BASICS OF DESIGN OF PILED FOUNDATIONS

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Case Histories of Results from Instrumented Piles — and Soil

Brisbane November 28, 2008

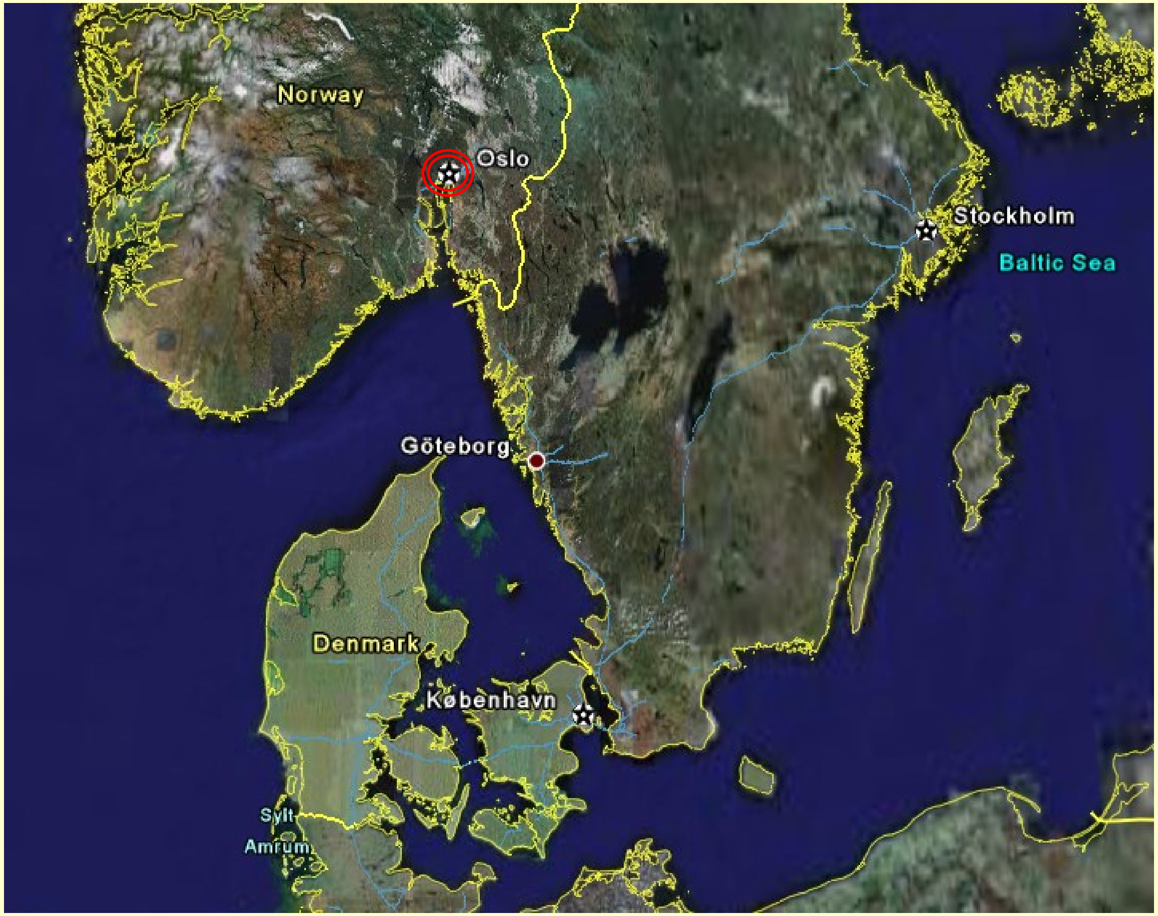

HORIZONTAL LOADS ON PILES
X ICSMFE
X ICSMFE
X ICSMFE
X ICSMFE
THE NETHERLANDS COMMEMORATIVE VOLUME
PROCEEDINGS OF THE 5th EUROPEAN CONFERENCE ON SOIL MECHANICS
PROCEEDINGS OF THE 5th EUROPEAN CONFERENCE ON SOIL MECHANICS
GEOTECH - YEAR 2000... Developments in Geotechnical Engineering
SOUVENIR VOLUME
GEOTECH - YEAR 2000... Developments in Geotechnical Engineering
Geotechnical Site Characterization
Geotechnical Site Characterization
Application of Stress Wave Theory to Piles
APPLICATION OF STRESS-WAVE THEORY TO PILES
Third International Conference
STATNAMIC Loading Test
Soil Properties
QUALITY in the CONSTRUCTED PROJECT
ASCE
GEOTECHNICAL ENGINEERING
GÉOTECHNIQUE

- Design of piles requires understanding of how load is transferred from pile to soil and, less obvious but equally important, from soil to pile.

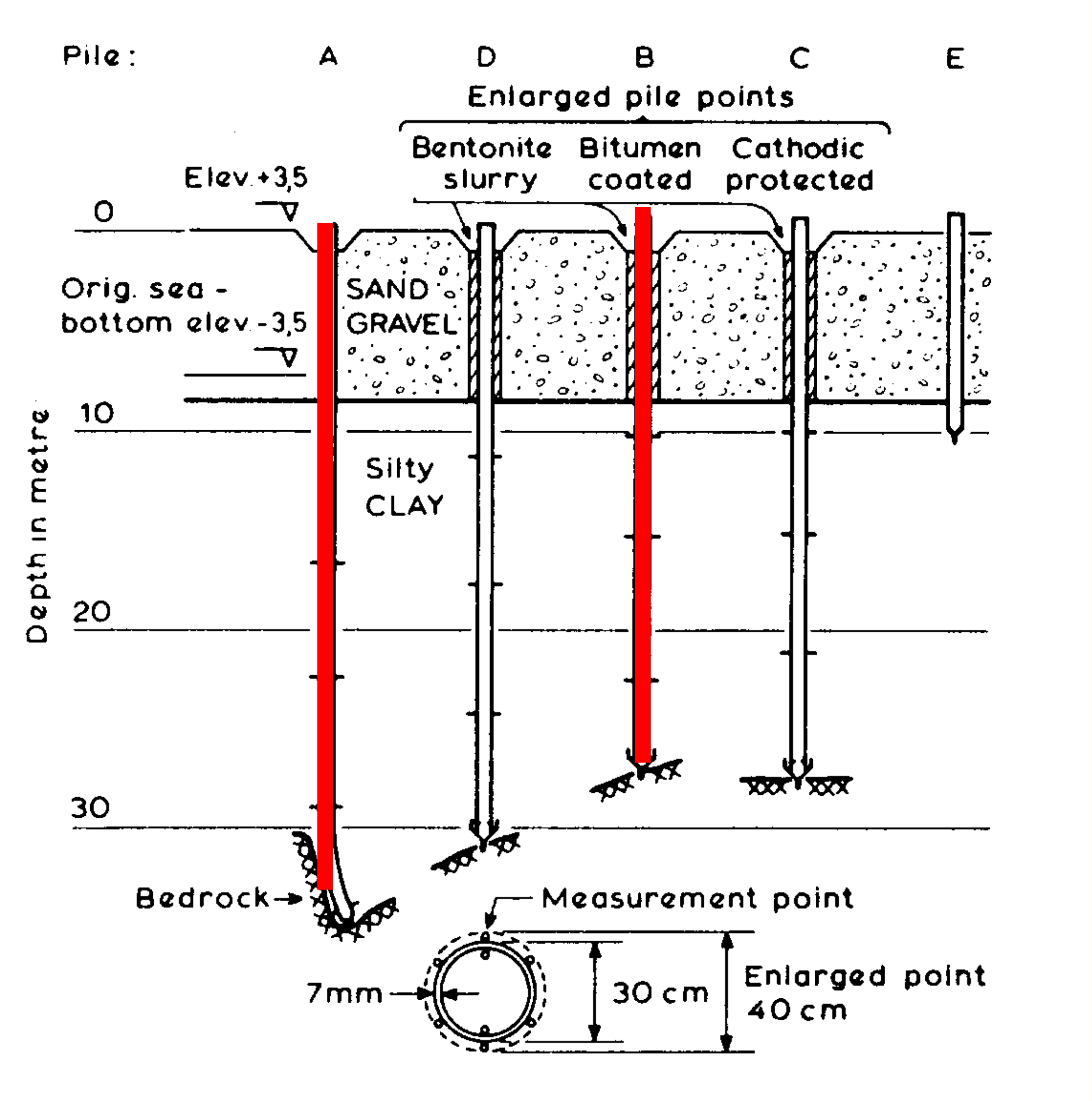
- The current state-of-the-art has evolved from a few ground-breaking case histories that appeared in the late 1960s and early 1970s.

CASE #1

Bjerrum et. al., (1965; 1969) presented case histories of 300-mm diameter steel piles driven through **marine clay** deposited directly on bedrock in Oslo Harbor, Norway.

