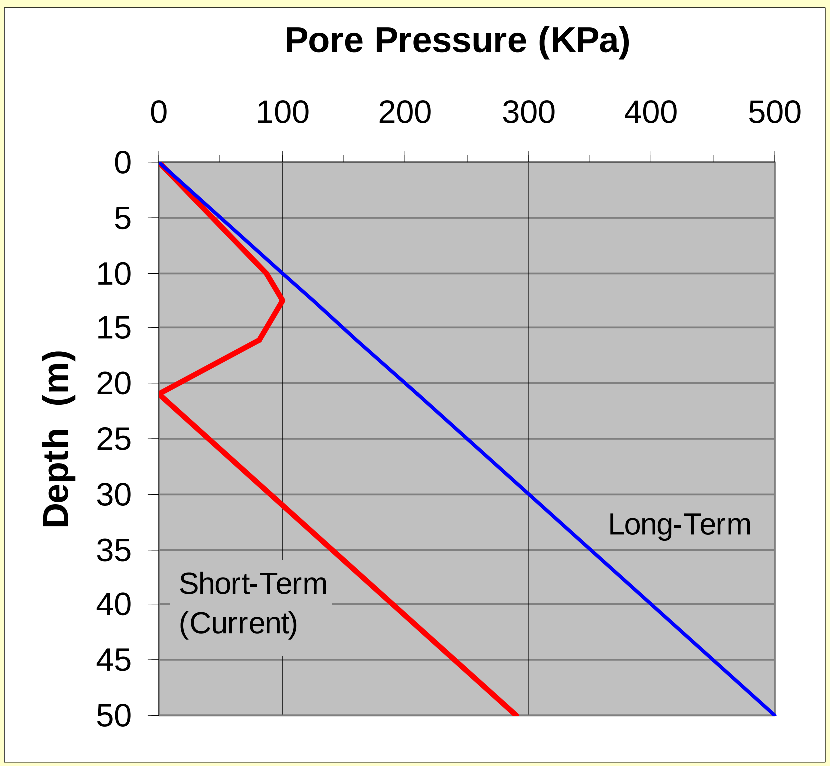
Regional settlement occurs at and around the airport area due to mining of ground water



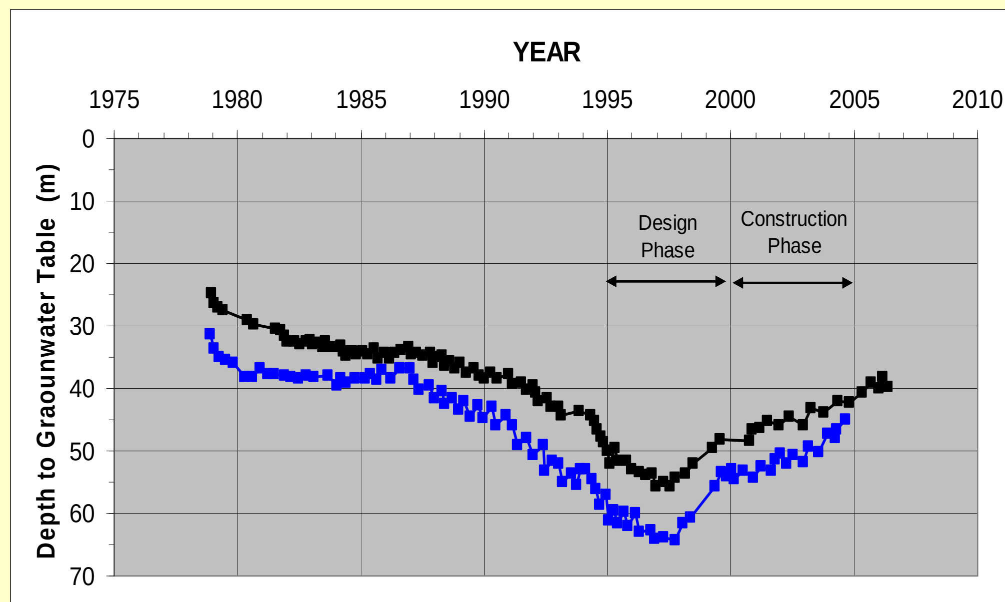
Pumping (mining) of groundwater has reduced the pore pressures in the Bangkok delta resulting in significant regional settlement. In 1996, coinciding the beginning of the design process, pumping in the area was stopped. Pore pressure measurements indicate that the desired effect is being reached; the pore pressures are rising and the distribution may become hydrostatic in the future.

Current and Future (long-term)

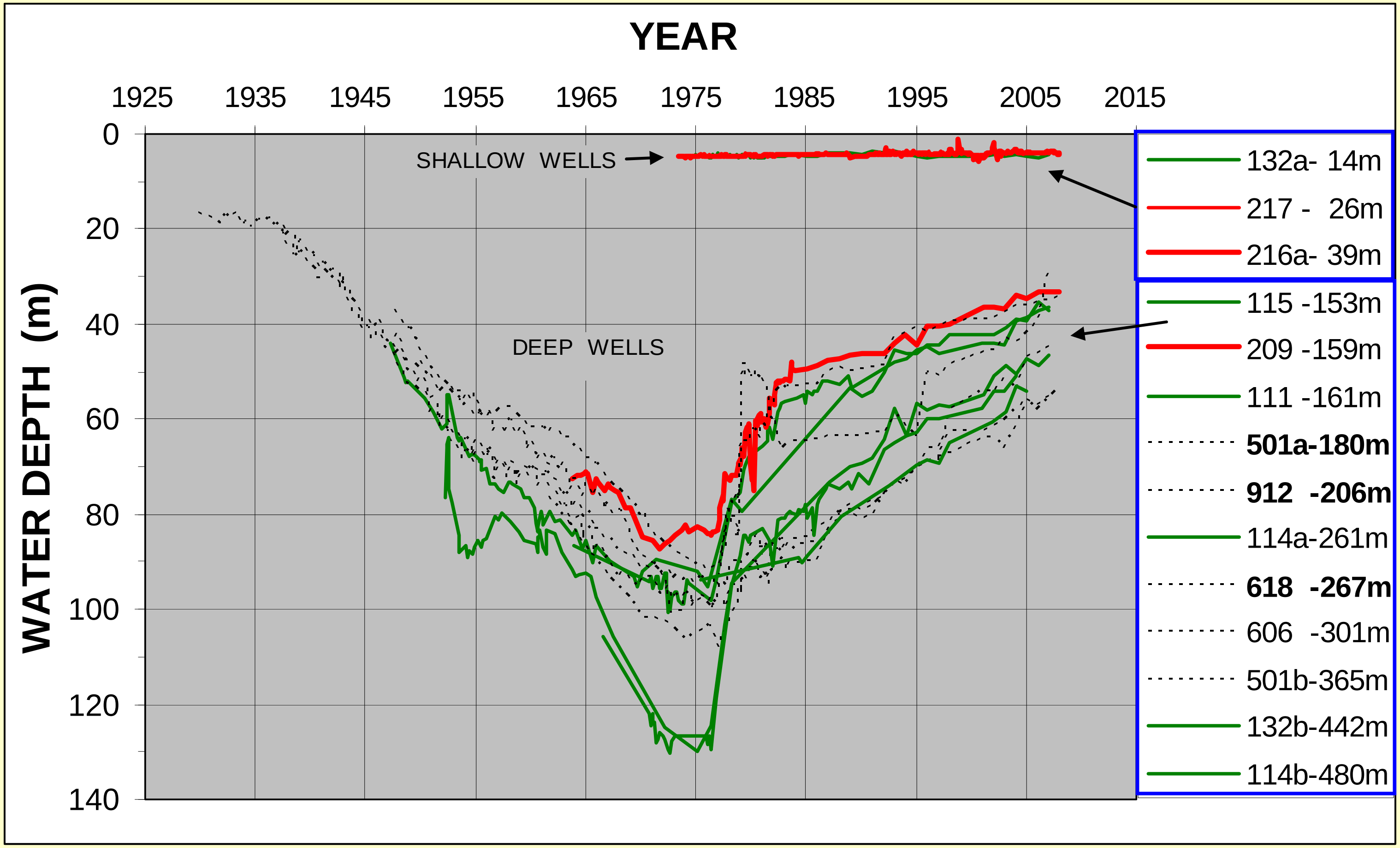
Pore Pressure Distribution



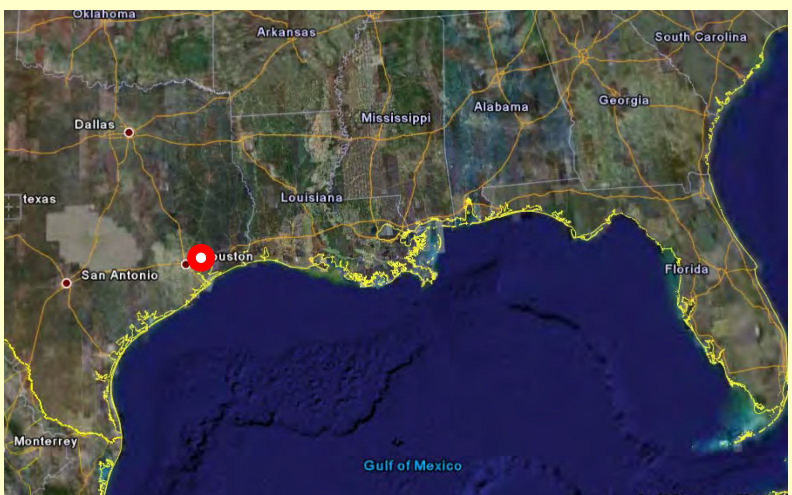
Nearby Observations of Groundwater Table

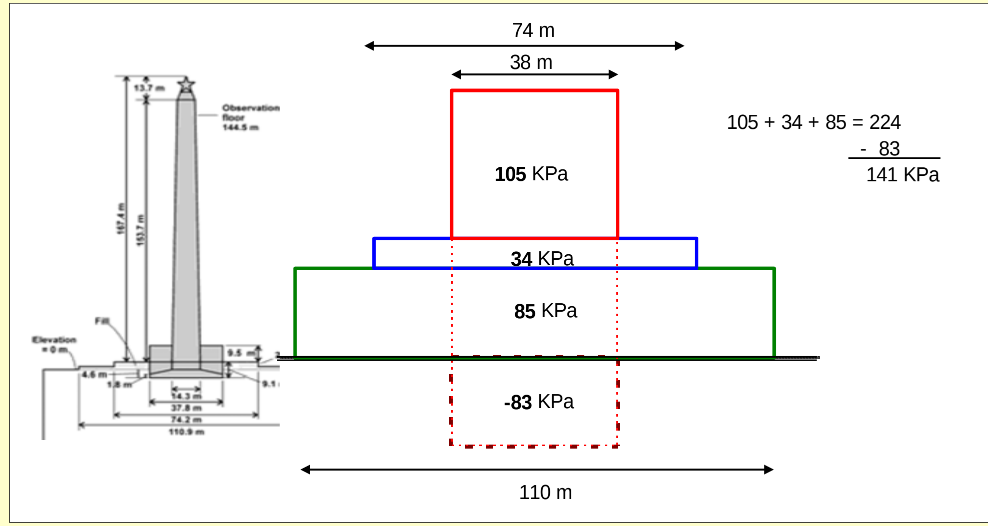


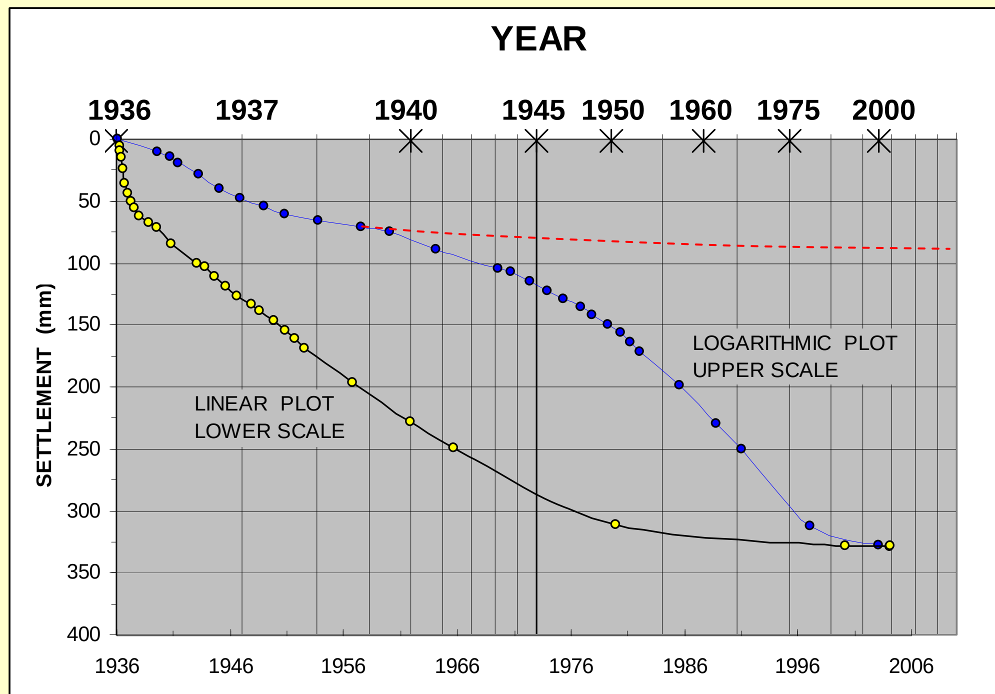
The lowering of the groundwater table due to mining of water in the Bangkok delta is not unique. Below is a compilation of depth to the water table measured in the San Jacinto-Houston-Pasadena area in Texas.



The San Jacinto Monument.