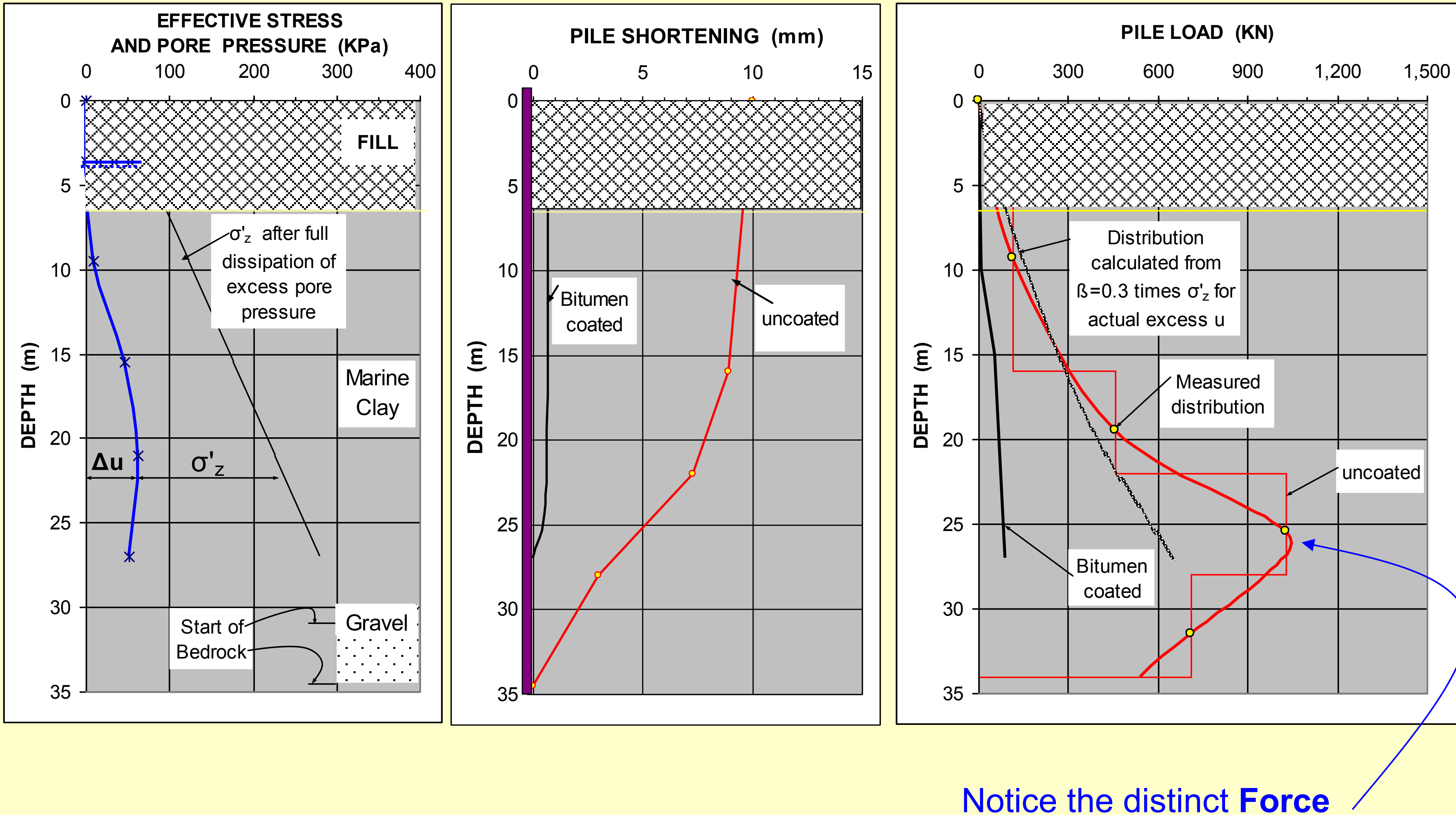


Heröya



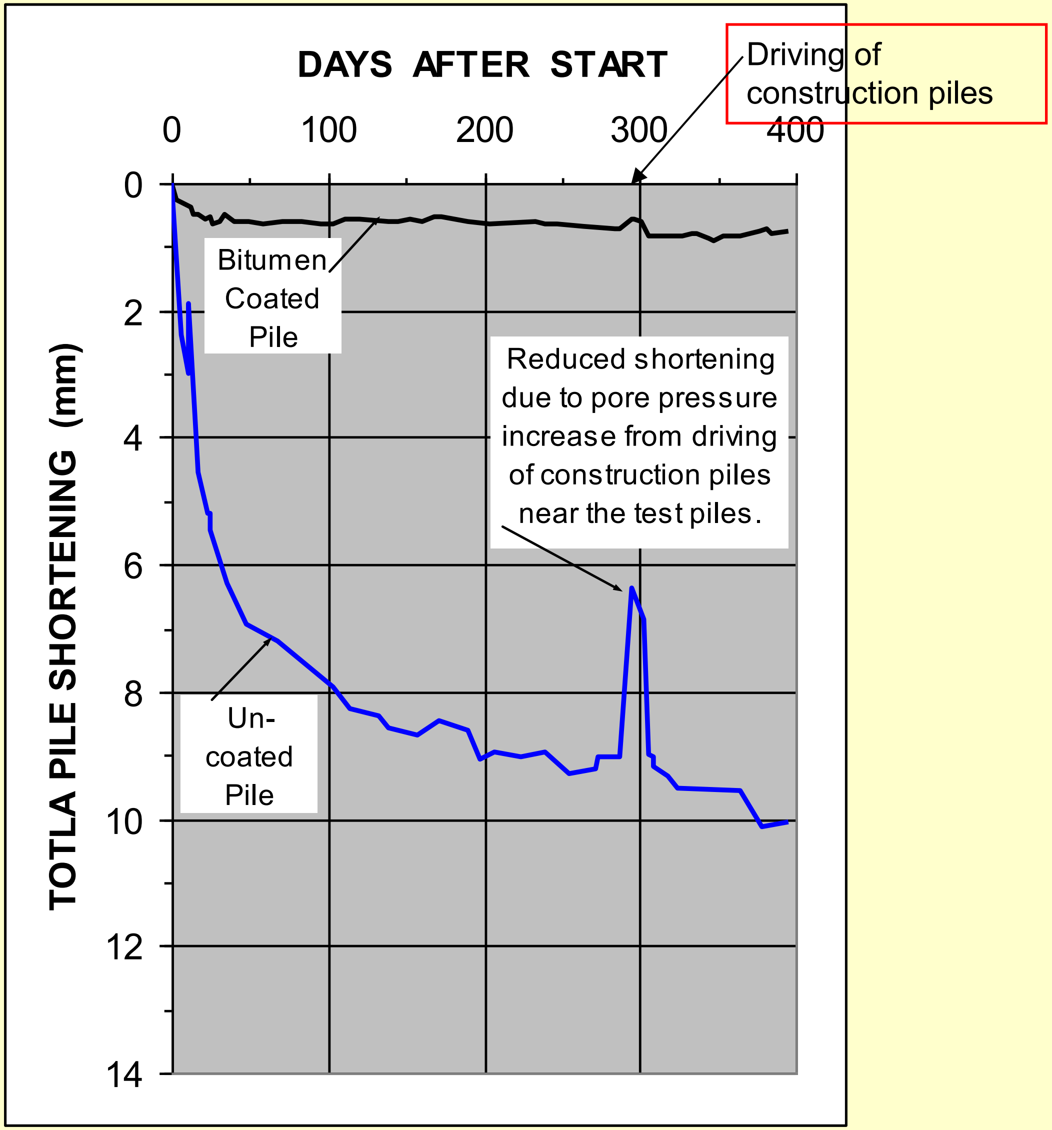
Profile of test site and piles. Heröya site.

(Bjerrum et al., 1969)



Notice the distinct **Force Equilibrium**, the **Neutral Plane**

Distribution of soil stress, excess pore pressure, soil settlement, and pile shortening. Heröya site. (Data from Bjerrum et al., 1969).



Pile shortening versus days after initial driving.
Heröya site. (Data from Bjerrum et al., 1969)

Compilation of Norwegian results

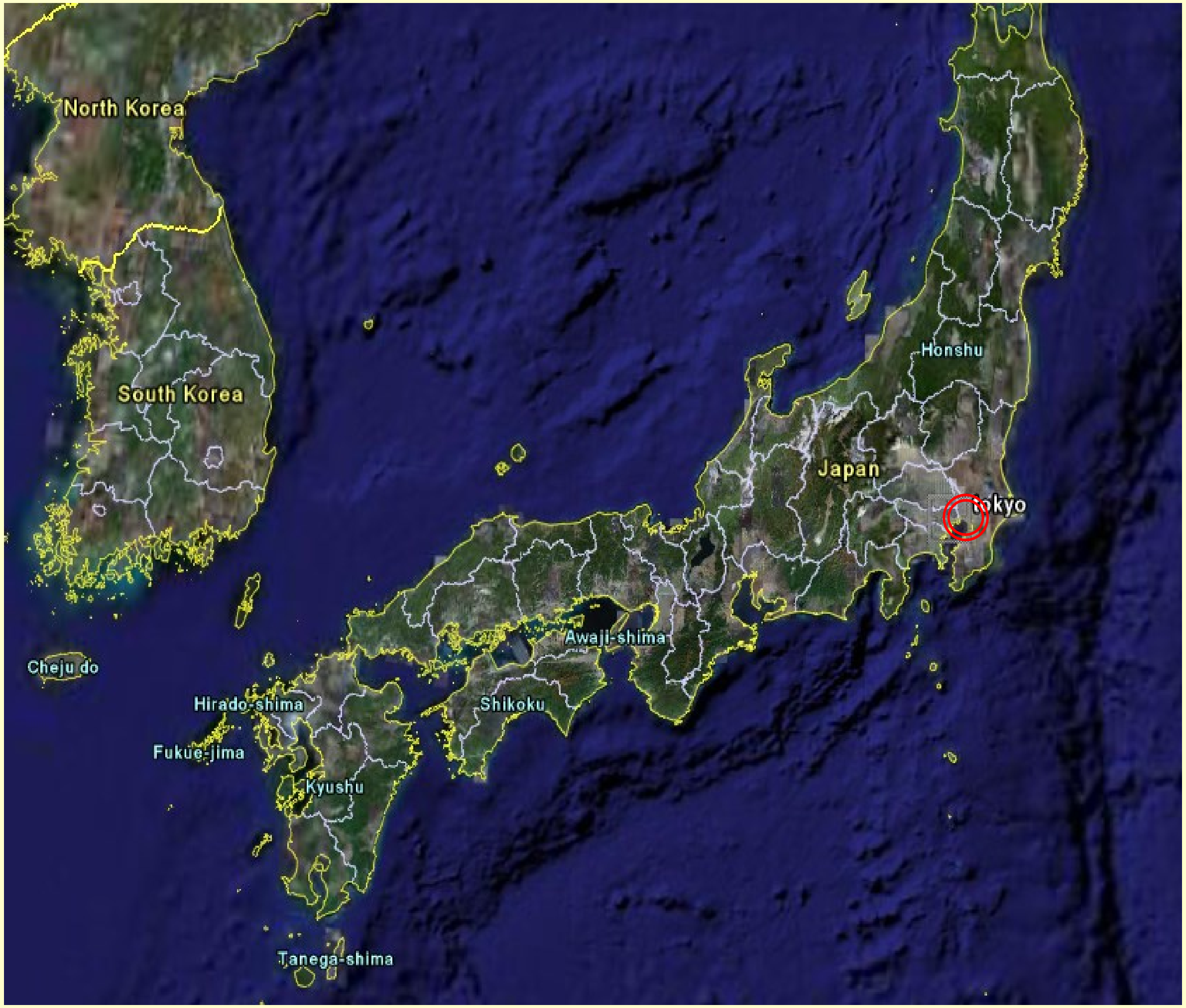
Table 1 Results of previous tests on unprotected steel piles.

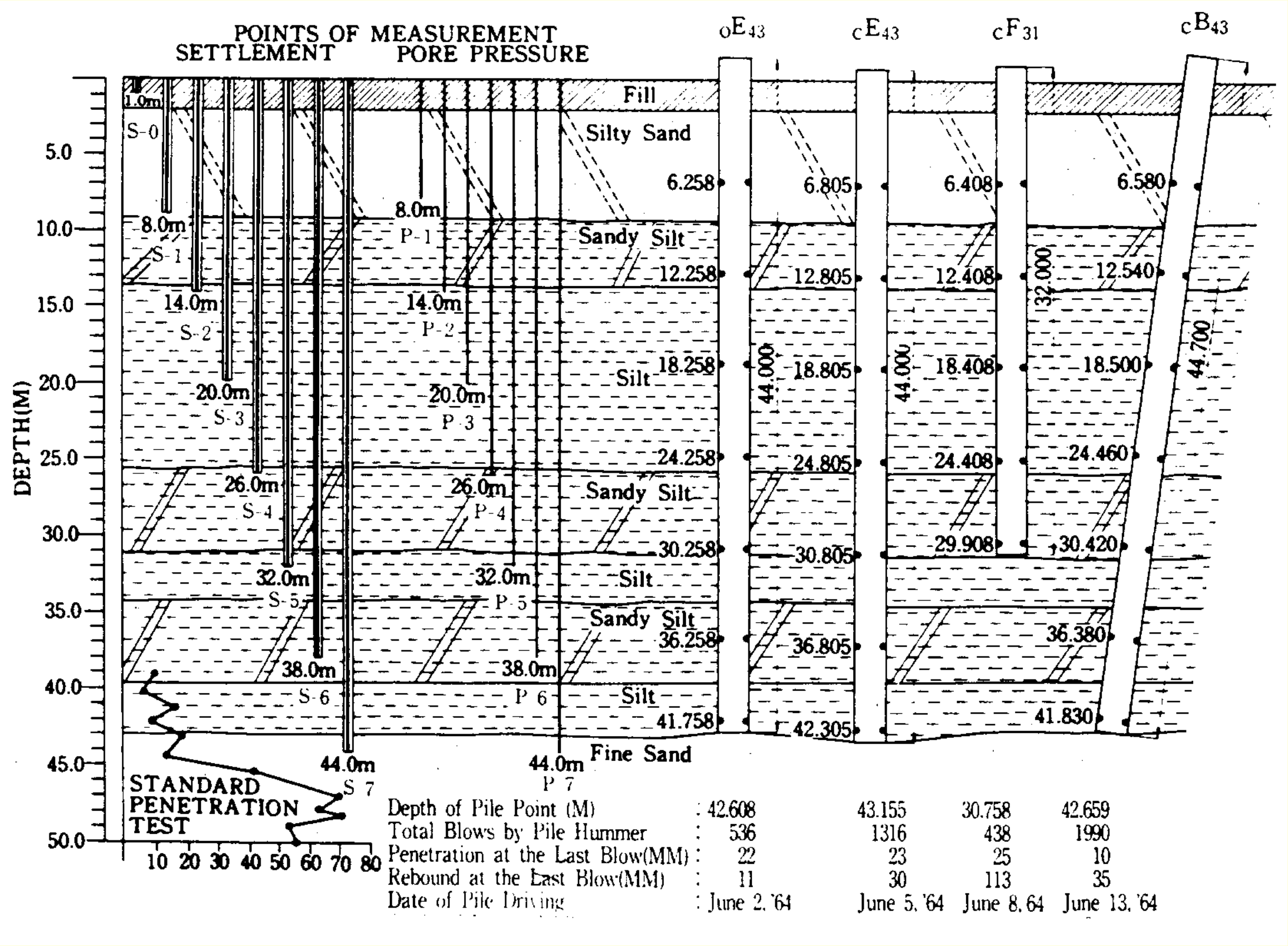
Site / Pile No.	Pile data: type	Pile data: length m	Time after driving years	Drag load tons	Settlement during observation: ground cm	Settlement during observation: pile cm	K tan φ' β
Sörenga							
B	I	53	5	≈400	≈ 200	10.0	0.20
C	II	57	2	300	≈ 27	5.3	0.18
G	II	41	2	250	≈ 7	3.2	0.23
Heröya							
85	III	32	$1\frac{1}{2}$	300	≈ 30		0.25
A	IV	≈30		120	≈ 20	3.3	0.26
Alnabru							
F 6	III	32	1	≈300	0		

Pile type: I: KP 24, 47 cm, II: Tubular steel pile, Ø 50 cm, III: Tubular steel pile with concrete, Ø 50 cm; IV: Tubular steel pile, Ø 30 cm.

CASE #2

Endo et al. 1969, presented a very ambitious study in Japan on dragload on four instrumented steel piles during a period of three years. The soils consist of **silt and clay on sand**. The case history is one of the few that actually also measured settlement.