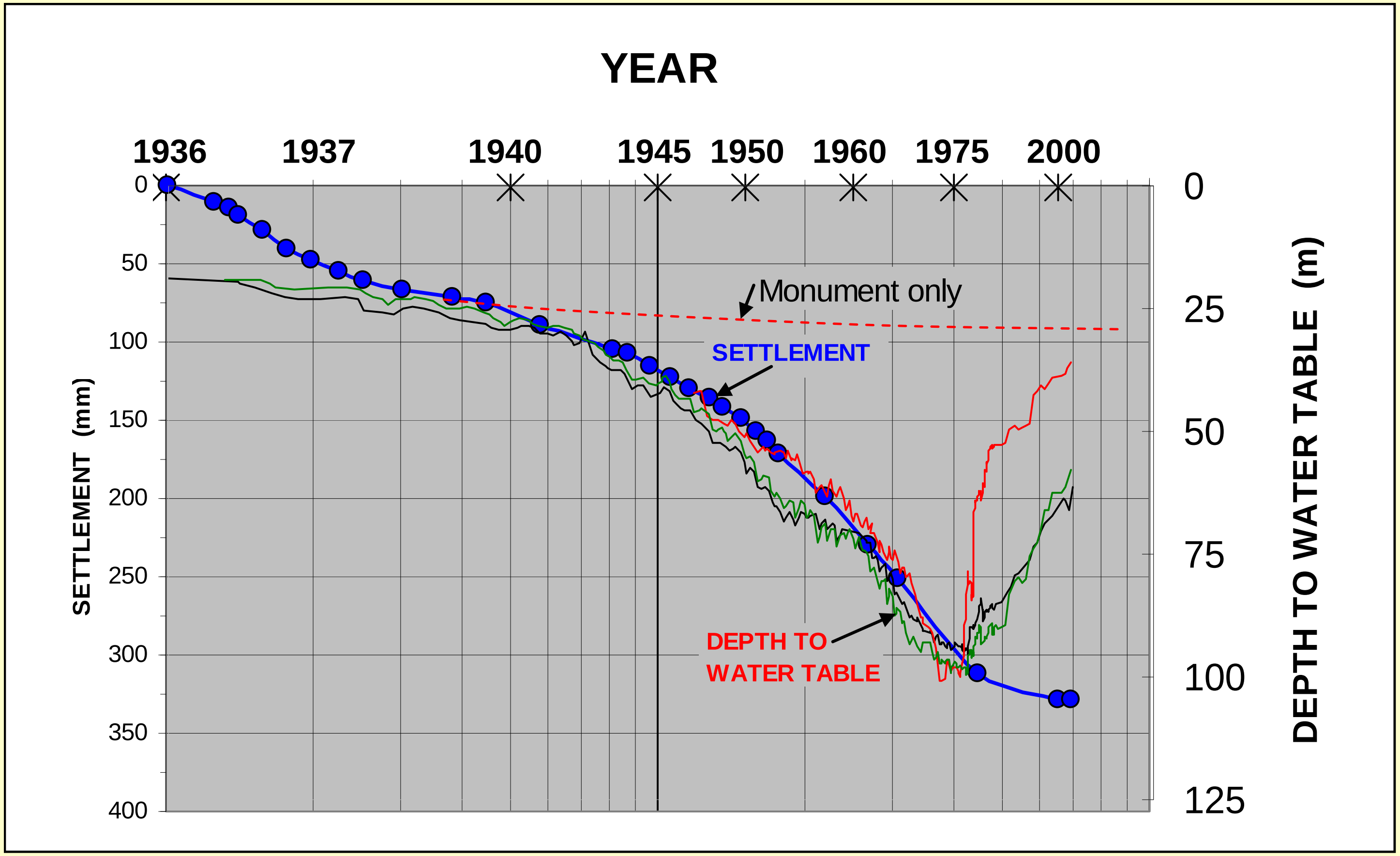






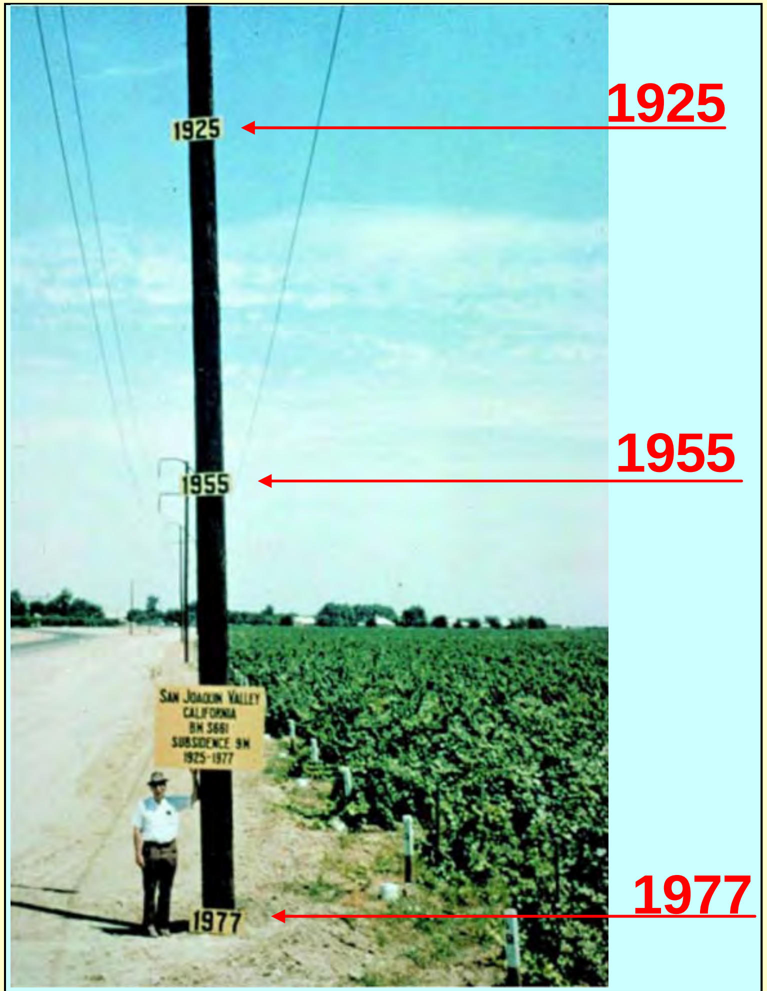
Briaud et al. 2007; Fellenius and Ochoa 2008

Measured depths to water table and measured settlement of the Monument plus estimated settlement of the Monument had there been no drawdown of the water table.

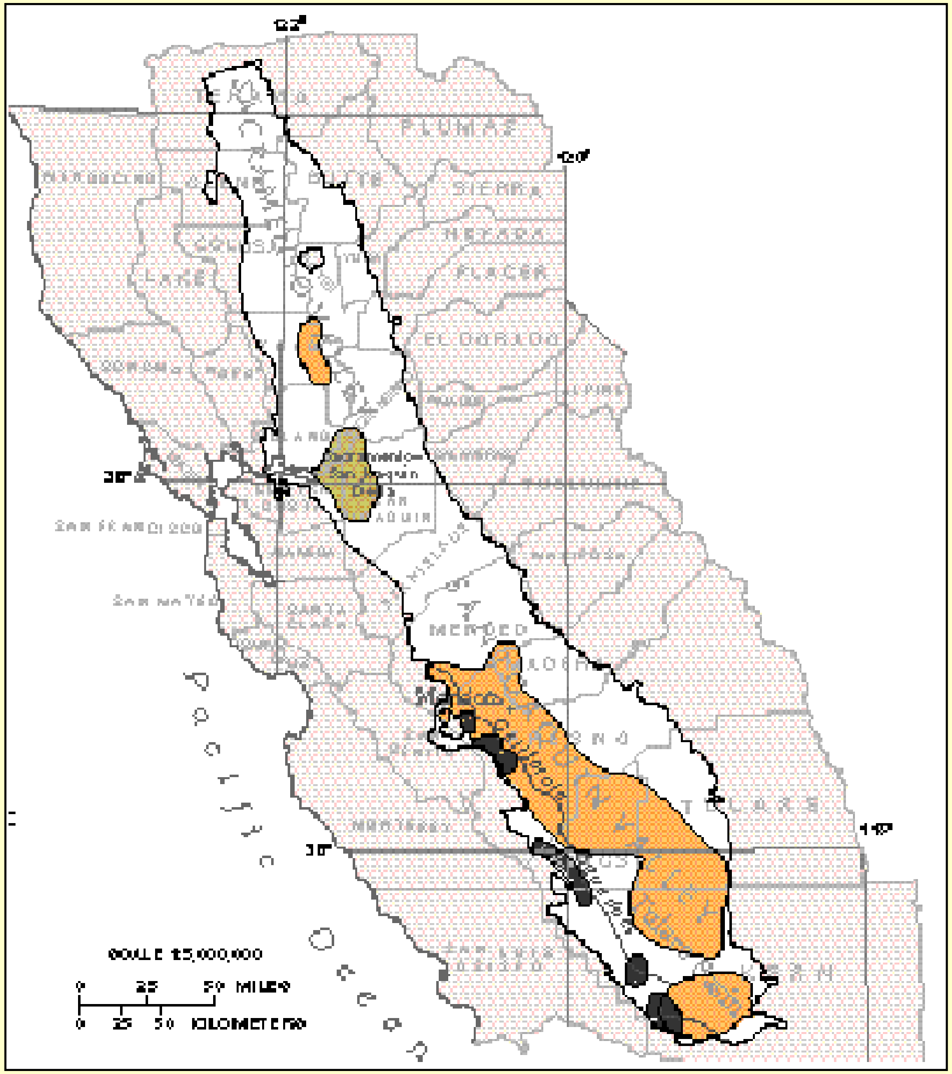


And in the San Joaquin Valley in California:

Approximate location of maximum subsidence in United States identified by research efforts of Joseph Poland (pictured).Signs on pole show approximate altitude of land surface in 1925, 1955, and 1977. The pole is near benchmark S661 in the San Joaquin Valley southwest of Mendota, California,



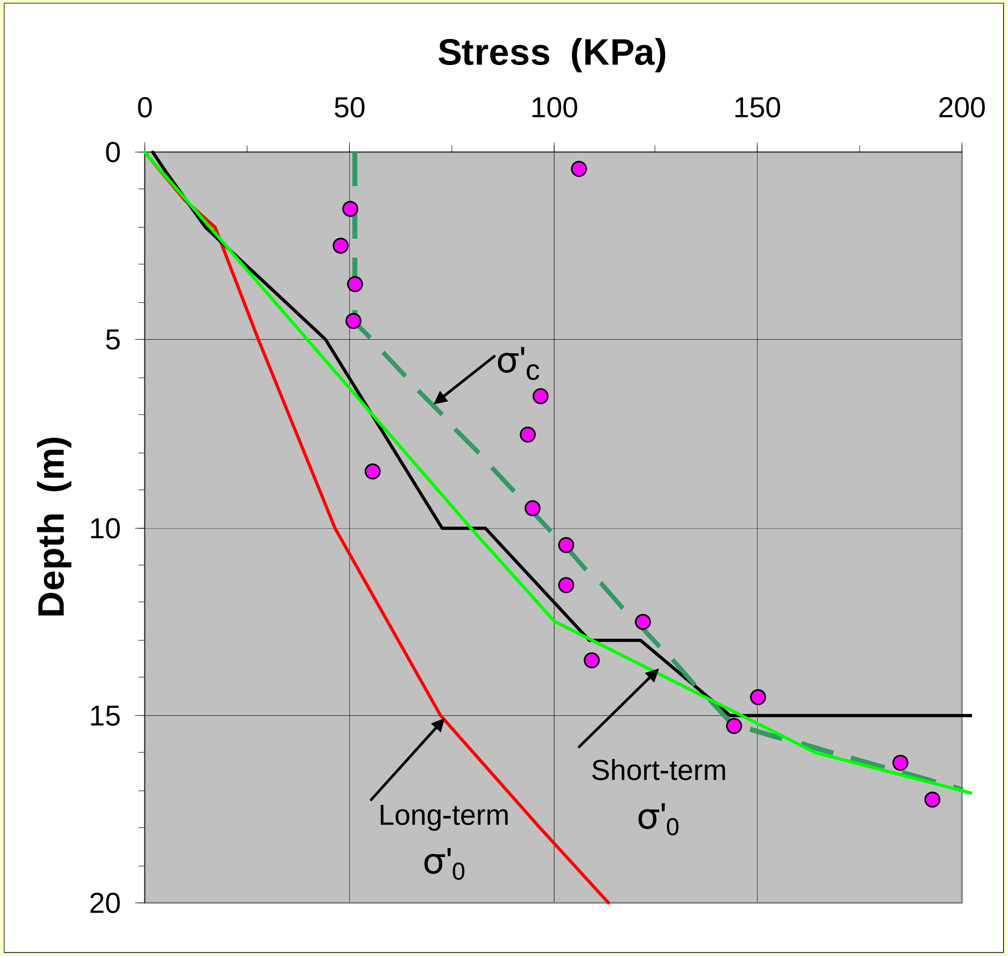
Subsidence at San Joaquin Valley, California



Devin Galloway and Francis S. Riley, U.S. Geological Survey

But back to Bangkok:

Stress Profile

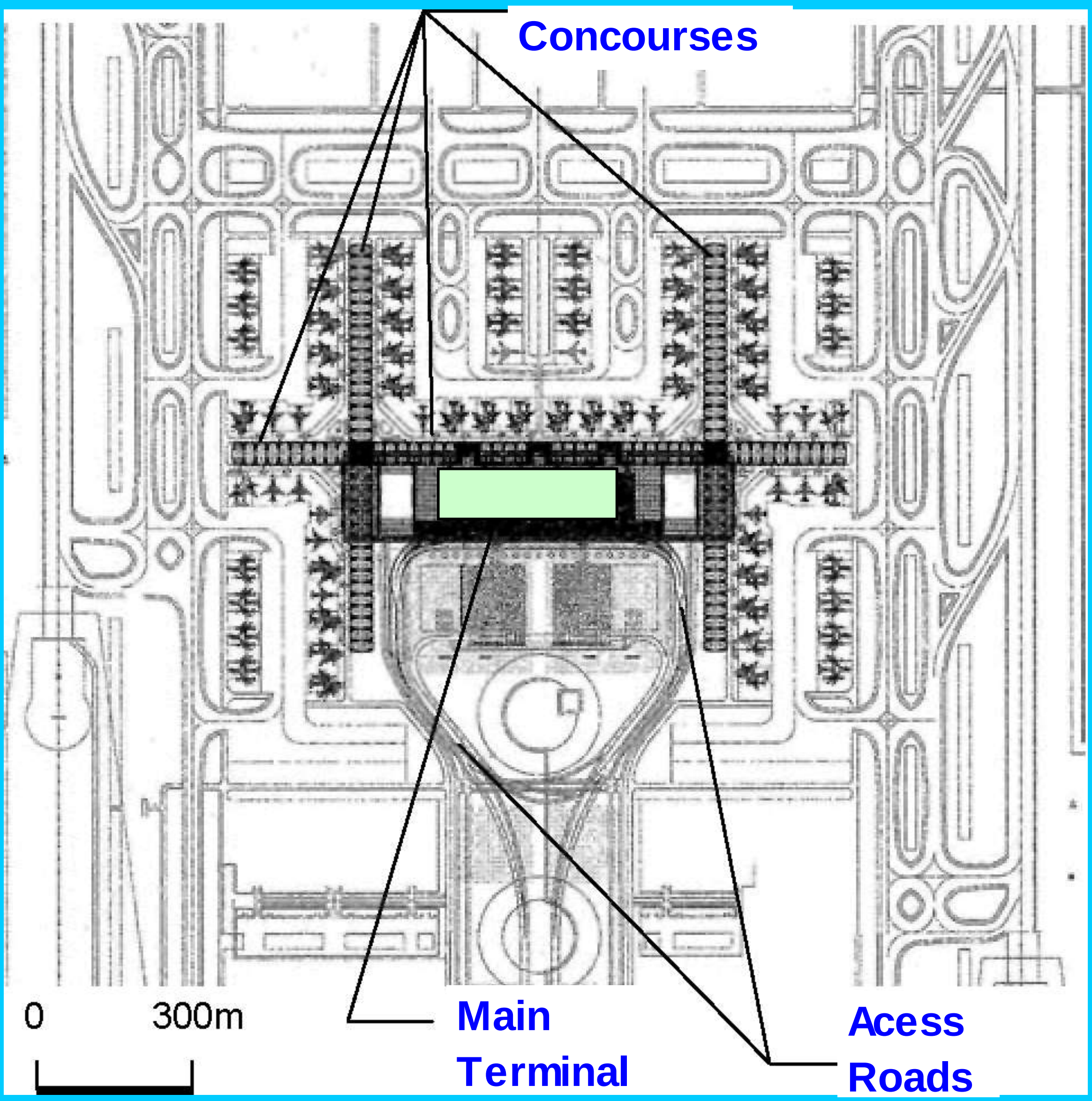


Circles are preconsolidation data points

Dashed green line shows distribution for design

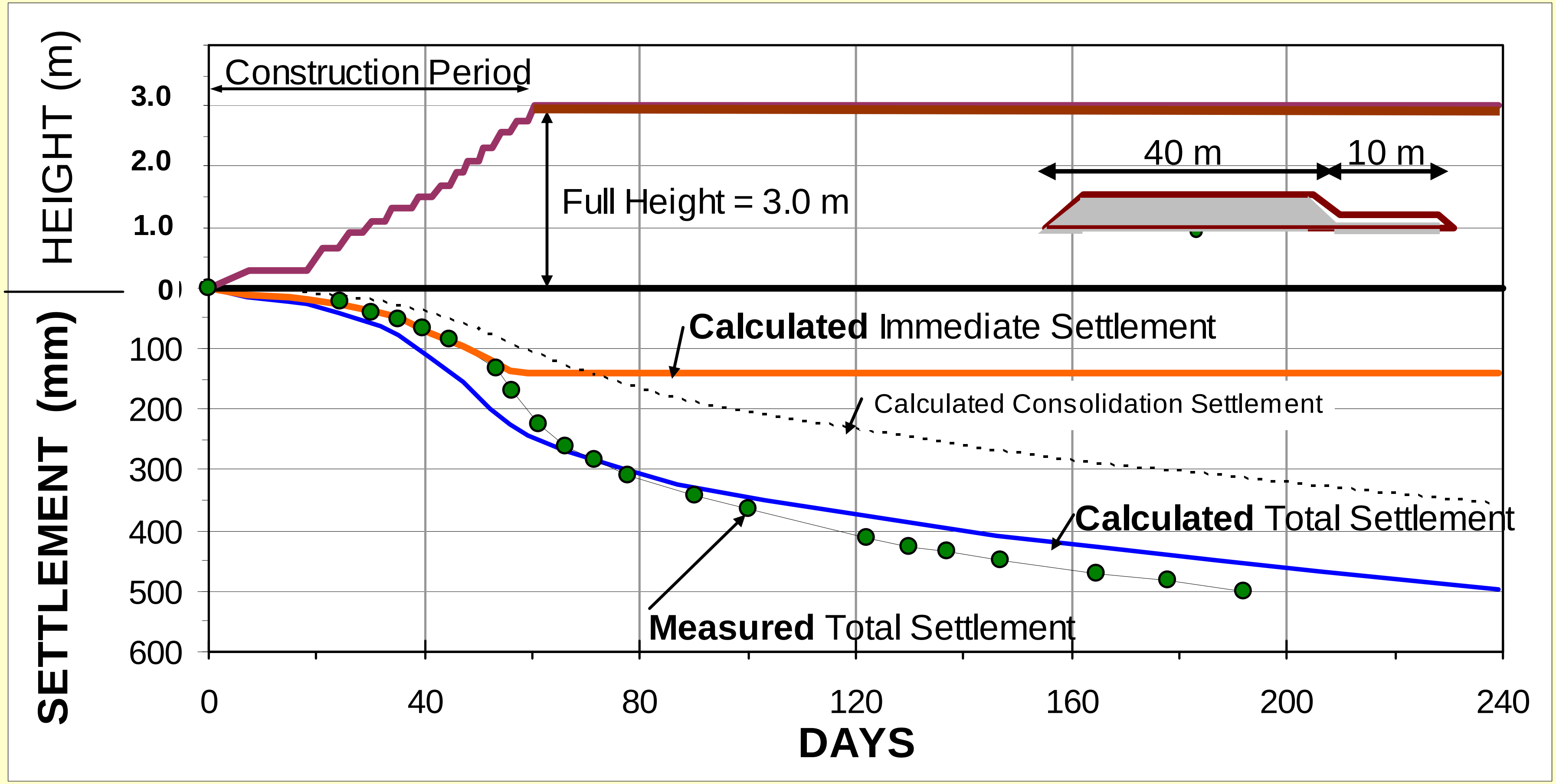
“Short-term” : Effective stress distribution at time of design

“Long-term” : Effective stress distribution after groundwater table is raised

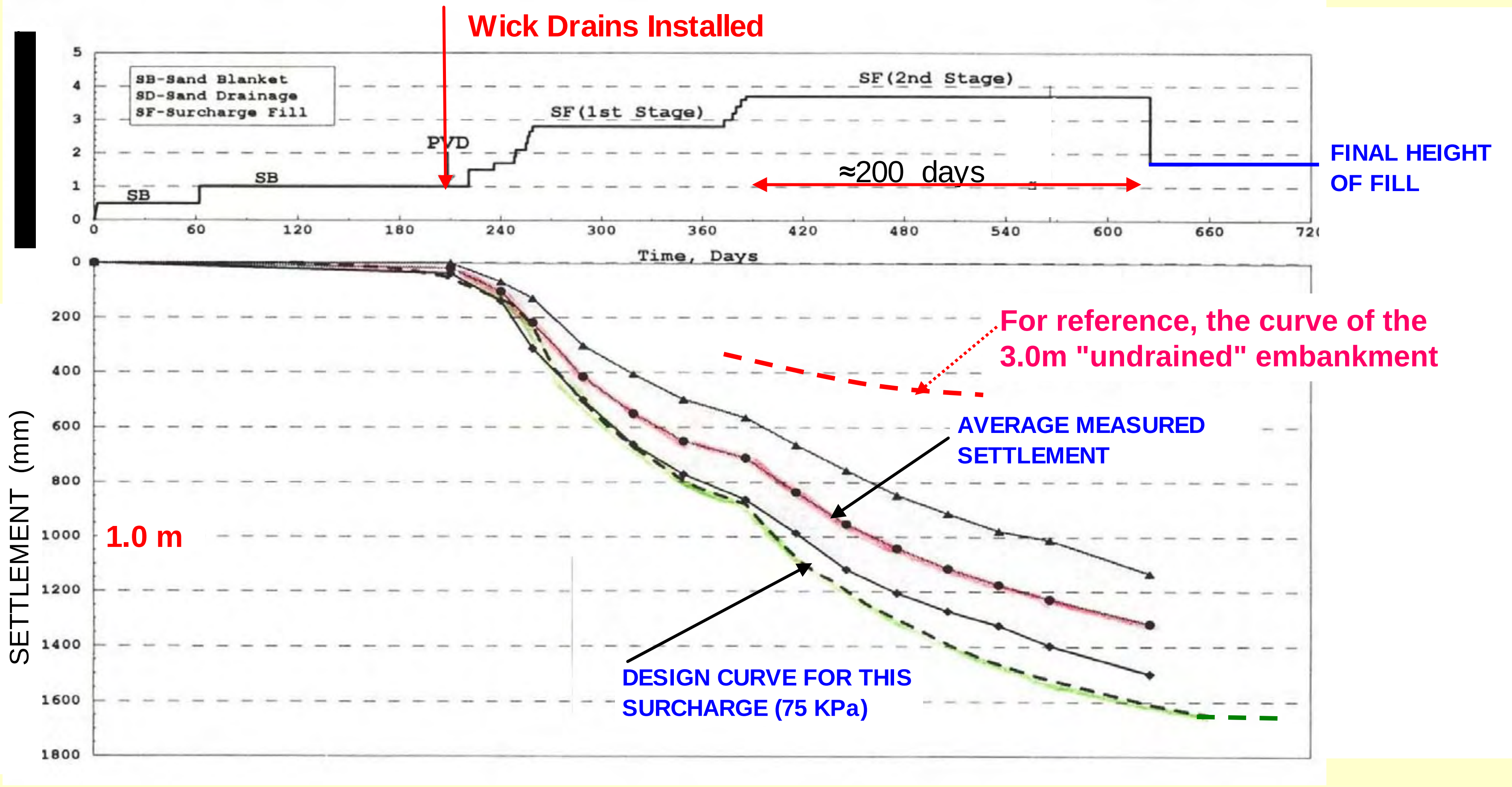

Concourses
0 300m
Main Terminal
Acess Roads

First a few words on the soil improvement work

Measured and Calculated Settlement at center line of for a 3.0 m Embankment during 200 days. **No drains**, i.e., incomplete consolidation.