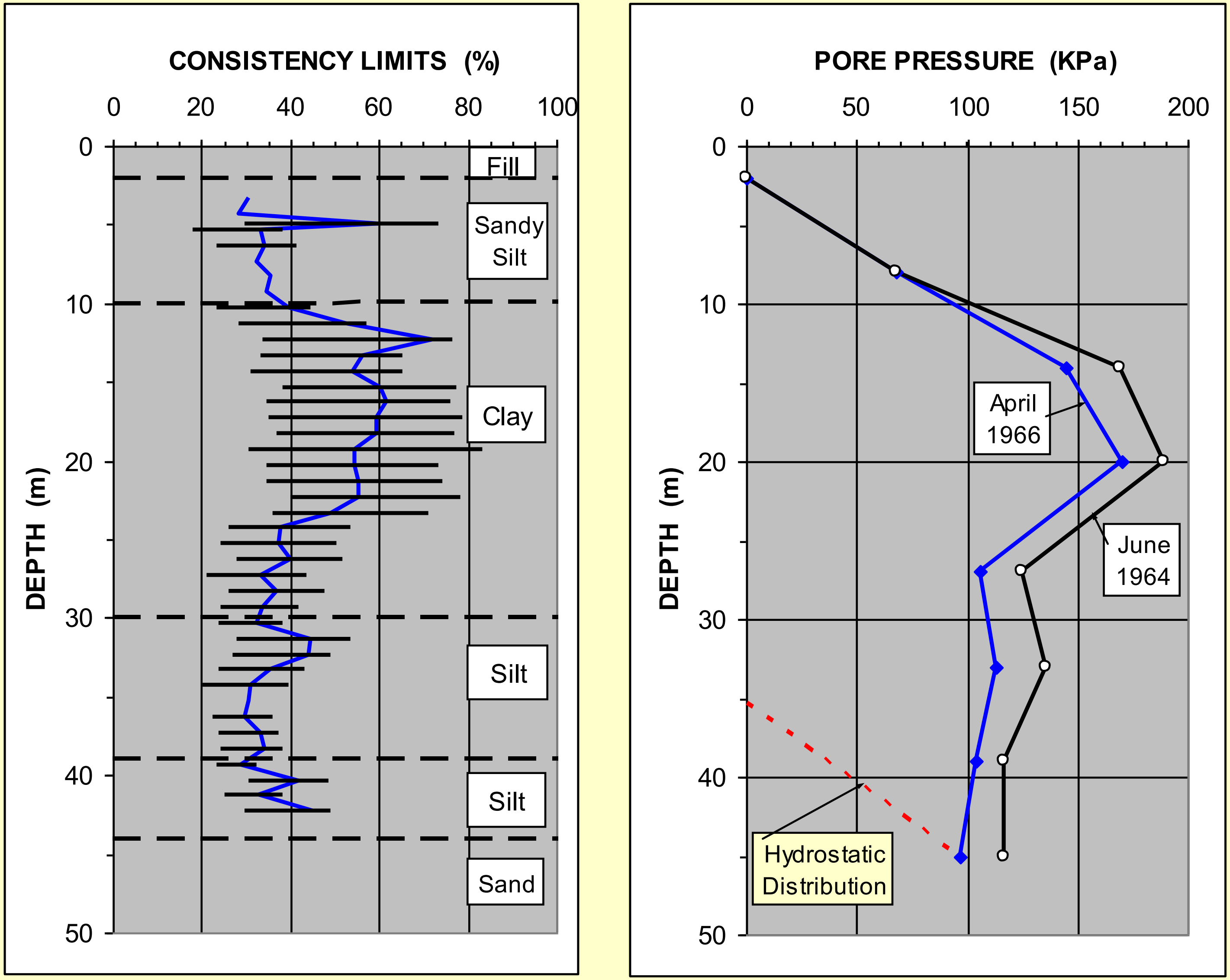




Profile of test site and piles

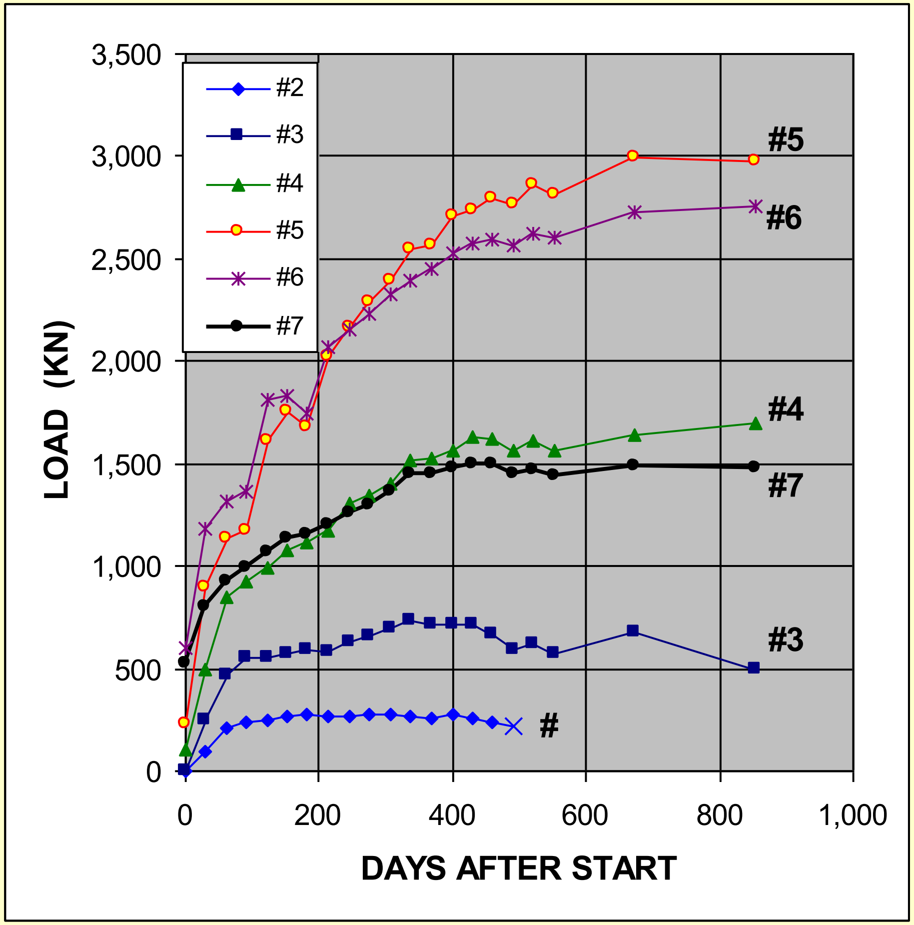
Closed-toe, Open-toe, Inclined, and short pile

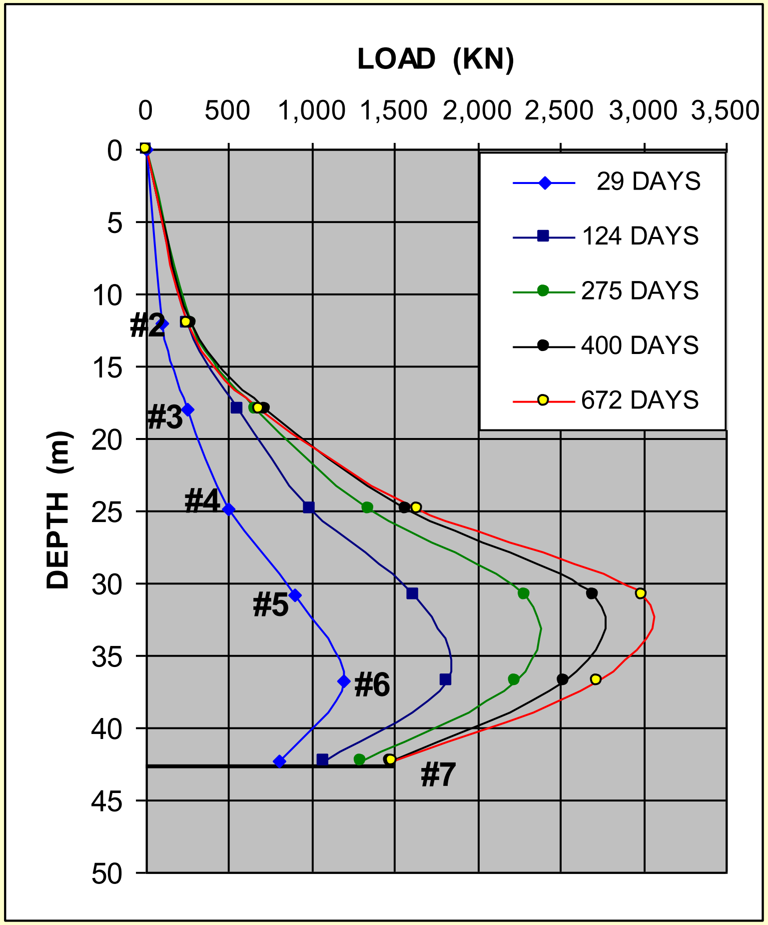
(Endo et al., 1969)



Soil profile and pore pressure distribution

Data from Endo et al. 1969

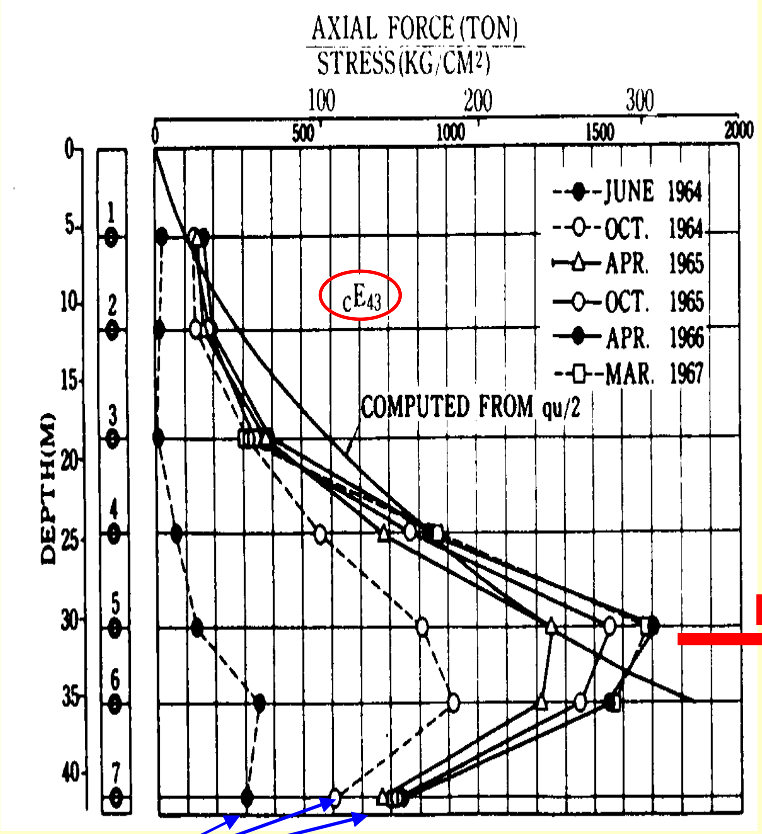




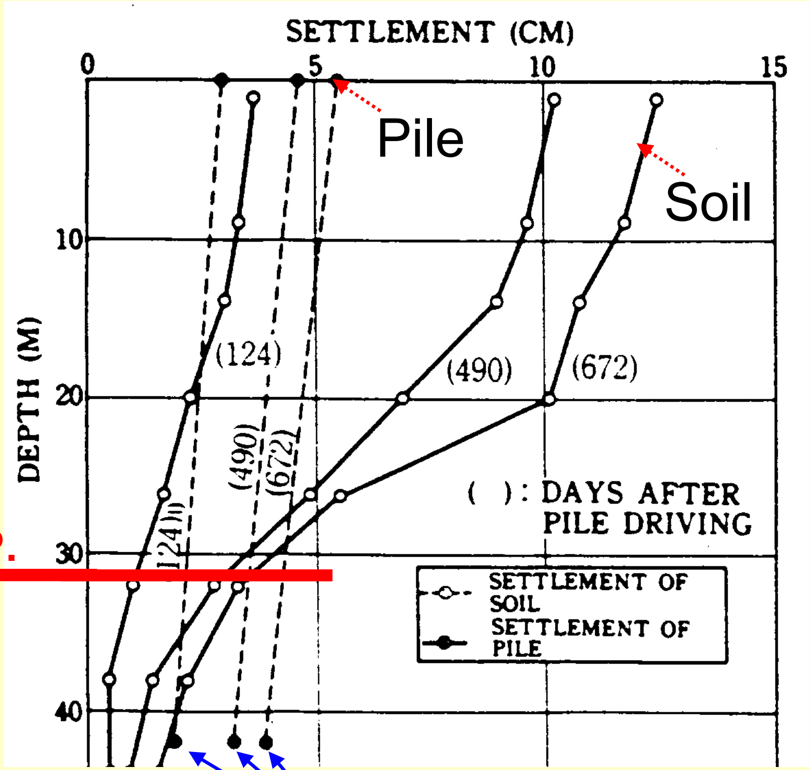
Loads from shortening of closed-toe pile
June 1964 through March 1967
(Data from Endo et al., 1969)