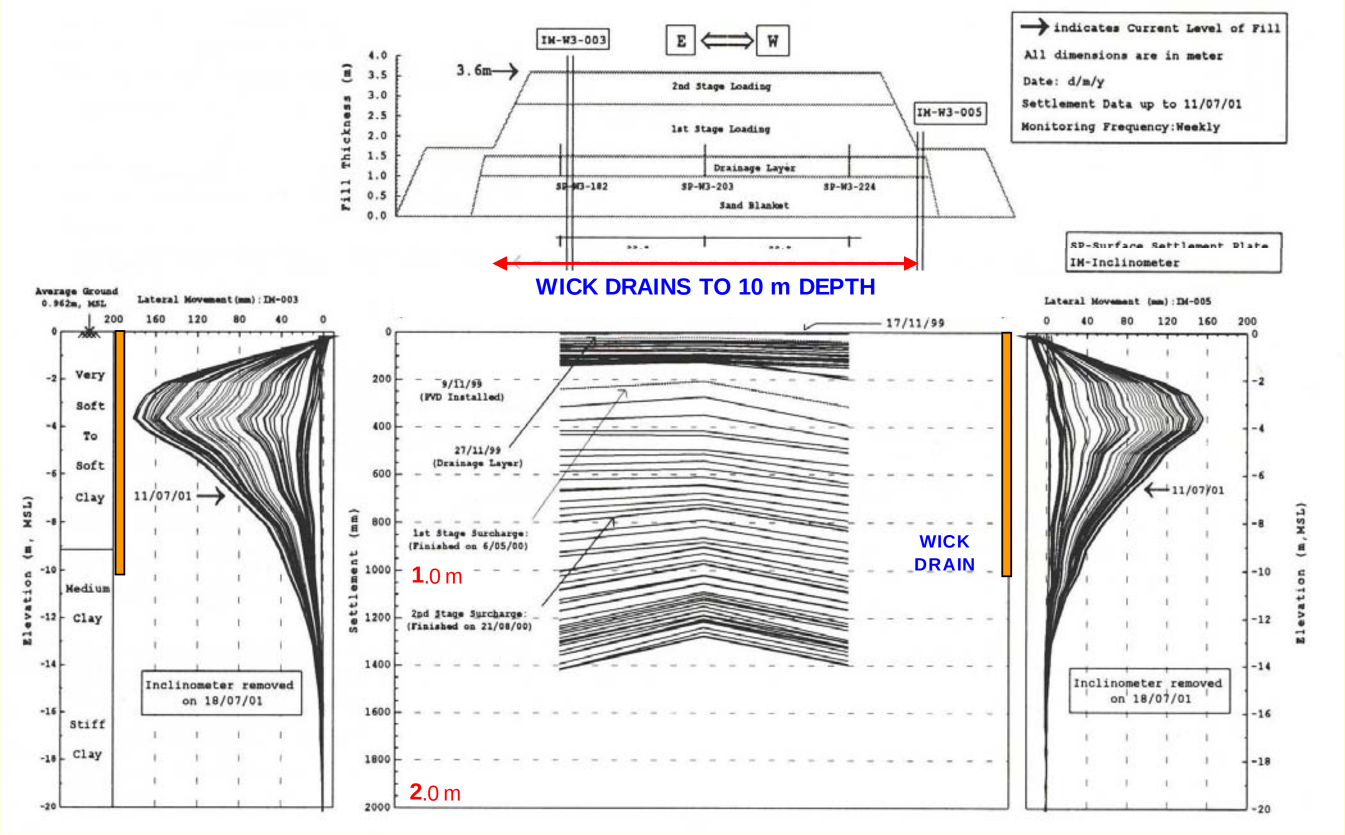


Settlement at center line of a 3.6m Embankment on **Wick Drains**



Settlement and Horizontal Movement for the 3.6 m Embankment

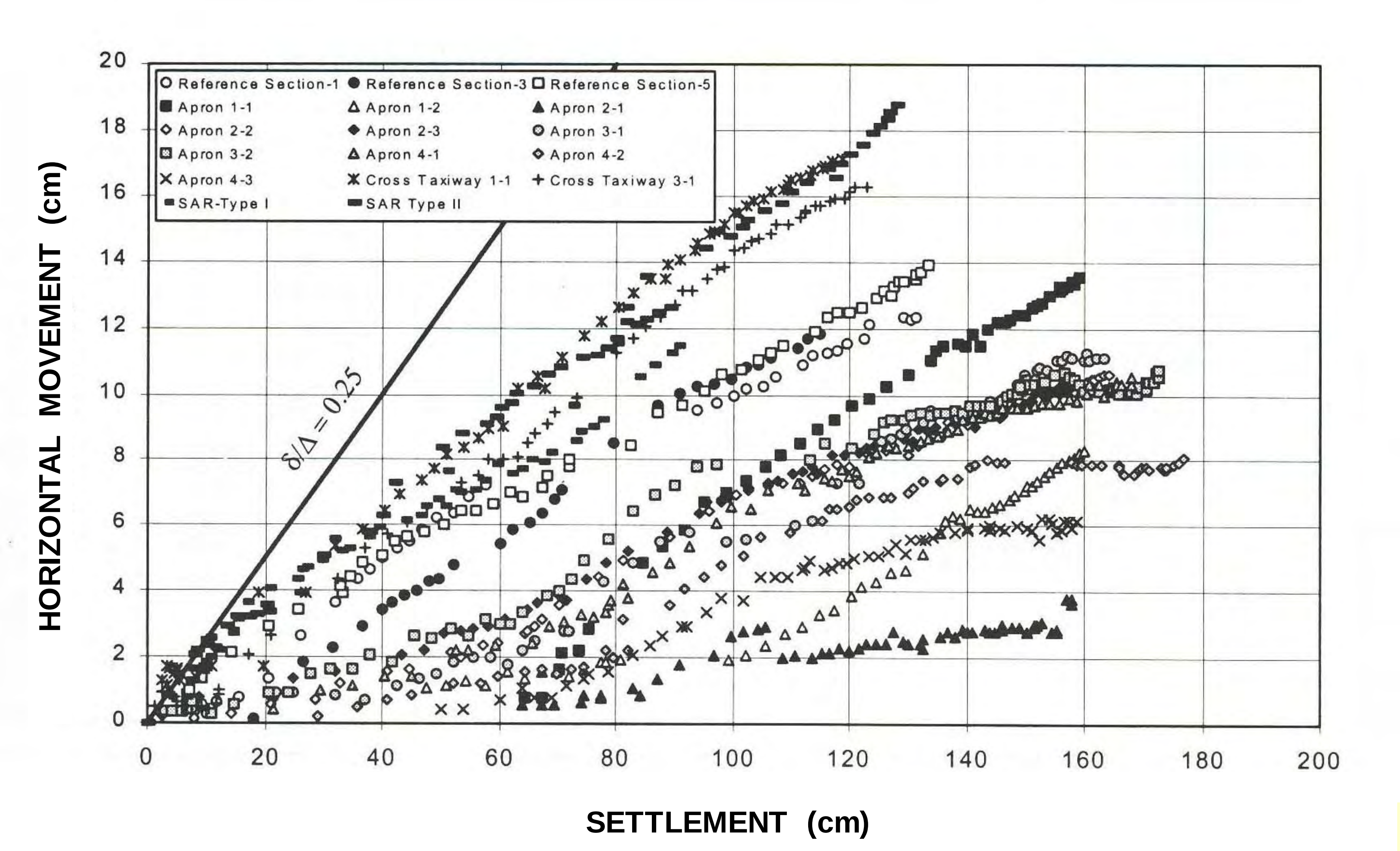
Settlement was monitored in center and at embankment sides and horizontal movement was monitored near sides of embankment



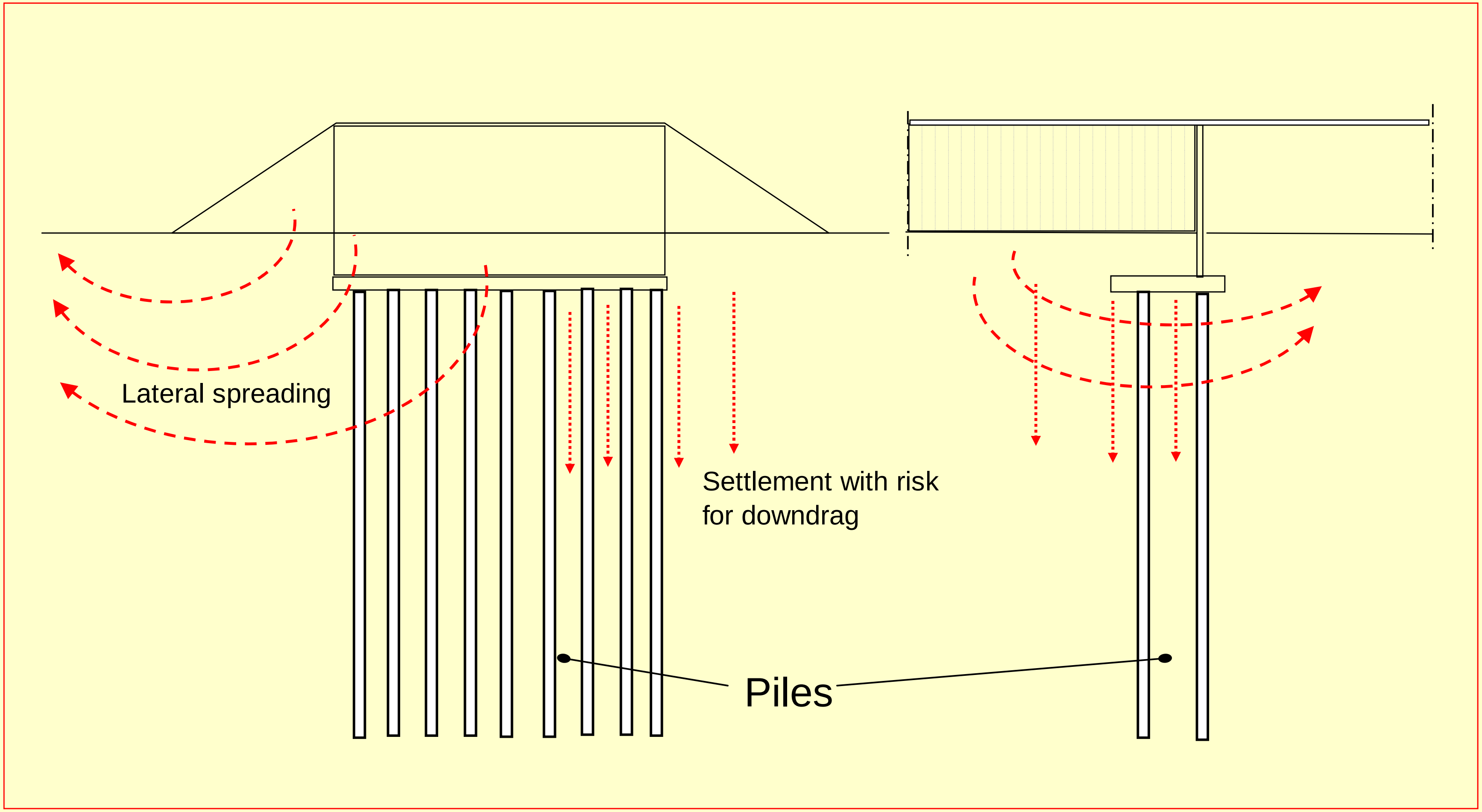
Time from start to end of surcharge placement = 9 months

Observation time after end of surcharge placement = 11 months

Horizontal Movement versus Settlement at Different Test Locations



The Problem

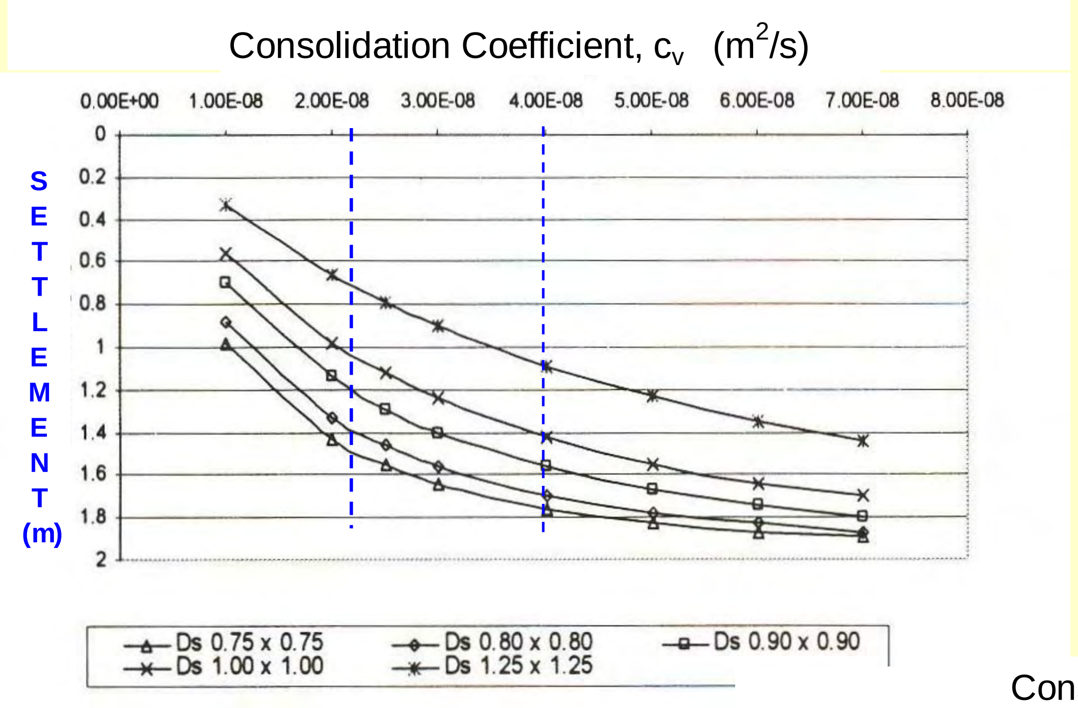


Lateral spreading
Settlement with risk
for downdrag
Piles


Ch. Ste-Catherine
St-Zotique
St-Polycarpe

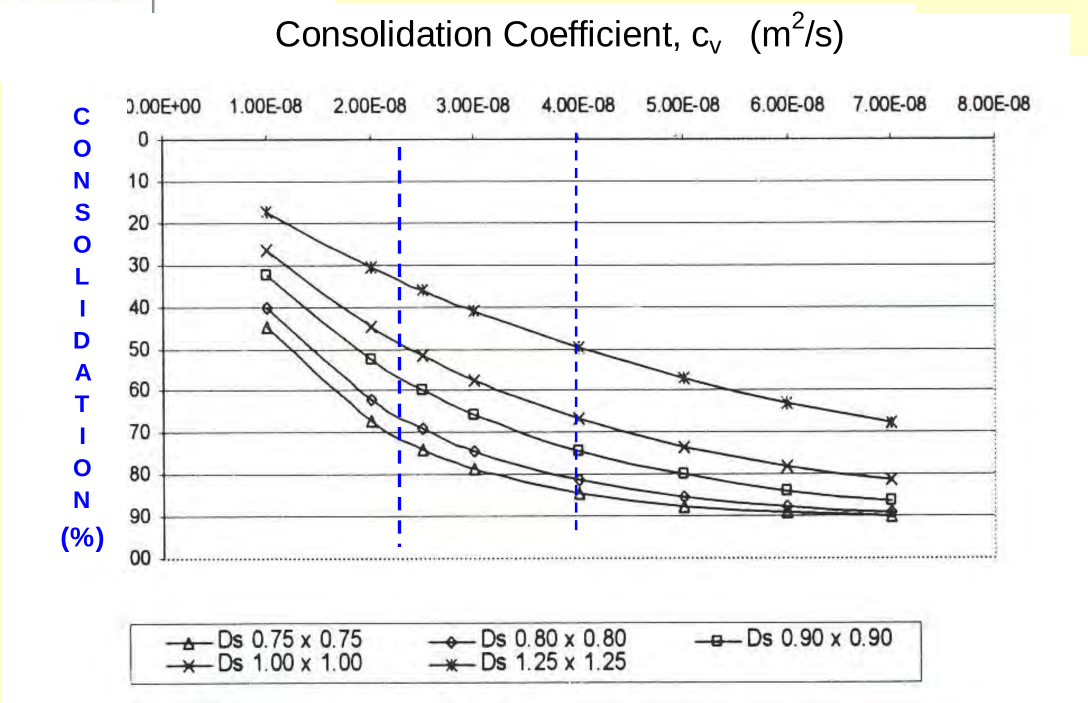
These photos of the bridge foundations illustrate a common problem affecting maintenance (**$$$!**), as well as, on occasions, being one compromising safety.


FOTO

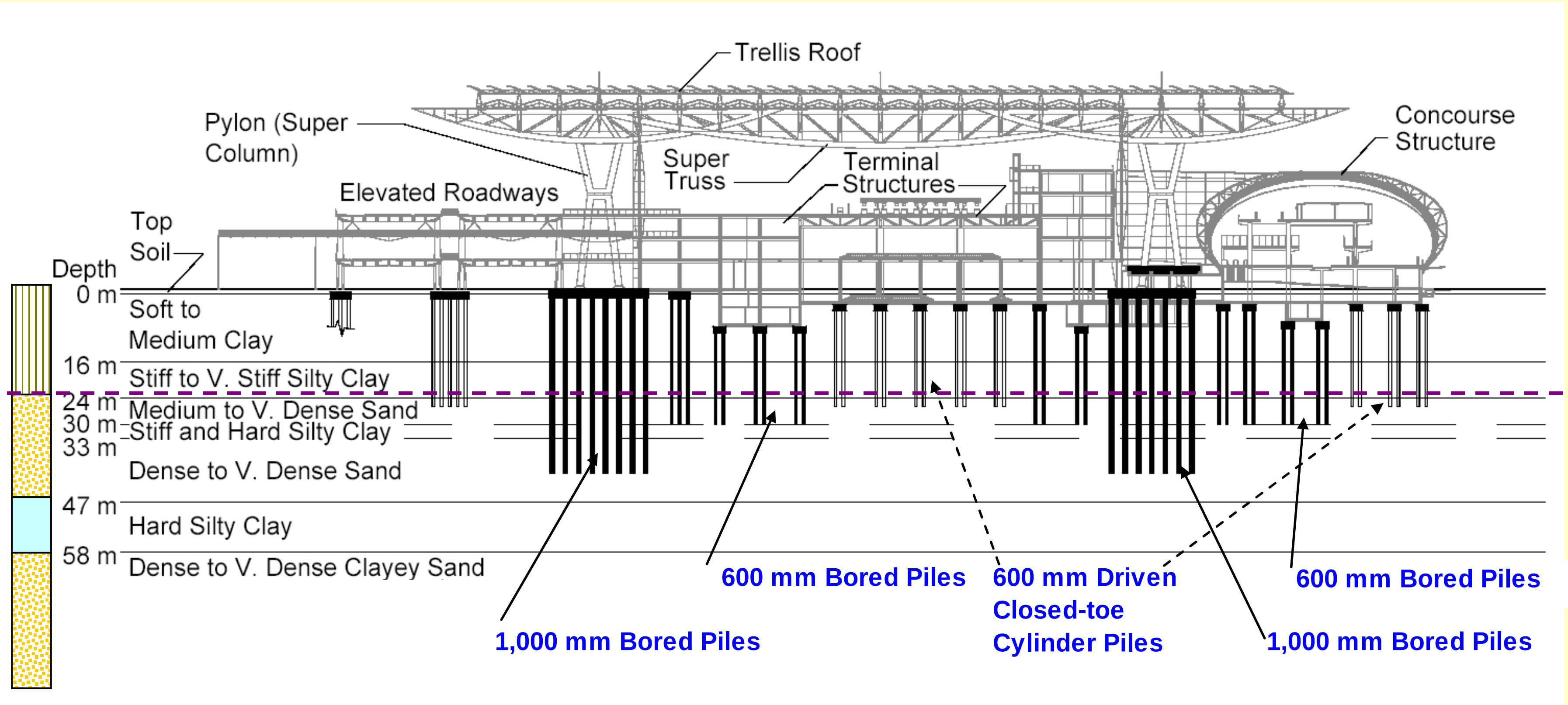



Settlement after <u>4 months</u> as a function of the Consolidation Coefficient and Drain Spacing

$$t = T_v \frac{H^2}{c_v}$$



Degree of Consolidation after 4 months as a function of the Consolidation Coefficient and Drain Spacing