Combining the Pile cE43 distributions of load and of settlement measured June 1964 through March 1967

Load Distribution



Settlement Distribution



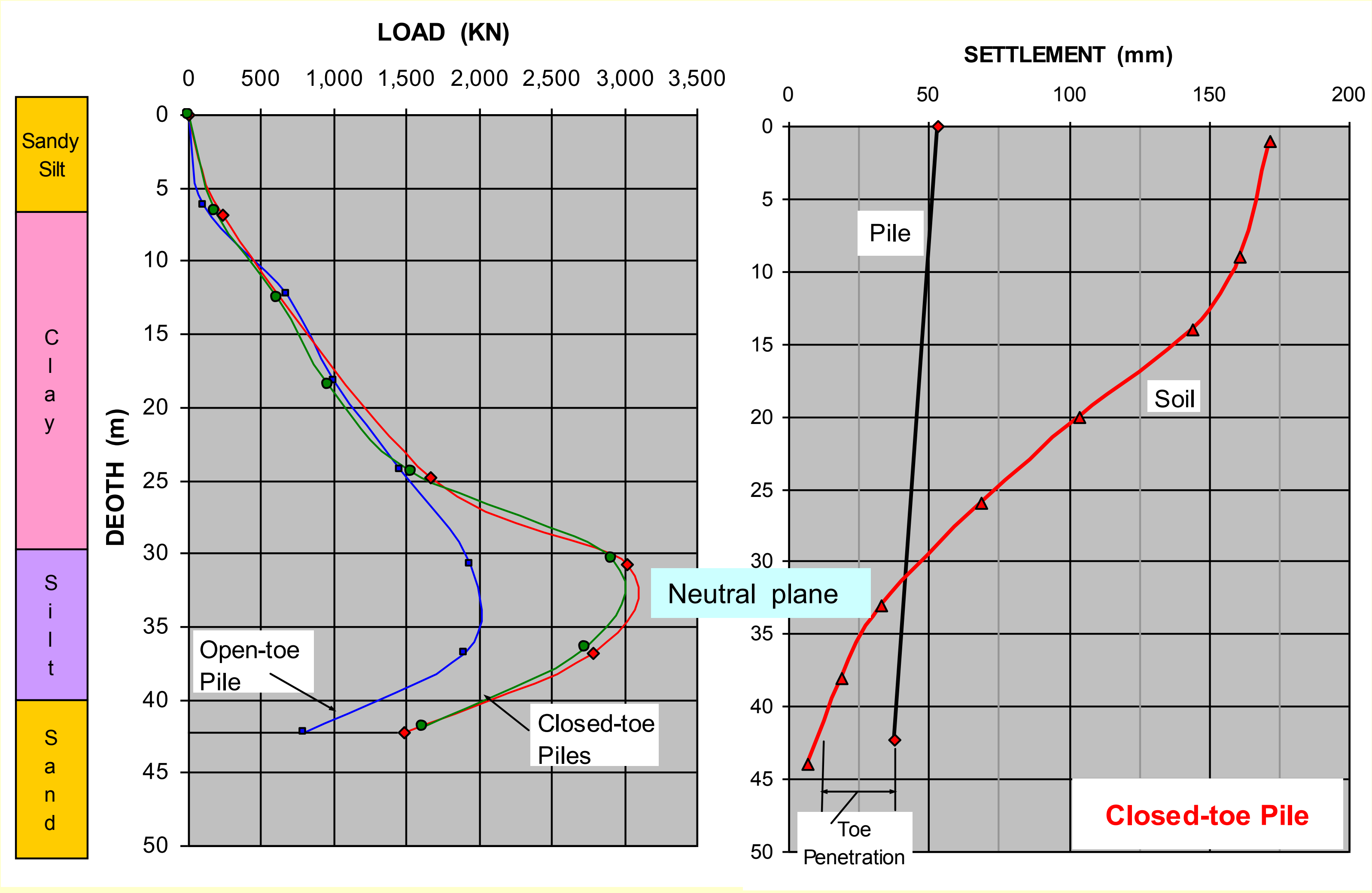
N.P.

Notice the increasing mobilization of toe resistance

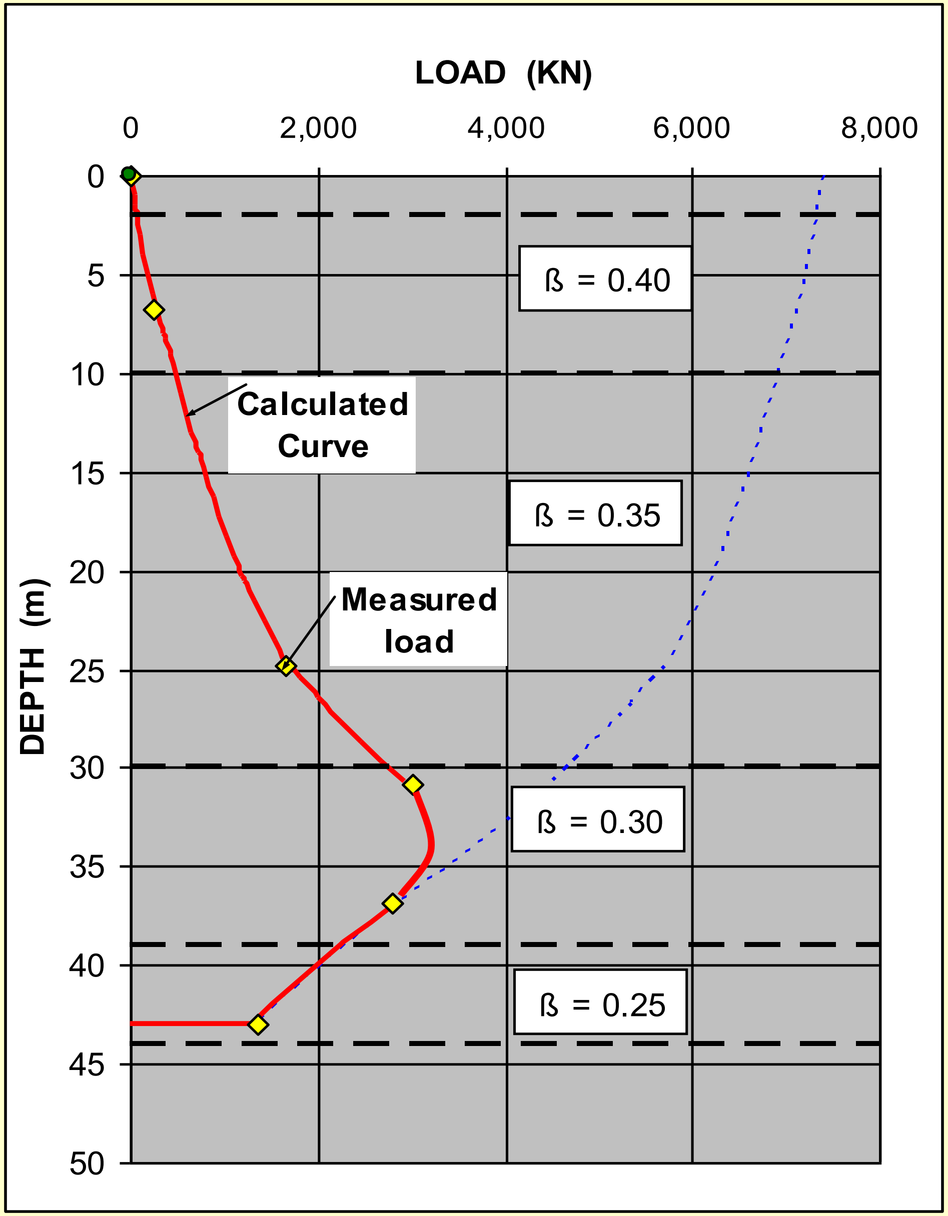
Notice the increasing movement of the pile toe

(Endo et al., 1969)

Neutral plane = Force Equilibrium = Settlement Equilibrium



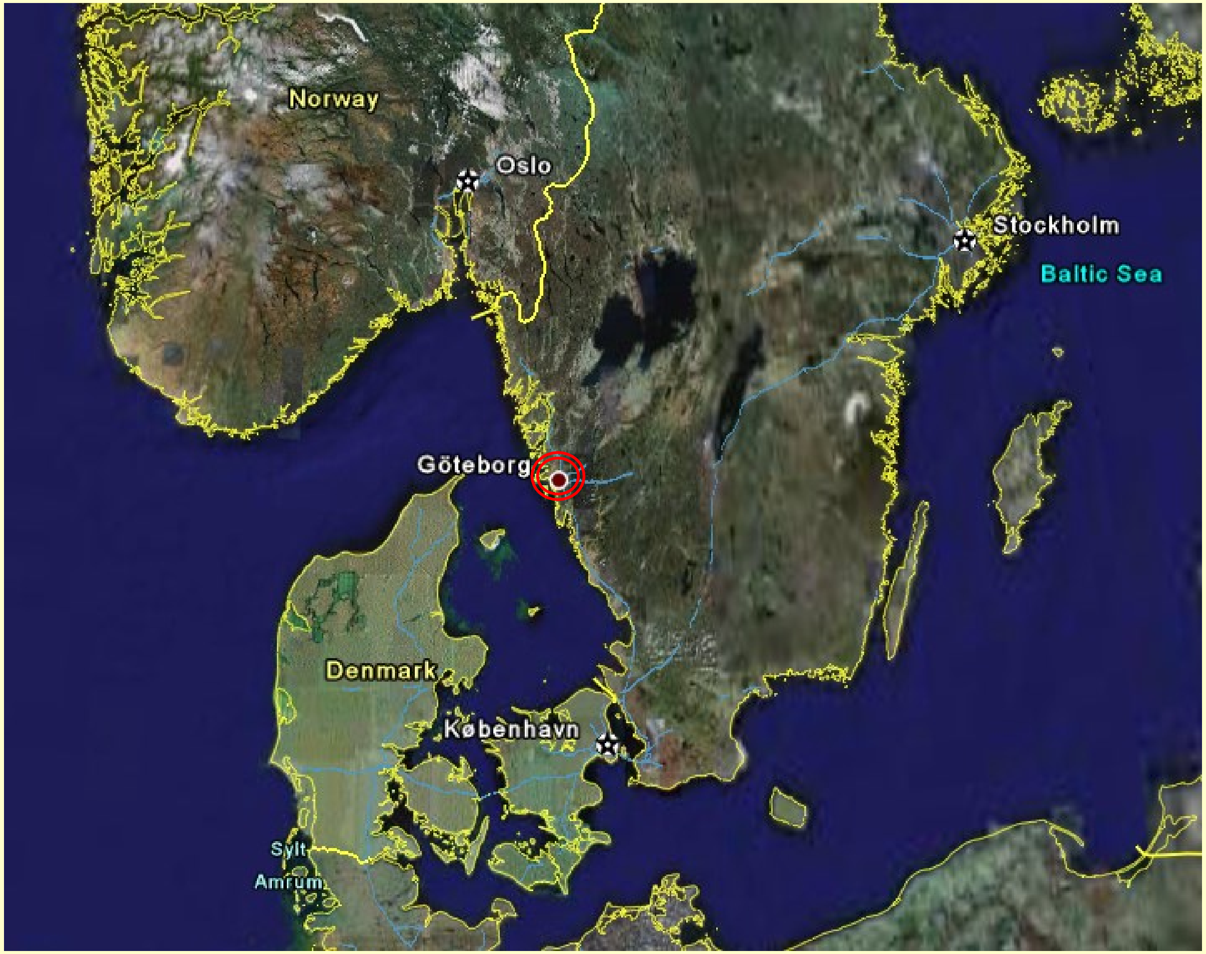
Load distribution in **the three long piles together** and settlement of soil and piles measured March 1967 **672 days** after start. (Data from Endo et al., 1969).