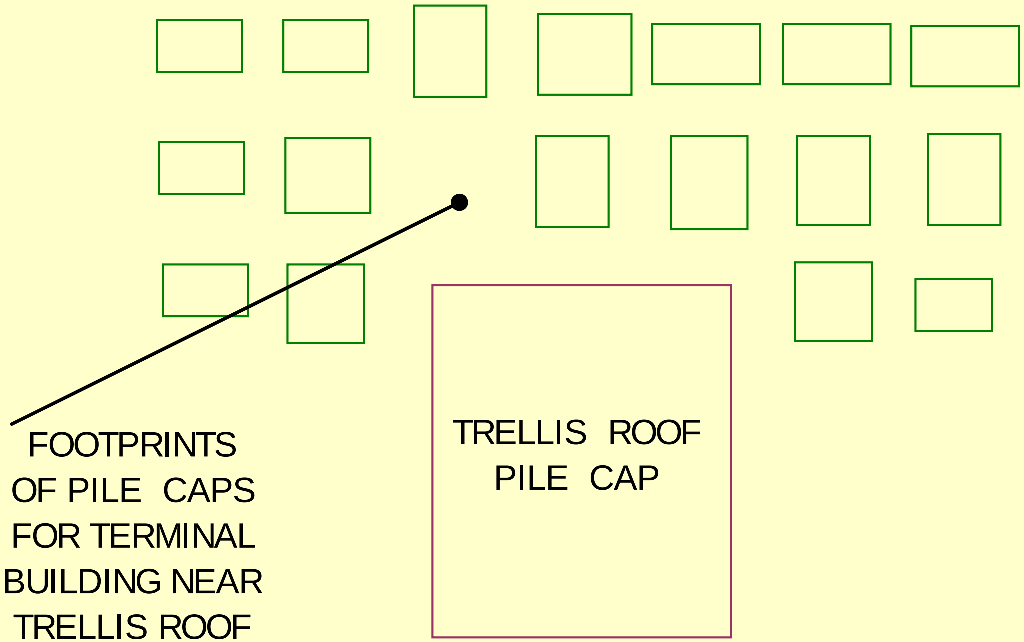


The clay is soft and normally consolidated with a modulus number smaller than 10.

All foundations — the trellis roof, terminal buildings, concourse, walkways, etc. — are placed on piles. The stress-bulbs from the various foundations will overlap each other's areas resulting in a complicated settlement analysis.

TYPICAL FOOTPRINT LAYOUT OF TRELLIS AND BUILDING FOUNDATIONS

Additional features, such as Embankments, Aprons, Concourse foundations, Area Fills, etc. adversely affect the piles and the piled foundations.



The stress interference between the foundations is significant and must be considered in the design analyses

Horizontal soil movement ("lateral spreading") toward piles can be critical

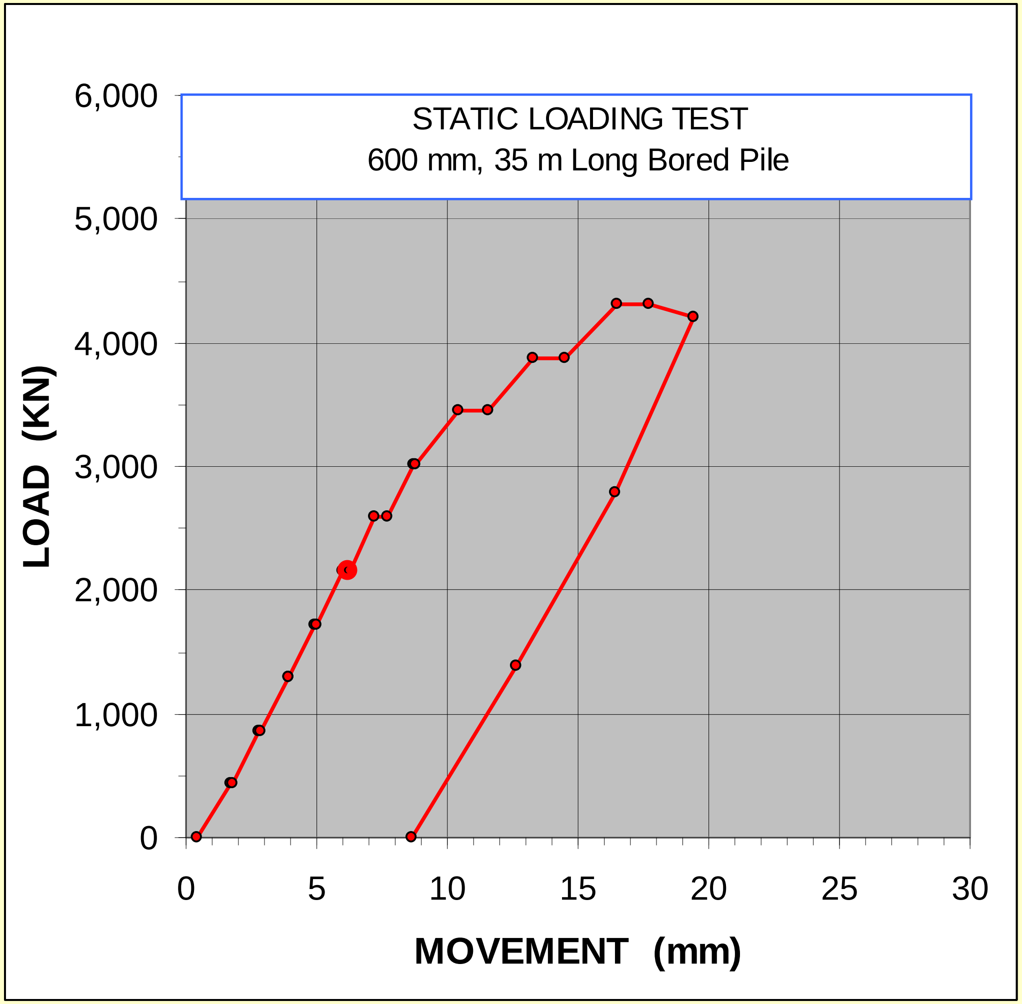
To minimize lateral spreading toward adjacent foundations, some embankments were "supported" on soil-cement columns. For others, vacuum surcharge was employed together with fill surcharge.

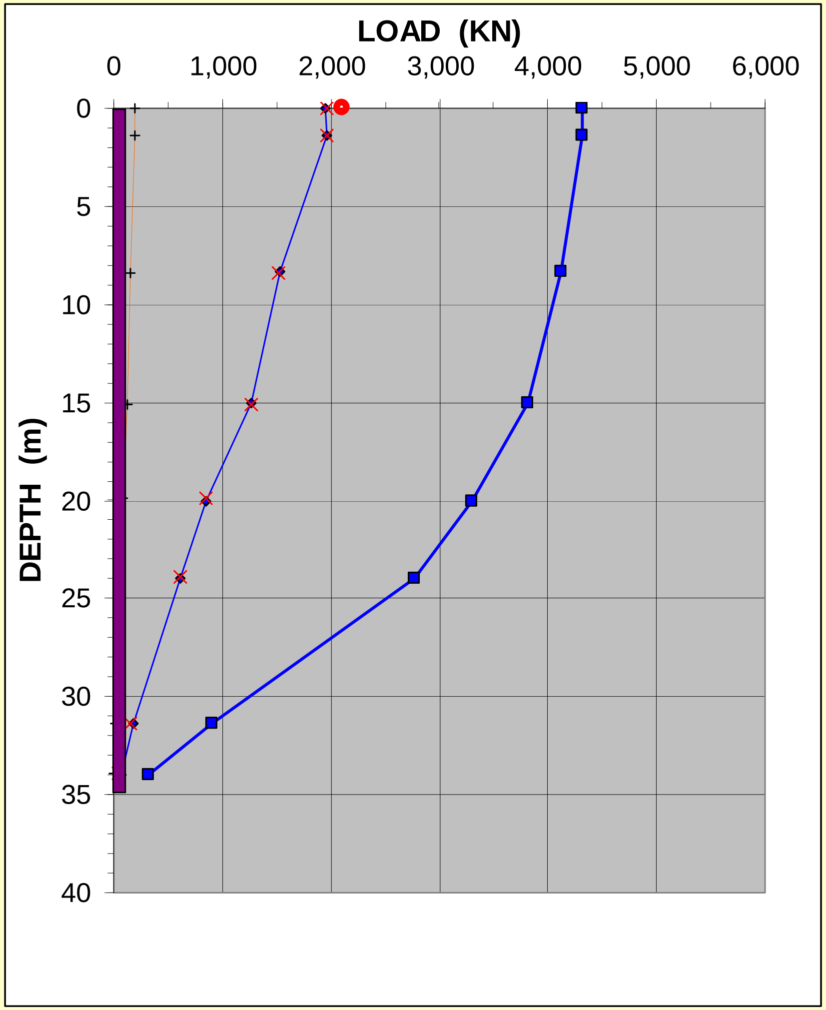
Vacuum surcharge will cause the perimeter soil to move inward. Combining vacuum and fill surcharge can minimize the horizontal soil movement.

Vacuum surcharge can theoretically reach a stress of 100 KPa, but in practice, the maximum stress is about 60 KPa, equivalent to a fill height of about 3 m.

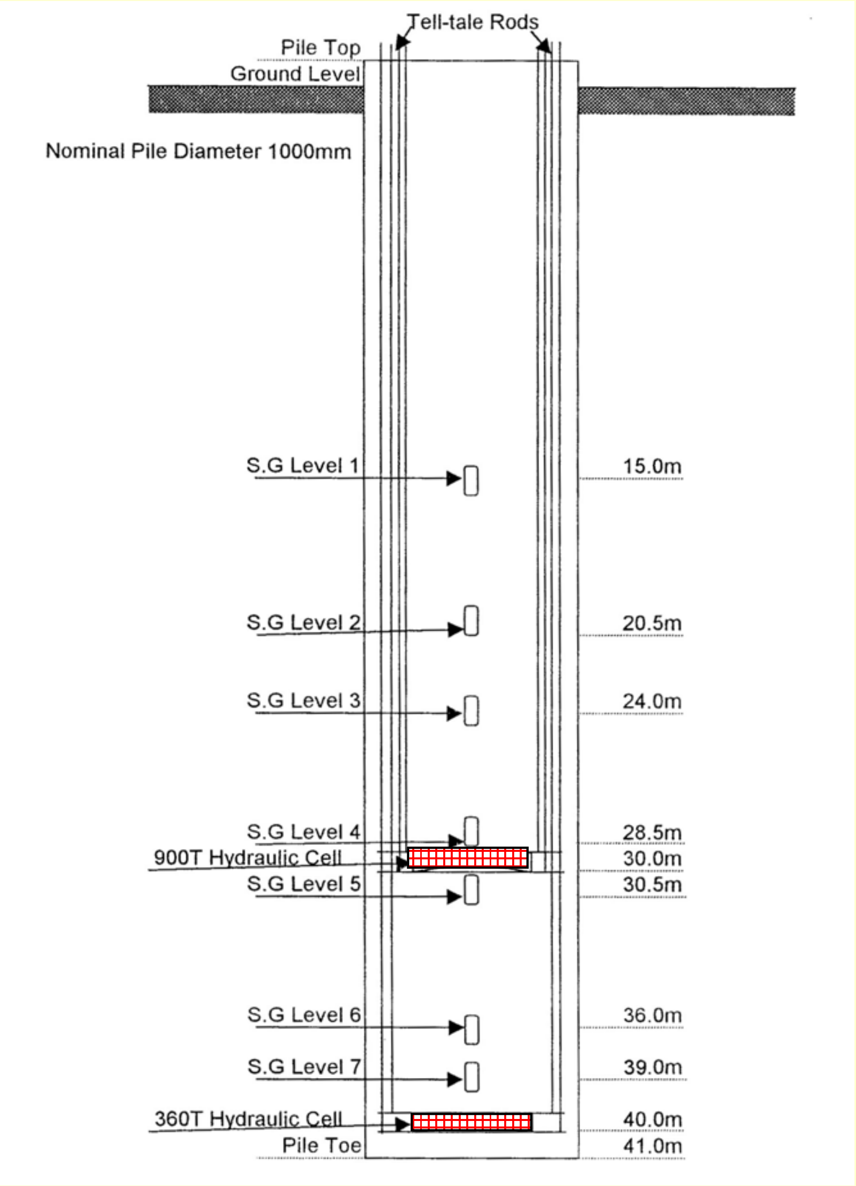
The final design employed a vacuum surcharge (considered to be effective at 60 KPa) combined with an about 3 m surcharge fill (= 56 KPa) and a 0.9 m c/c triangular drain spacing. The target time for 60 % consolidation was 4 months at which time the extra surcharge was removed to bring the degree of consolidation to about 85 %.