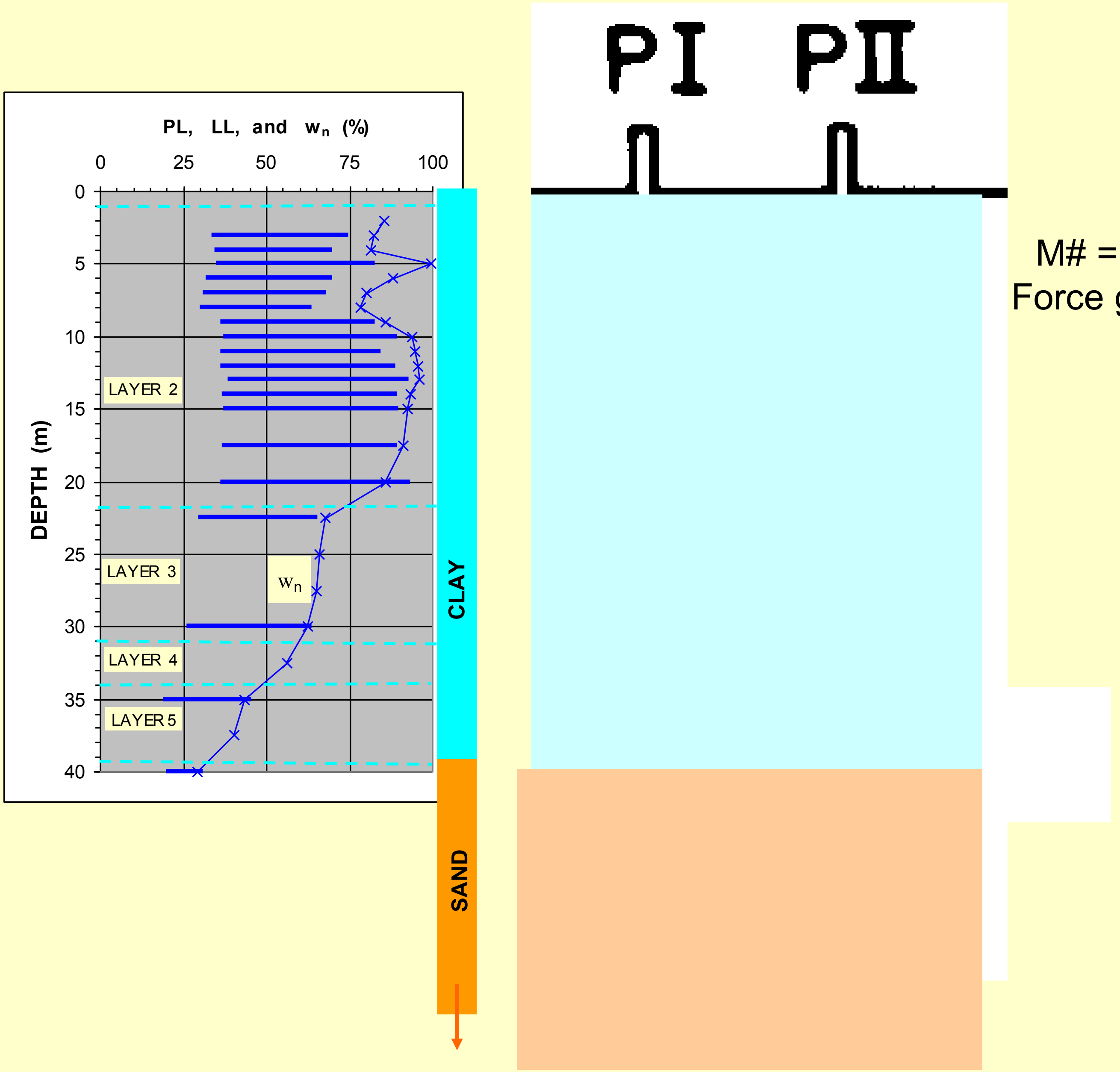


Measured load distribution and distribution matched to measured values in effective stress analysis. (Data from Endo et al., 1969).

CASE #3

Fellenius (1972) presented a study of two instrumented, precast concrete piles driven through marine clay and into sand at Bäckebol, Göteborg, Sweden





M# = Pile Force gages

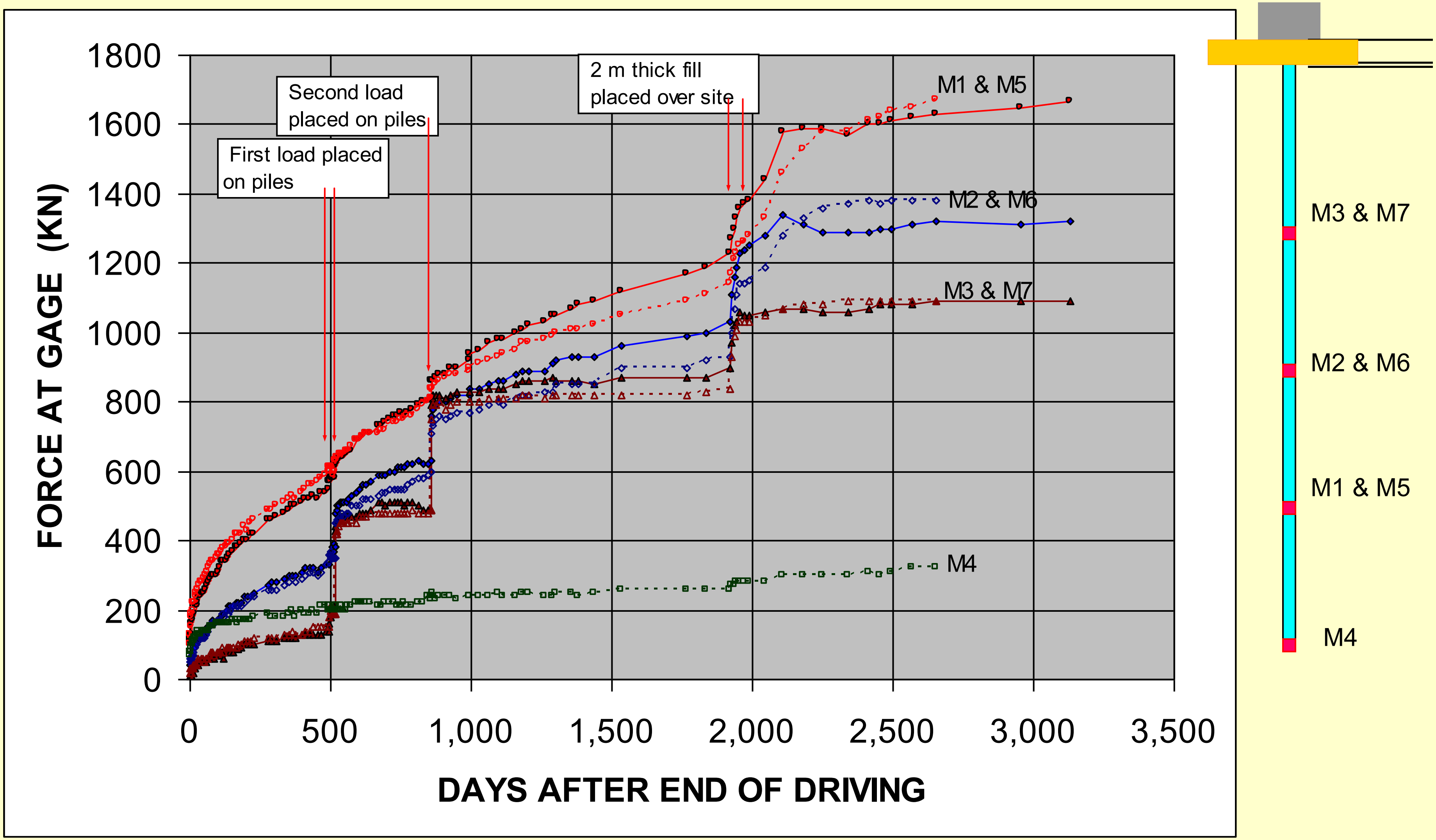
Pile II segments with Gage M4 at pile toe

View of site along the Göta River

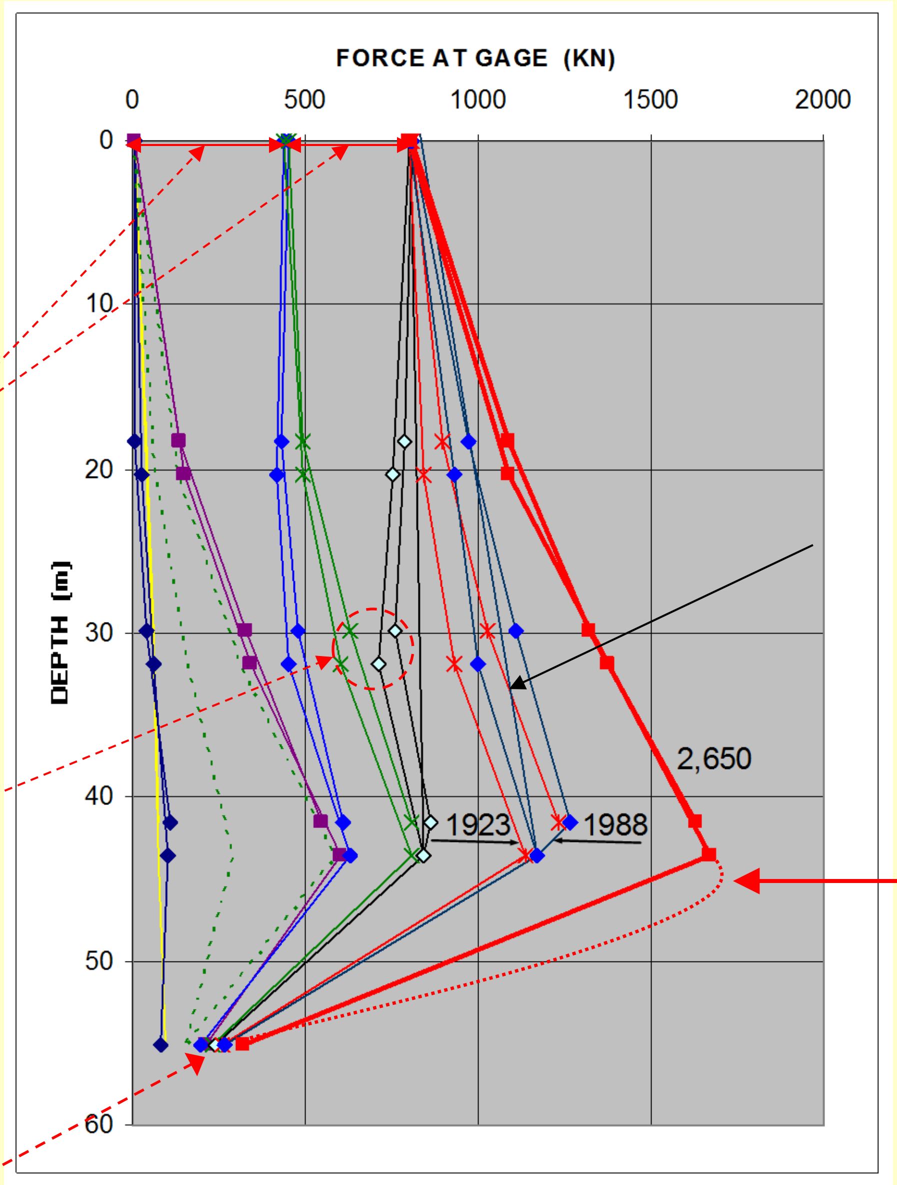
A few years earlier (1950) in Surte across the river

First load (360 KN) placed on the piles

Piles with loads applied but before fill was placed over the site



Measured loads in piles versus time after driving



= “LIVE LOADS”