Suvarnabhumi Airport
Bangkok Thailand
ToThailand.com


STATIC LOADING TEST
600 mm, 35 m Long Bored Pile
LOAD (KN)
MOVEMENT (mm)
6,000
5,000
4,000
3,000
2,000
1,000
0
0
5
10
15
20
25
30


LOAD (KN)
DEPTH (m)
0
1,000
2,000
3,000
4,000
5,000
6,000
0
5
10
15
20
25
30
35
40

Bi-directional Static Loading Test, "O-cell" test — *1,000 mm* Pile



Stage 1
Lower Cell activated
Upper cell closed

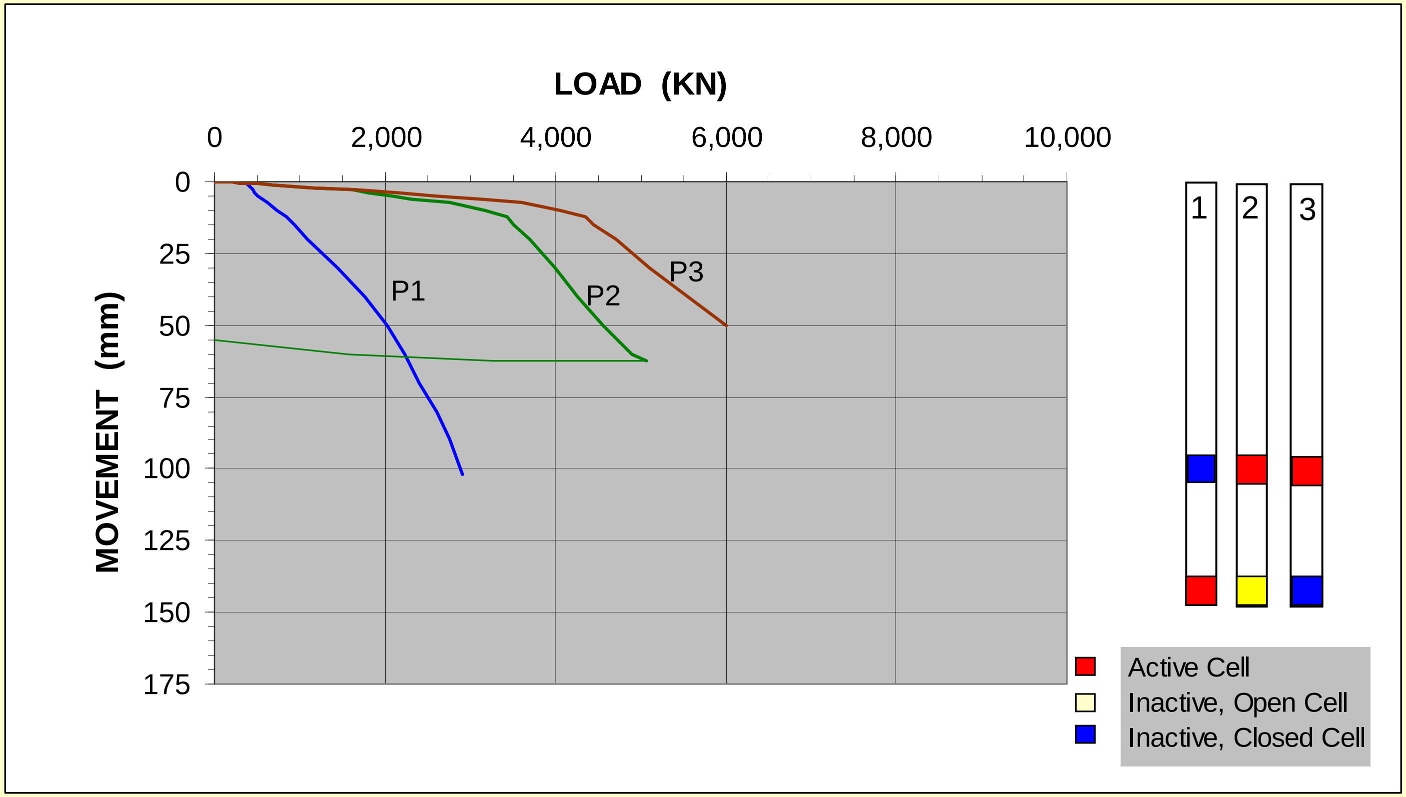
Stage 2
Lower Cell open
Upper Cell activated

Stage 2
Lower Cell closed
Upper Cell activated

Data from
Fox, I., Du, M. and Buttling,S. (2004)
Buttling, S. (2006)

O-Cell tests

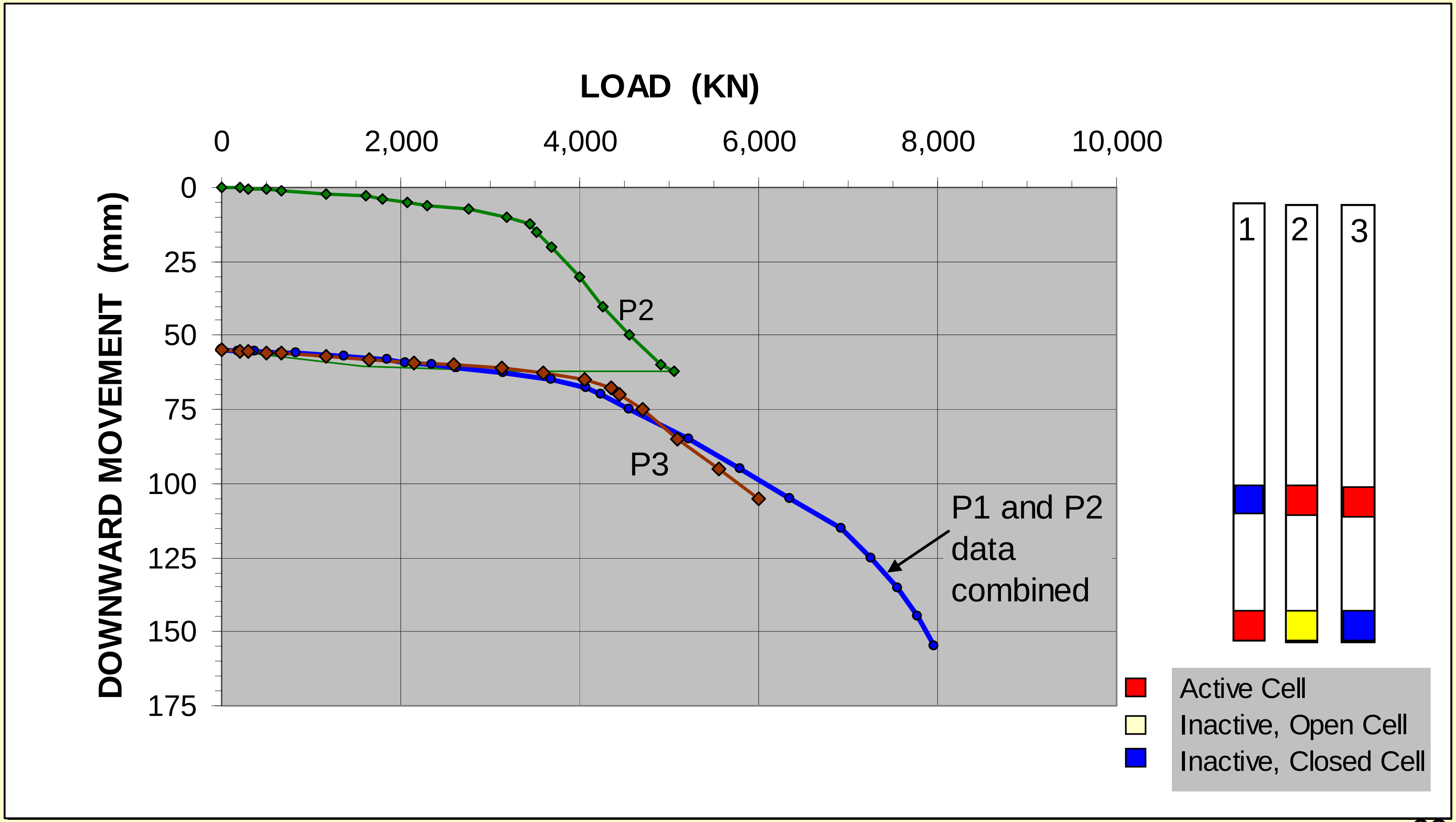
Downward movements during test phases 1, 2, and 3



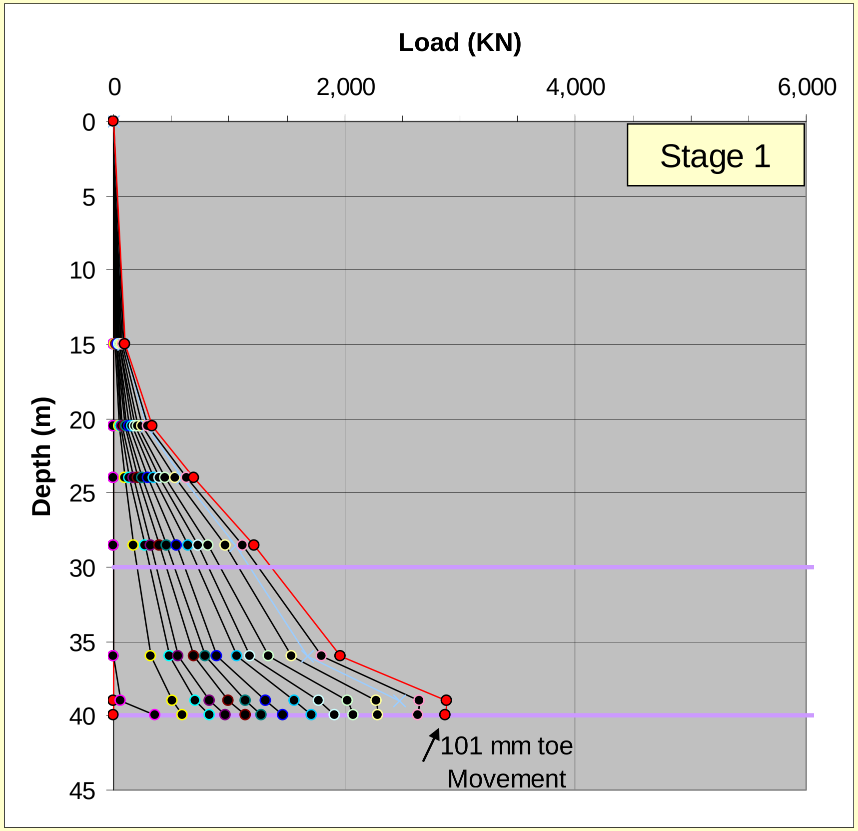
Concern was expressed that the toe resistance (Phase 1) was ≈3,000 KN and the shaft resistance for the lower segment was ≈5,000 KN (Phase 2), while in Phase 3 the combined shaft and toe resistances were only ≈6,000 KN. Should not the Phase 3 resistance be ≈8,000 KN rather than ≈6,000 KN (i.e., the sum of the values ≈5,000 KN and ≈3,000)?

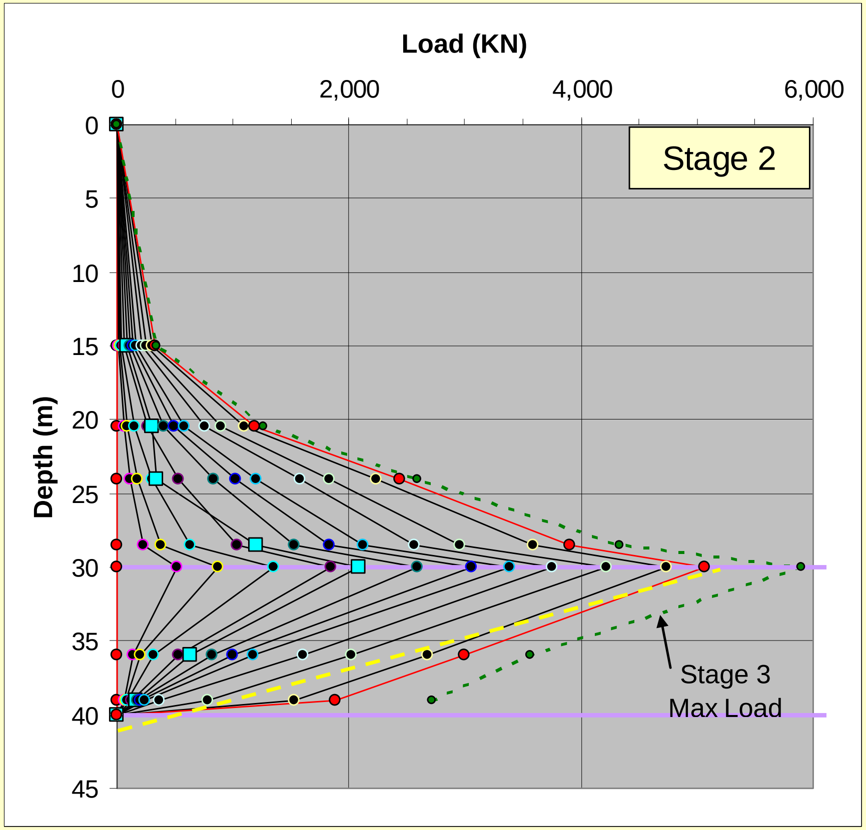
Downward toe movements

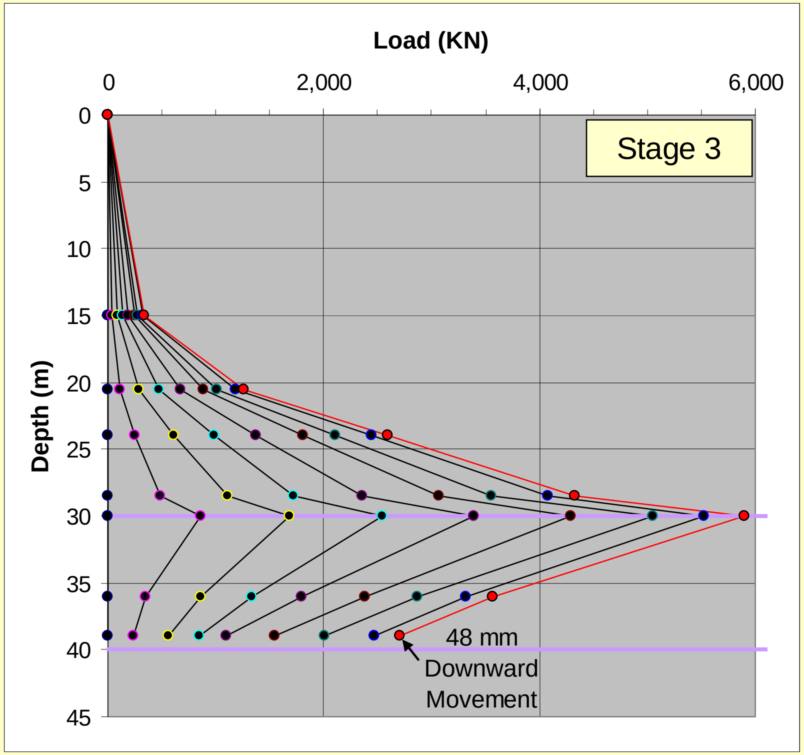
are best plotted per sequence of testing. Particularly when considering toe resistance, one must evaluate the load-movement response in comparing Phase 1 + Phase 2 to Phase 3 (i.e., P2 shaft below cell plus P1 toe).