Placing the fill

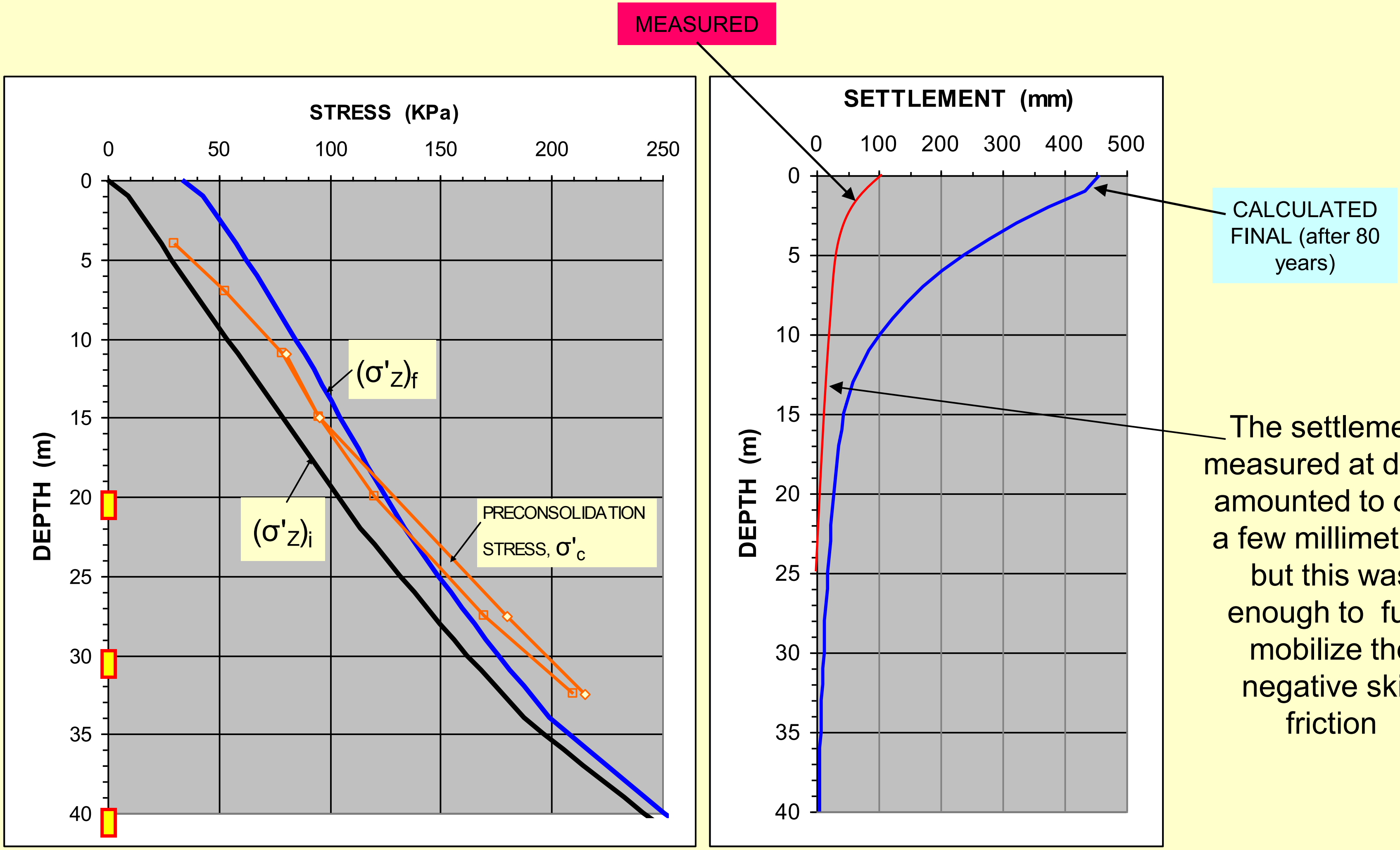
Note, the dragload was eliminated by the “live load”

Neutral Plane

That the toe resistance is small is due to that the movements are not large enough to mobilize any larger toe resistance

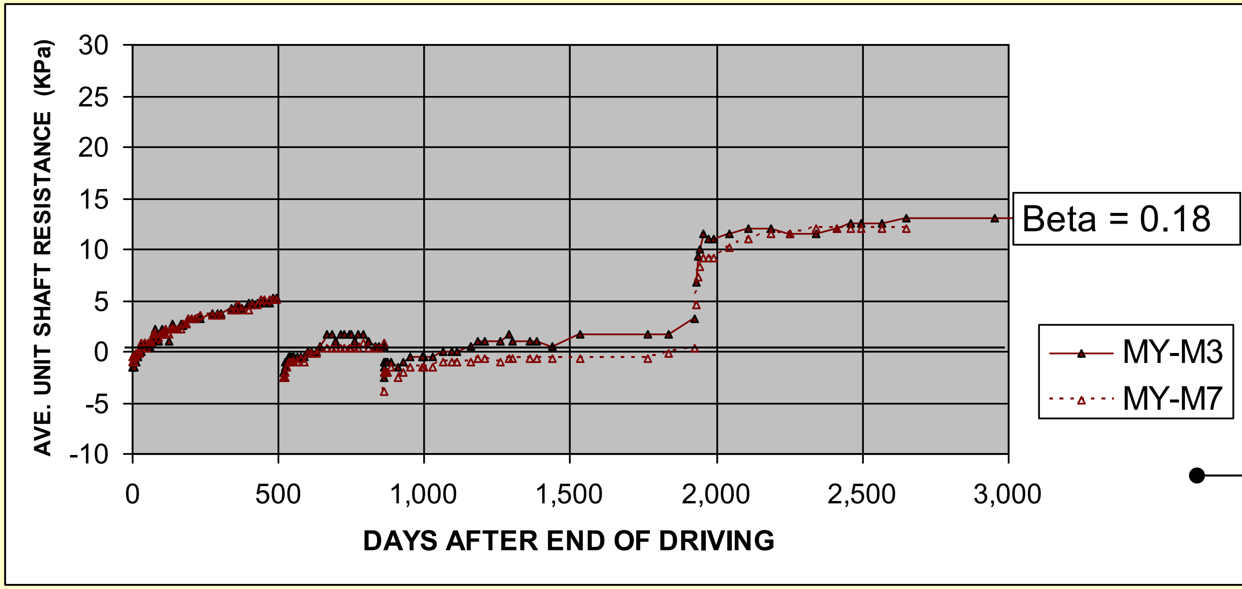
Distribution of load in Piles I and II

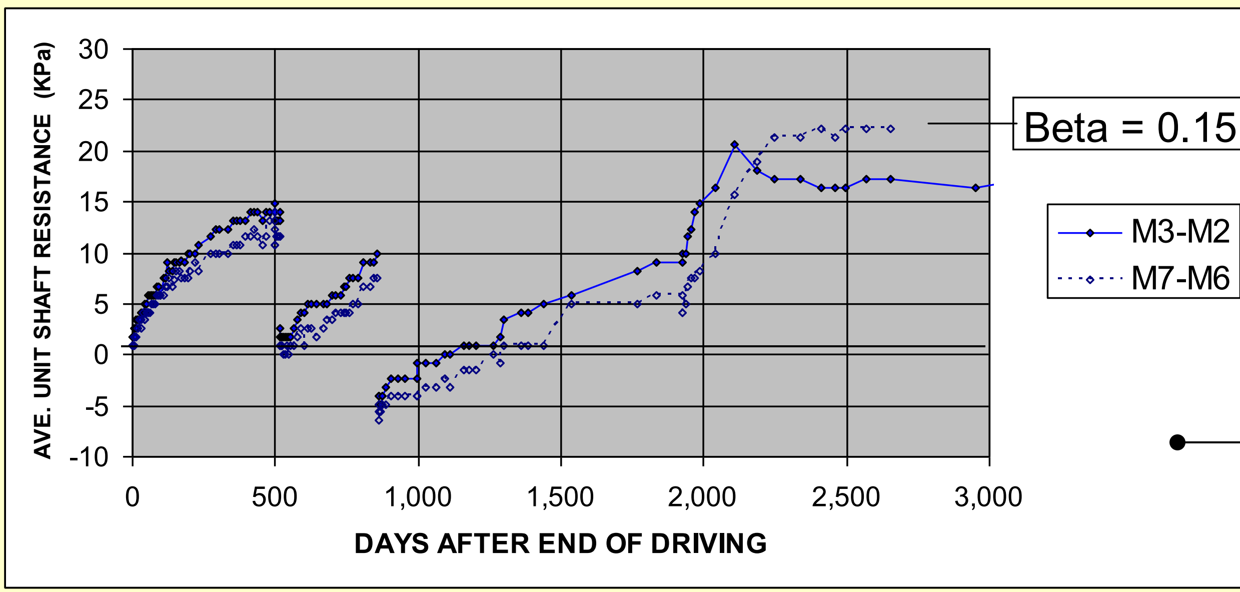
Distribution of measured and calculated consolidation settlement due to the fill



The settlement measured at depth amounted to only a few millimeters, but this was enough to fully mobilize the negative skin friction

Force gage locations





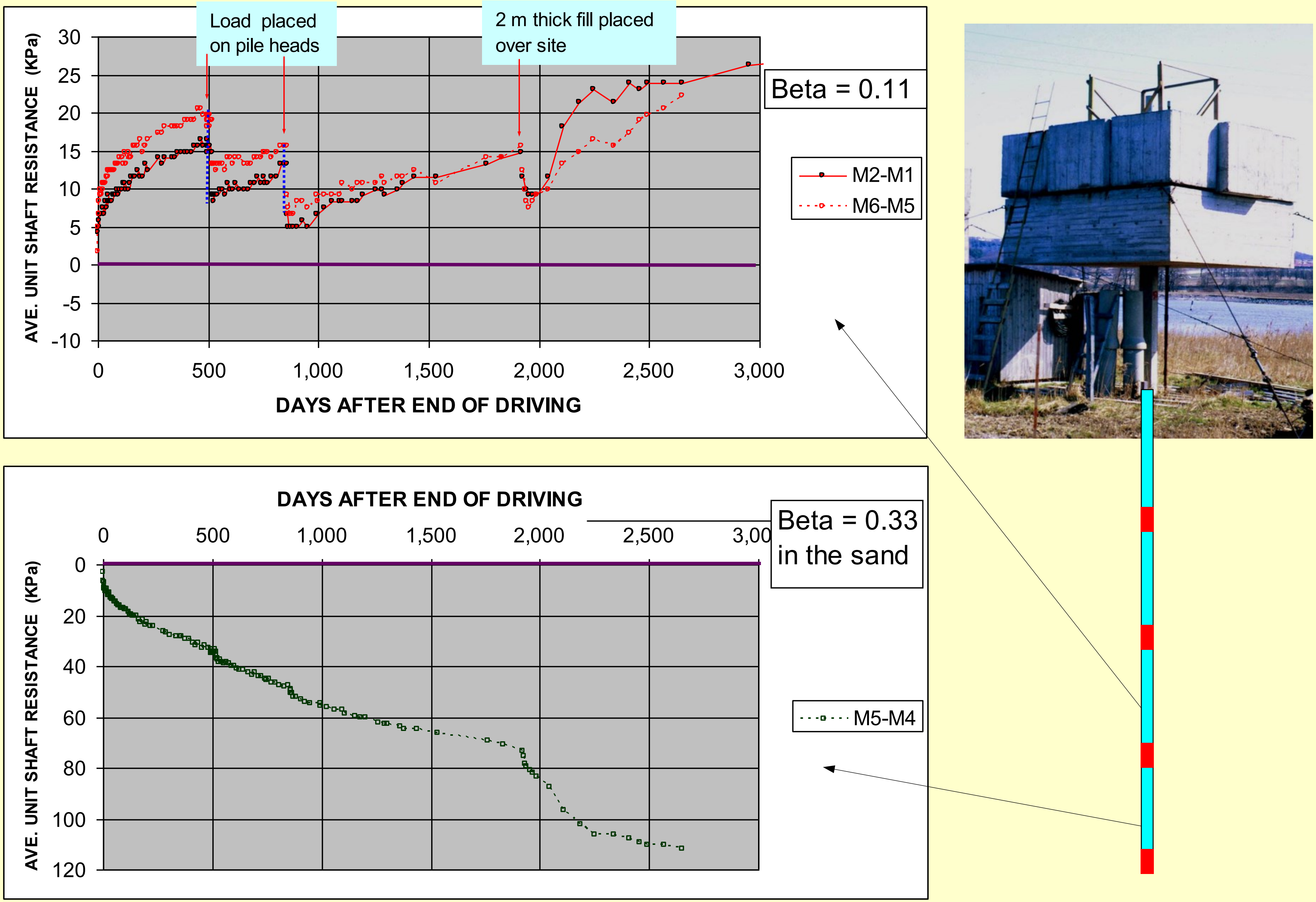
Average shaft shear in Piles PI and PII

between

- Ground Surface and Load Cells M2 and M3

and

- Load Cells M2 and M3, and M6 and M7



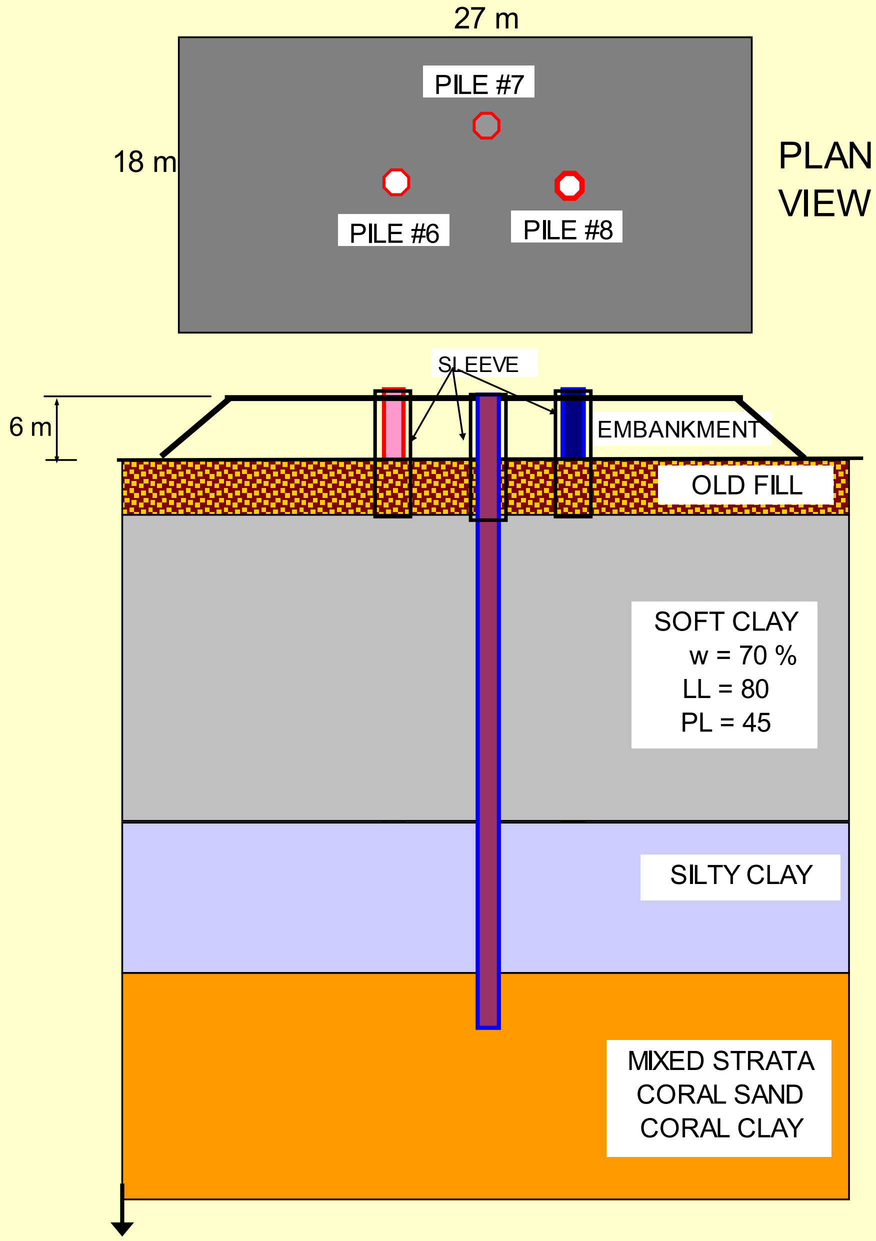
Average unit shaft shear in Piles PI and PII between Load Cells

CASE #5

FHWA Project, Keehi Interchange, Honolulu,Hawaii 1977

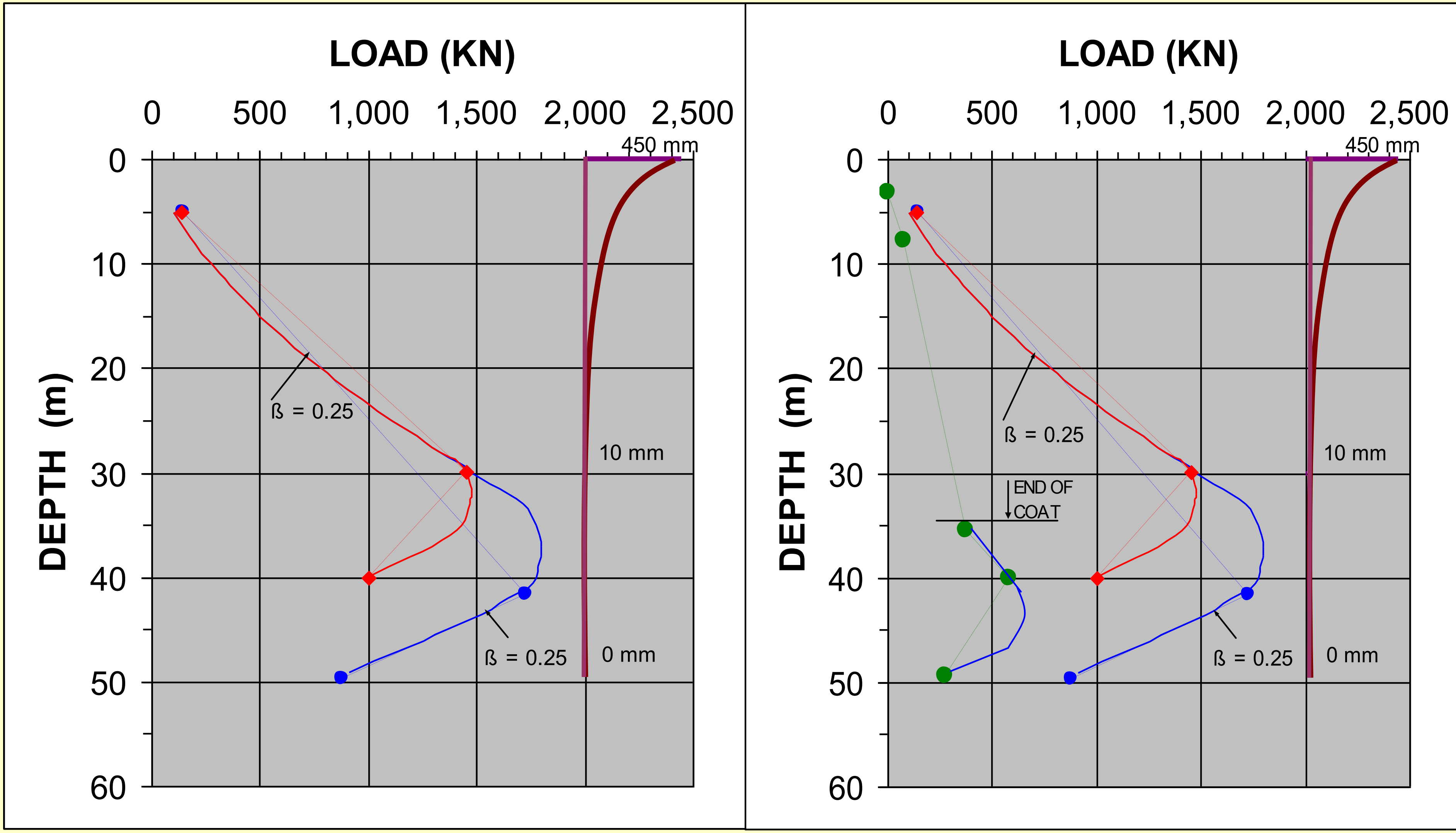
Data from Clemente, 1981

CASE #5



Test piles, embankment fill, and soil profile

Data from Clemente, 1981



35 m uncoated pile ==> 1,770 KN
35 m bitumen coated pile ==> 375 KN for less than 1/16-inch (1.5 mm) coat
= Reduction to 20 % of uncoated load
3/16 coat + stiff outer skin ==> elimination of drag load

3/16 inch = 4 mm

Bitumen coating of piles

Remember, bitumen is a liquid!

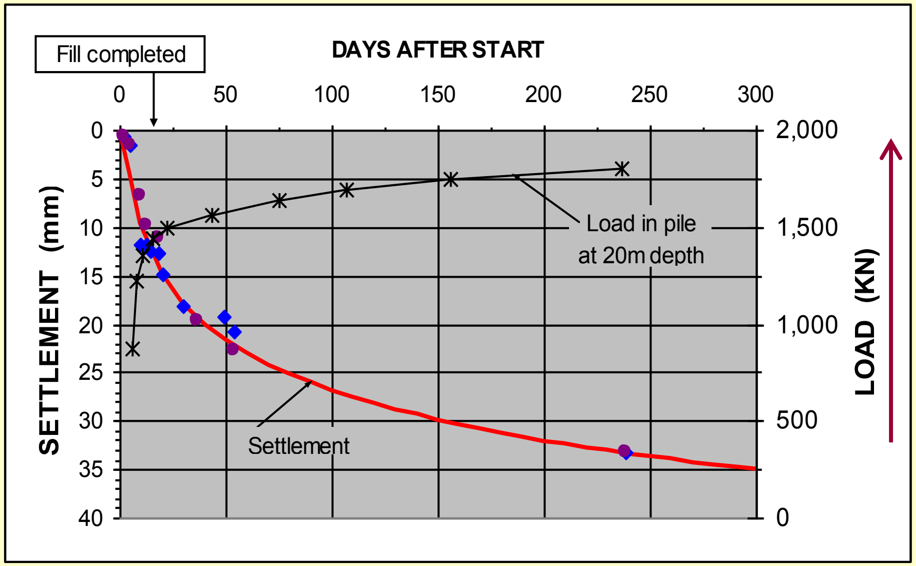
Verifying thickness

CASE #6

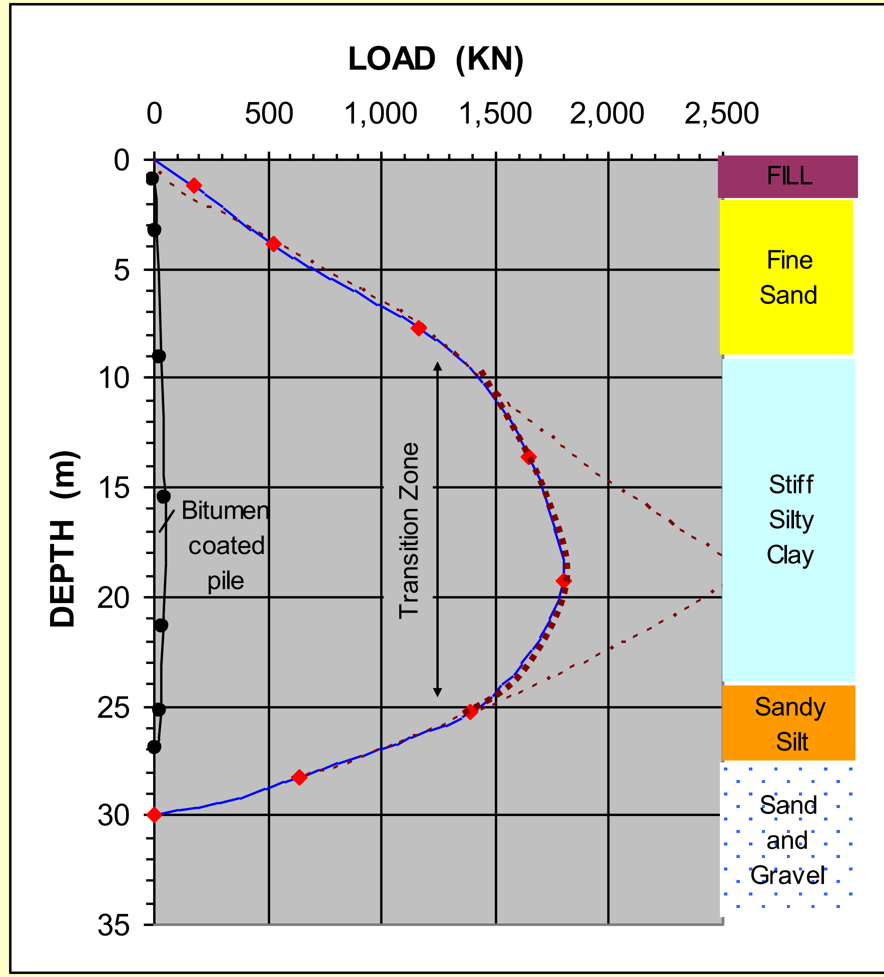
A study in Australia of two 760 mm, strain-gage instrumented, open-toe pipe piles driven through a 6 m sand layer over a 15.5 m thick <u>overconsolidated</u> silty clay deposited on silt and sand. Ground surface settlement was induced by placing a 3 m high surcharge over 200m x 100m area around the test piles, causing drag load.

Data from Walker et al. (1973

CASE #6



Ground surface settlement due to a 3 m high surcharge placed over 200m x 100m area around the test piles.