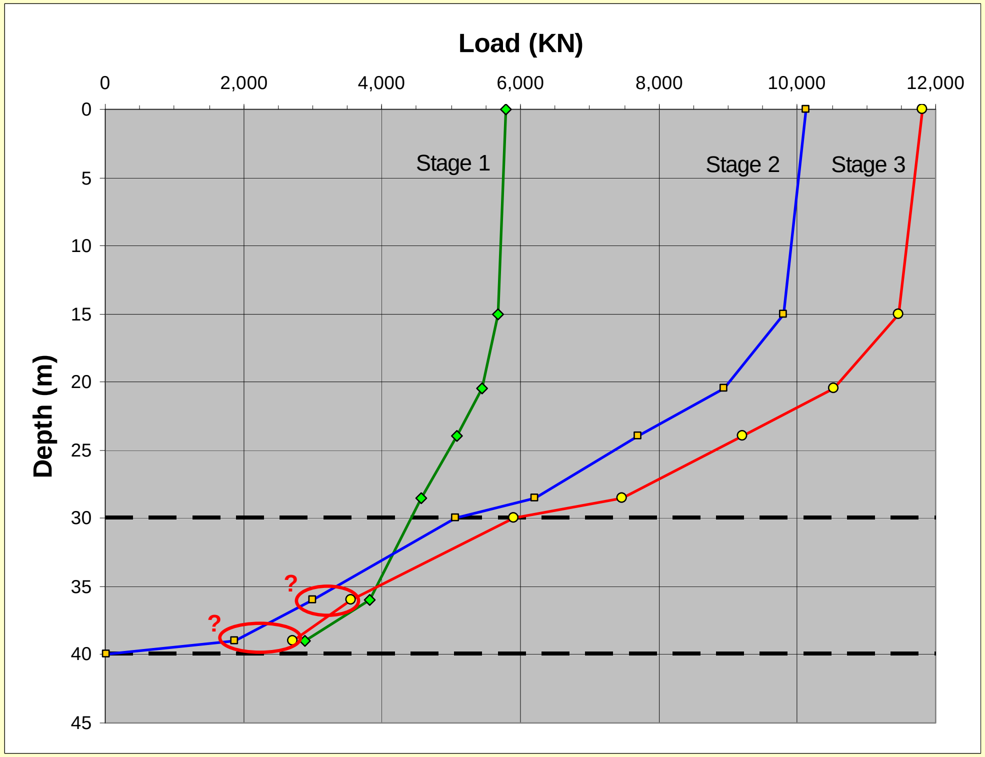
Load Distributions







Load (KN)
0
2,000
4,000
6,000
Stage 3
Depth (m)
0
5
10
15
20
25
30
35
40
45
48 mm
Downward
Movement



Load (KN)
0
2,000
4,000
6,000
8,000
10,000
12,000
Stage 1
Stage 2
Stage 3
Depth (m)
0
5
10
15
20
25
30
35
40
45
?
?

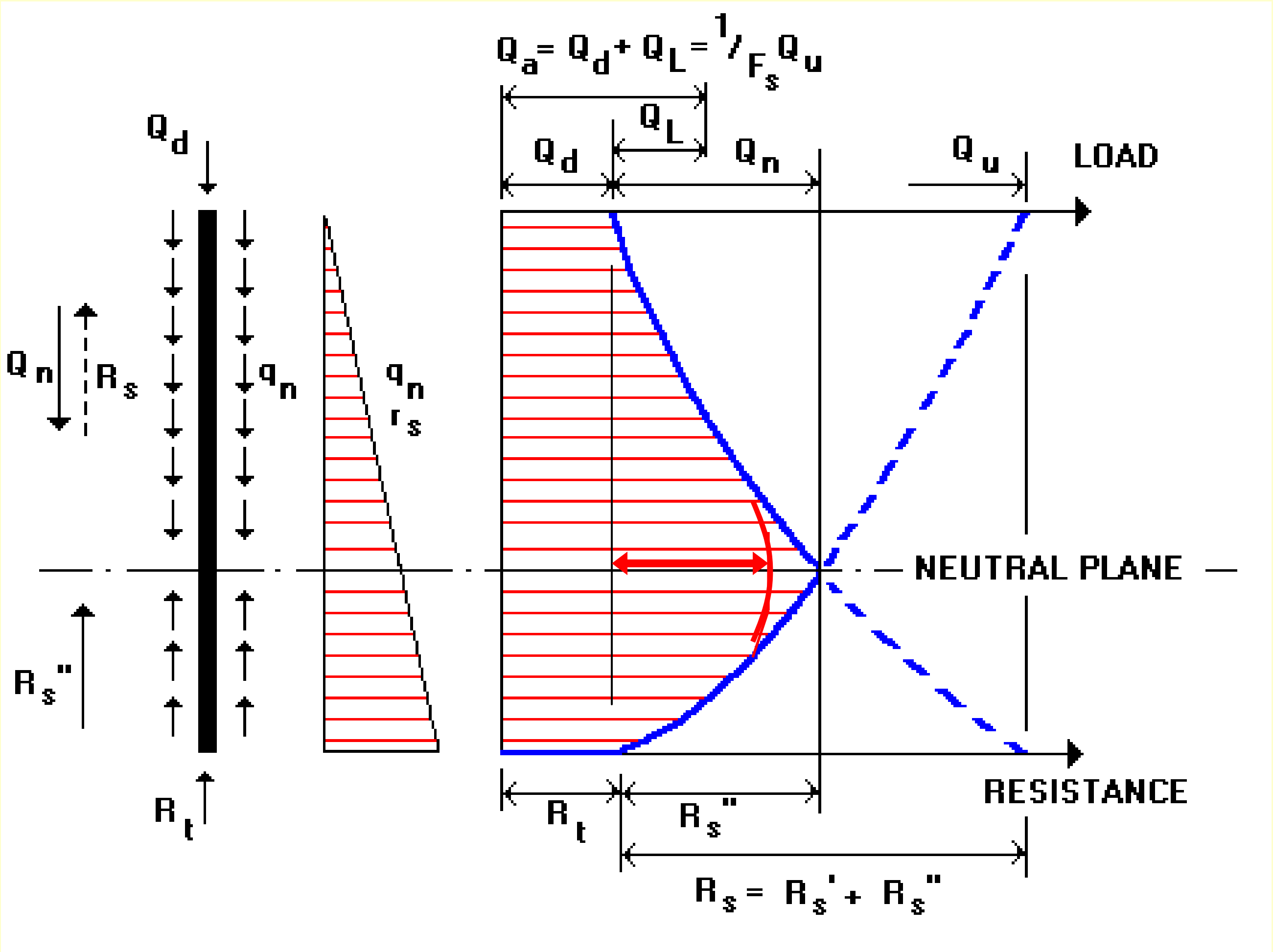
The test data — settlement data as well as pile test data — were applied to the design of the piled foundations employing the Unified Design Method.

The Unified Design Method is a three-step approach

1. The <u>dead plus live load</u> must be smaller than the pile **capacity** divided by an appropriate factor of safety. The drag load is not included when designing against the bearing capacity. [*The capacity of the pile toe should be defined from a movement criterion*].

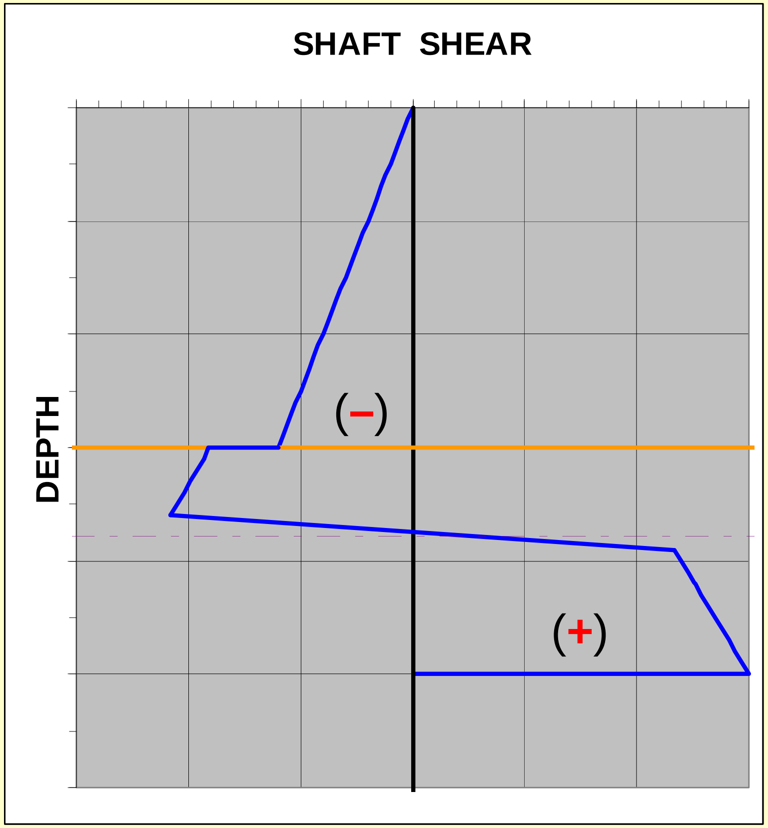
2. The <u>dead load plus the drag load</u> must be smaller than the **structural strength** divided with a appropriate factor of safety. [The live load must not be included because live load and drag load cannot coexist].

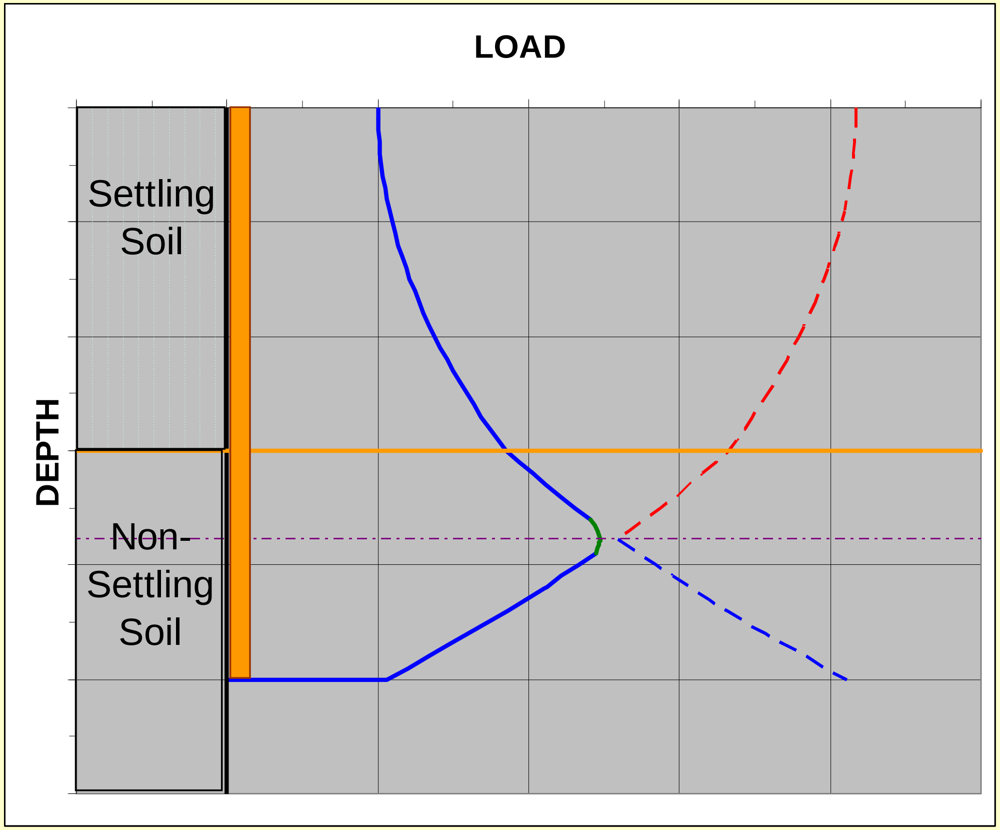
3. The **settlement** of the pile (pile group) must be smaller than a limiting value. The live load and drag load are not included in this analysis. [*The value(s) of acceptable settlement often governs a piled foundation design*].



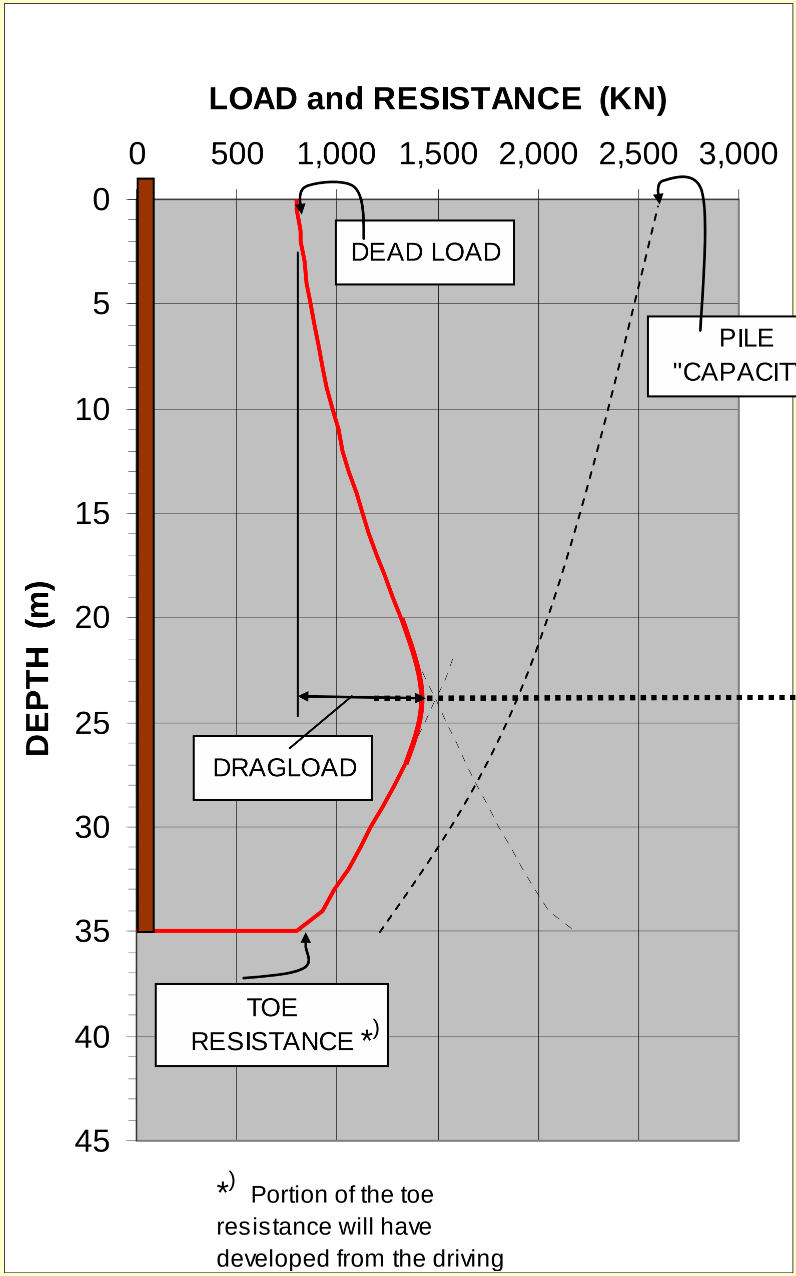
Construing the Neutral Plane and Determining the Allowable Load

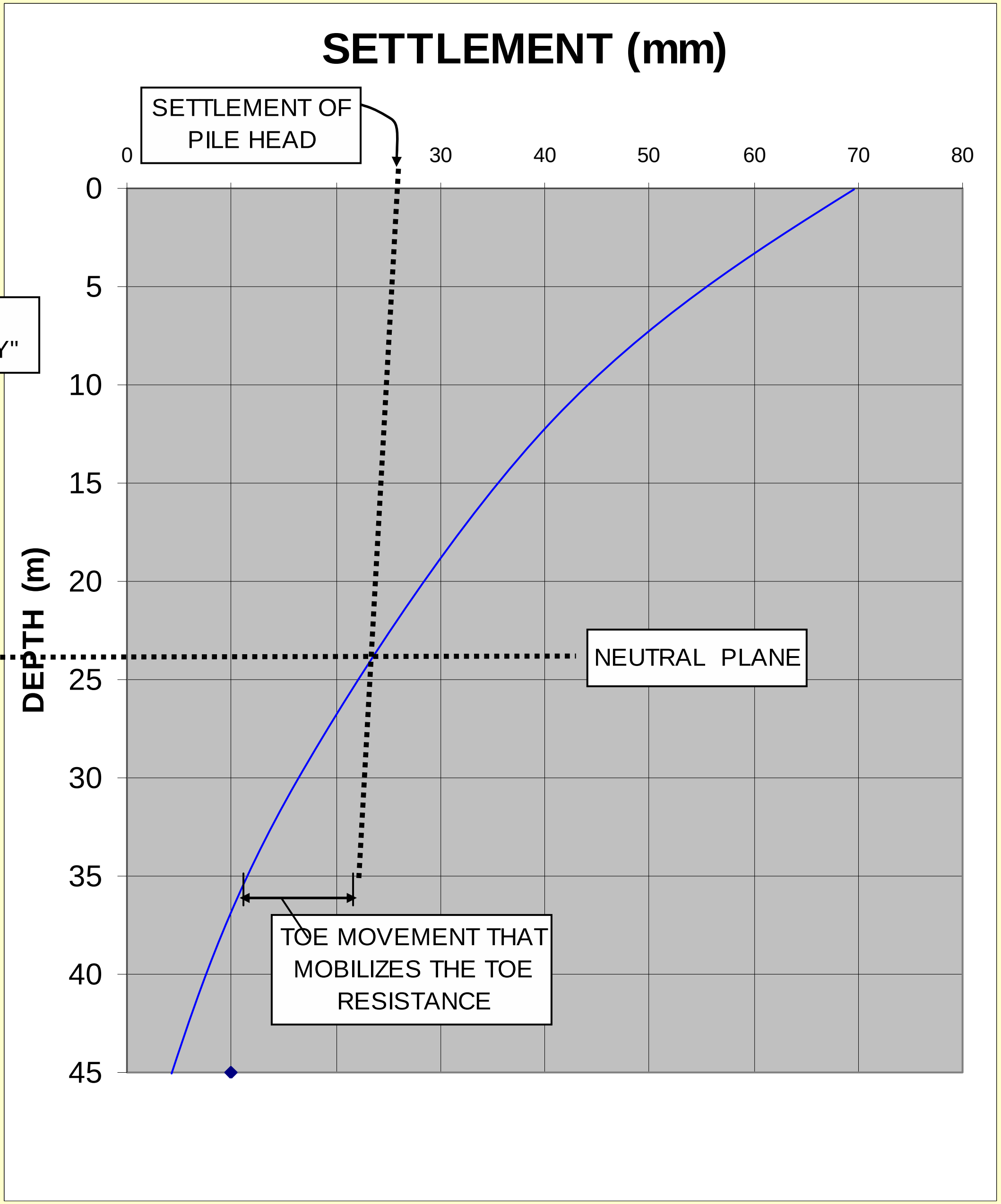
A repeat: Distribution of unit shaft shear and of load and resistance



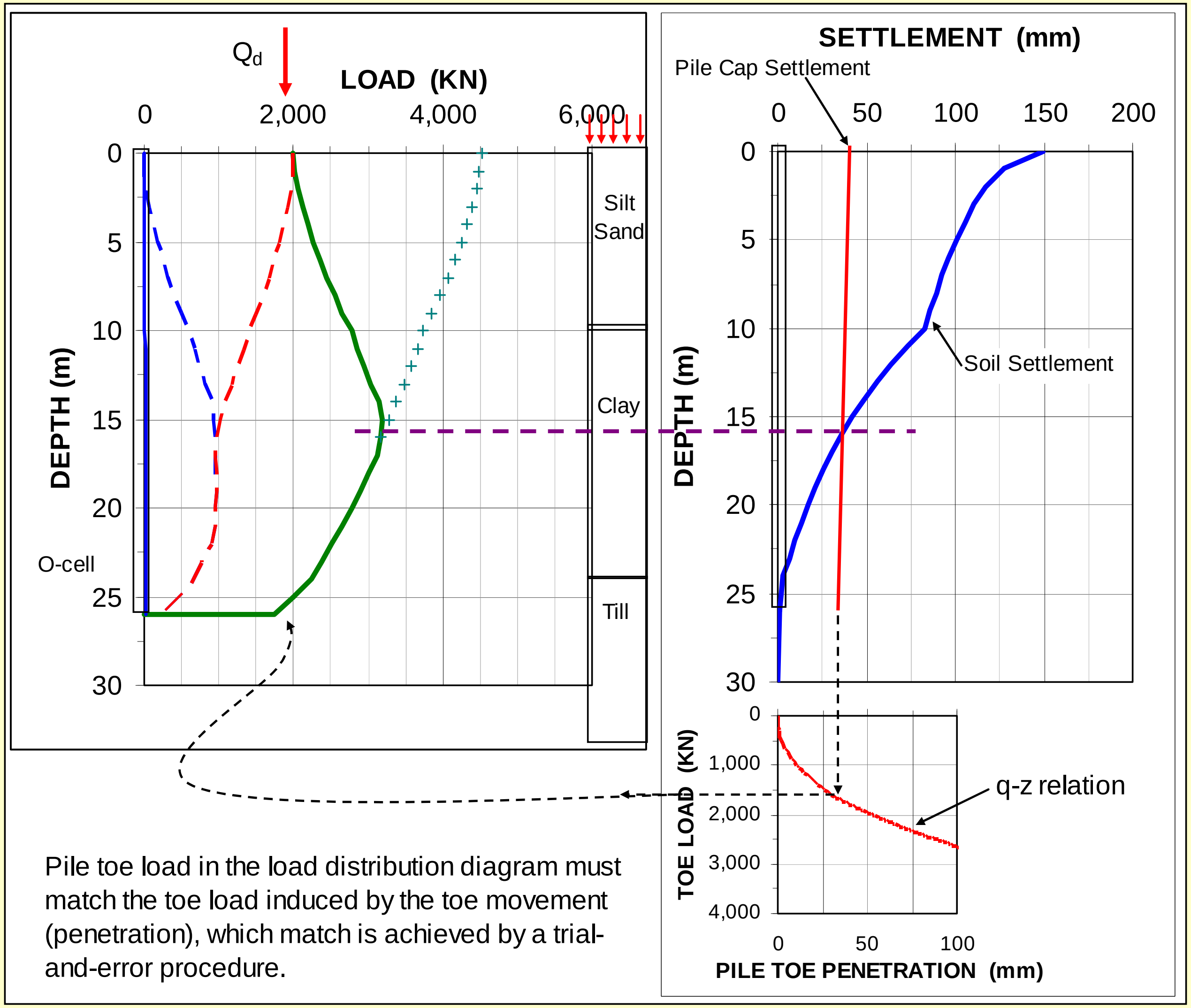


The Unified Method (typical example)





Force and settlement (downdrag) interactive design.
The unified pile design for capacity, drag load, settlement, and downdrag



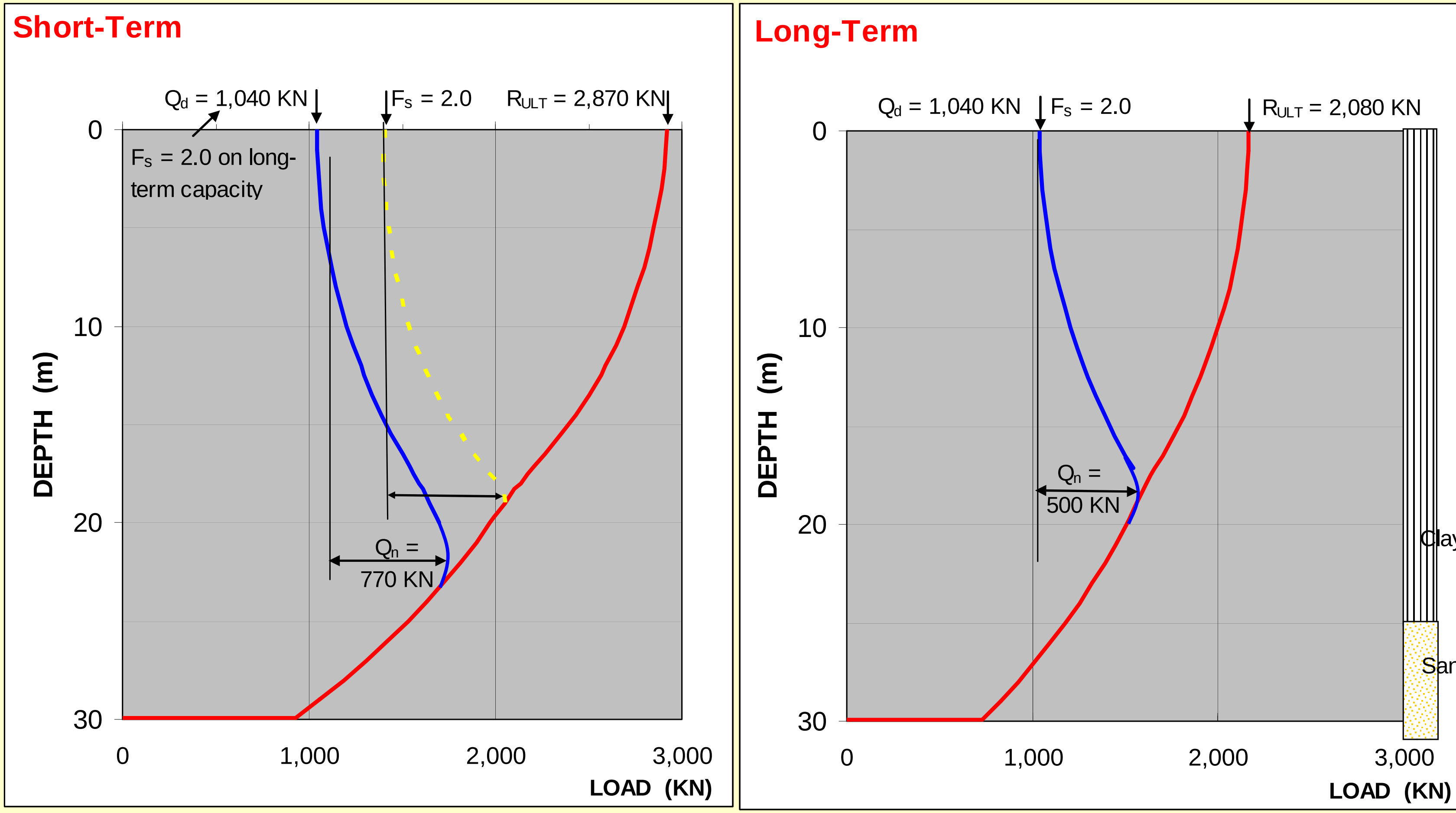
Pile toe load in the load distribution diagram must match the toe load induced by the toe movement (penetration), which match is achieved by a trial-and-error procedure.

Applied to the Bangkok Airport case

Several static loading tests on instrumented piles were performed to establish the load-transfer conditions at the site at the time of the testing, i.e., short-term conditions. Effective stress analysis of the test results for the current pore pressures established the coefficients applicable to the long-term conditions after water tables had stabilized.

A total of **25,400+** piles were installed.

Example of calculated resistance distribution for 600 mm diameter bored pile installed to a 30 m embedment depth.



The extensive testing and the conservative assumption on future pore pressures allowed an F_s of 2.0. The structural strength of the pile is more than adequate for the load at the neutral plane: $Q_d + Q_n \approx 1{,}500$ KN.

The settlements for the piled foundations were calculated to:

	Construction	Long-term	Total
Trellis Roof Pylons	20 mm	90 mm	**110 mm**
Terminal Building	30	15	**45**
Concourse	35	20	**55**

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Contour lines of settlement of ground surface and pile caps near a trellis roof pile cap