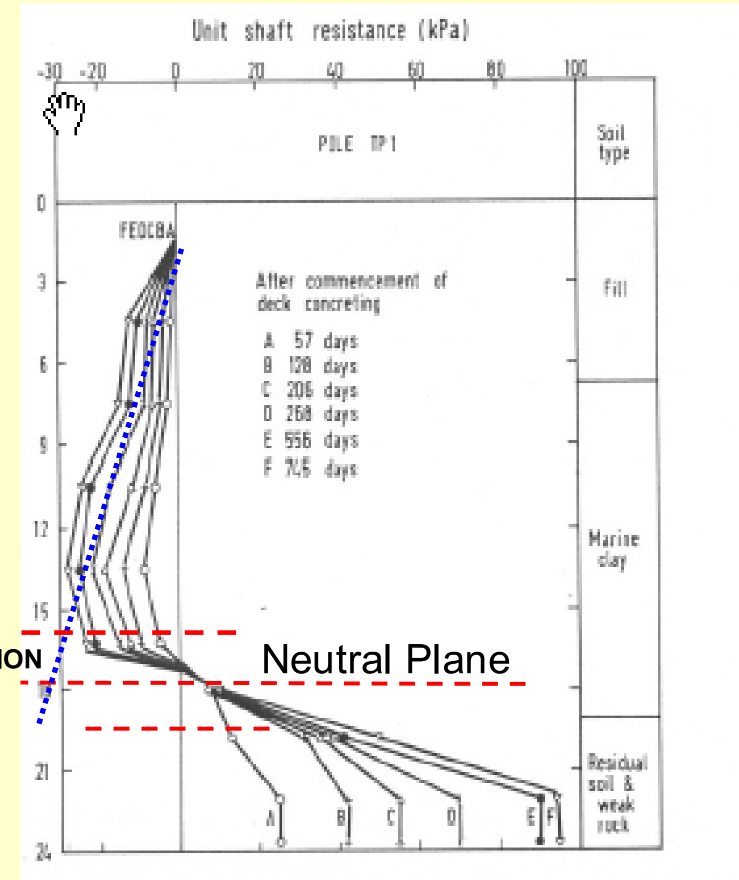


Load distribution on two pipe piles, one bitumen-coated and one uncoated

Data from Walker et al. (1973

CASE #7

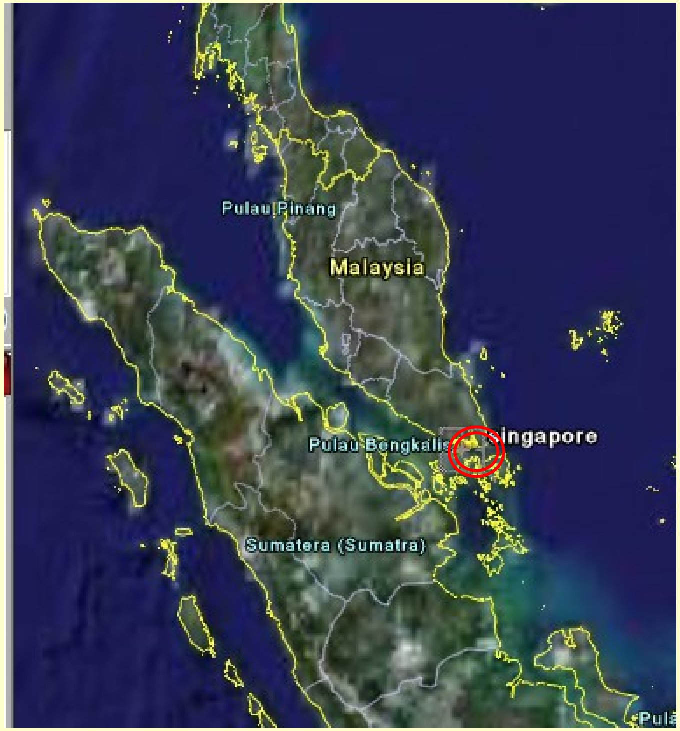
Leung, C.F, Radhakrishnan, R., and Siew-Ann Tan (1991) presented a case history on instrumented 280 mm square precast concrete piles driven in **marine clay** in Singapore

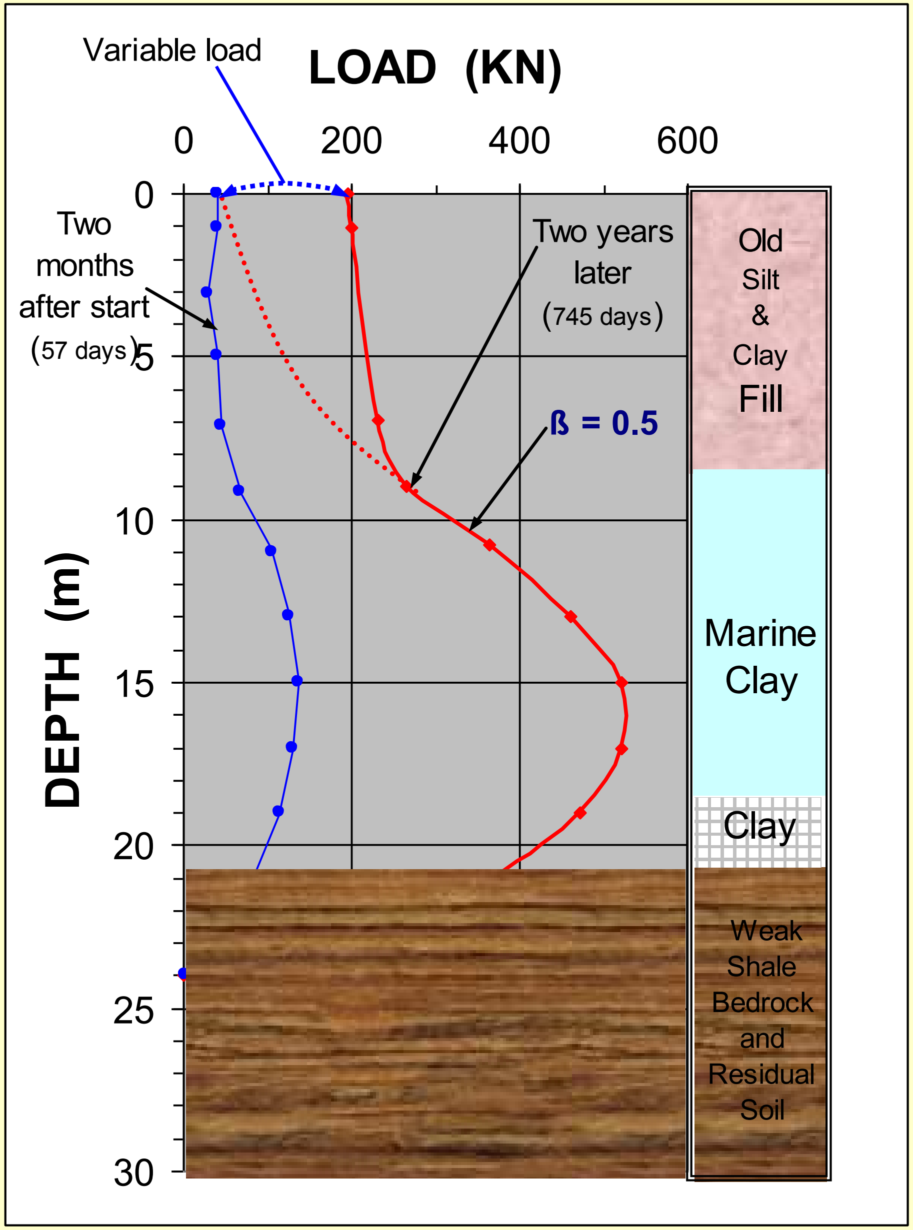


TRANSITION ZONE

Distribution of Unit Shaft Resistance along Pile TP1 under Service Loads

Note, the distribution of negative skin friction is linear (down to the beginning of the transition zone) indicating the proportionality to the effective overburden stress

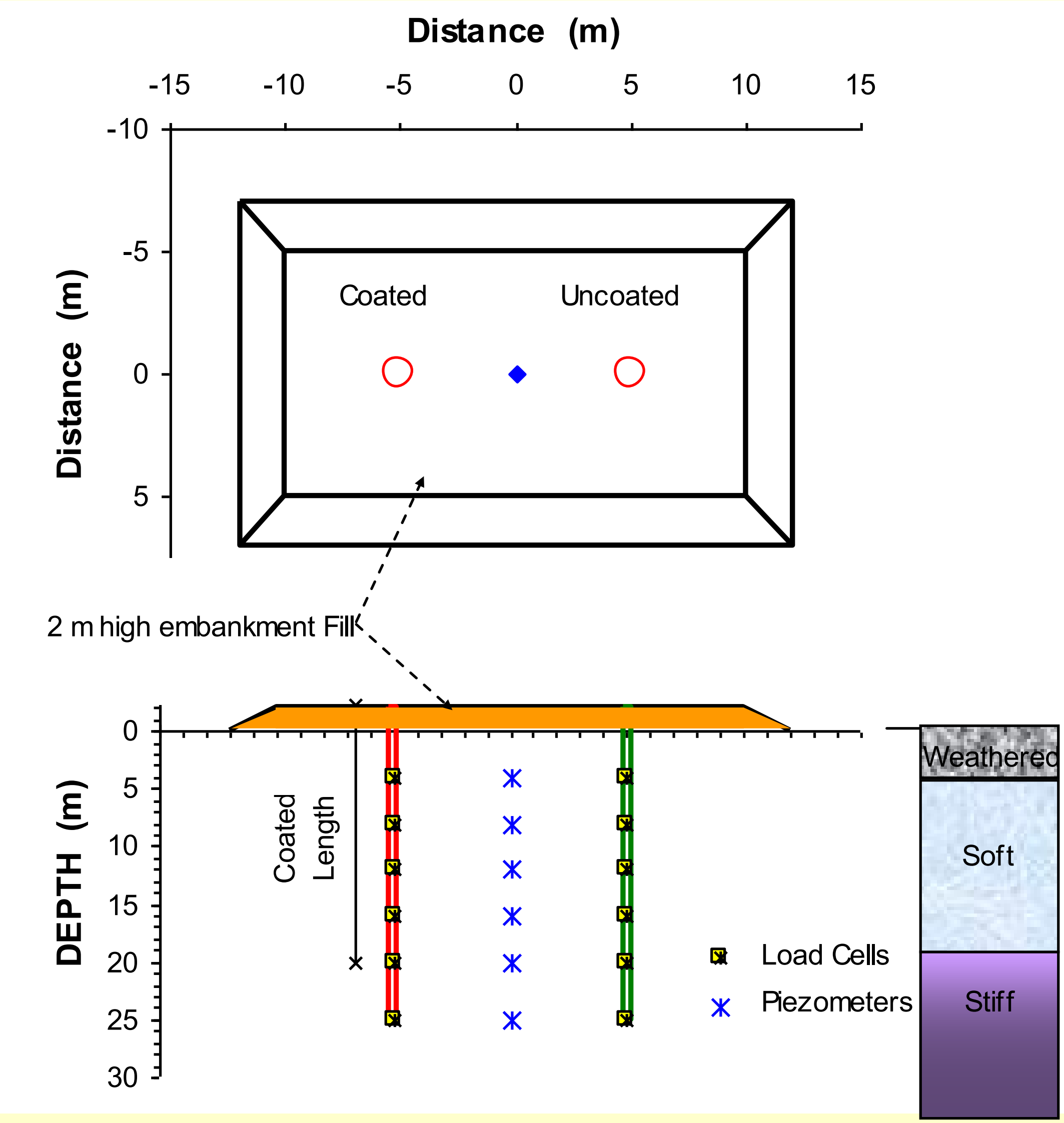




Data from Leung, Radhakrishnan, and Tan (1991)

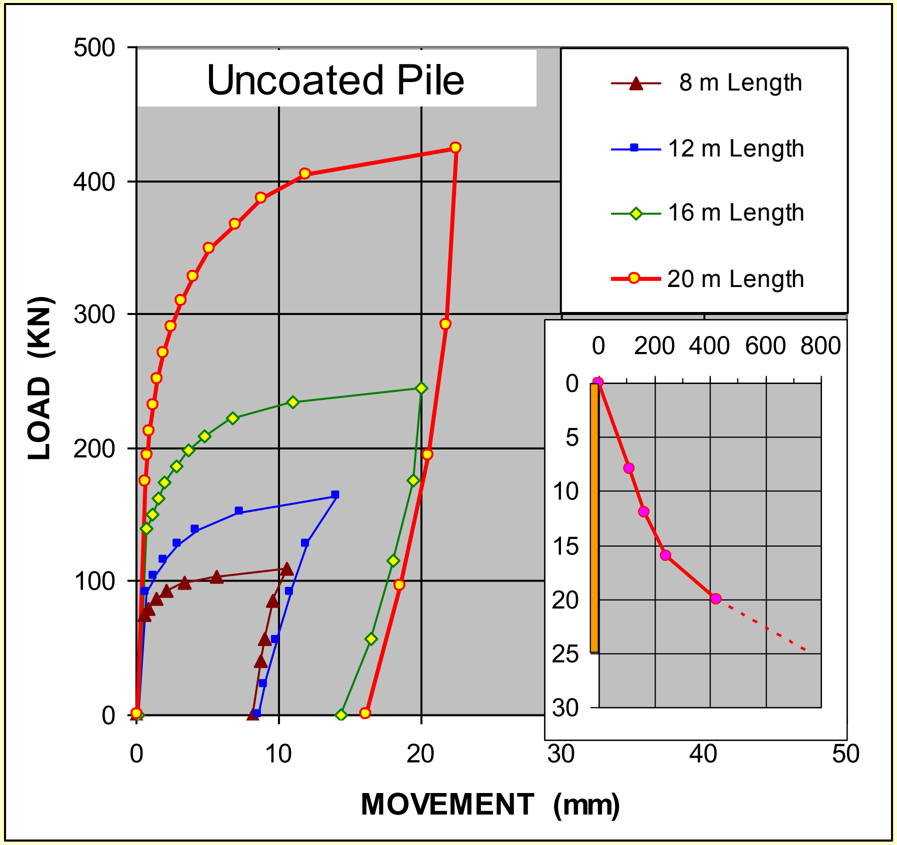
CASE #8

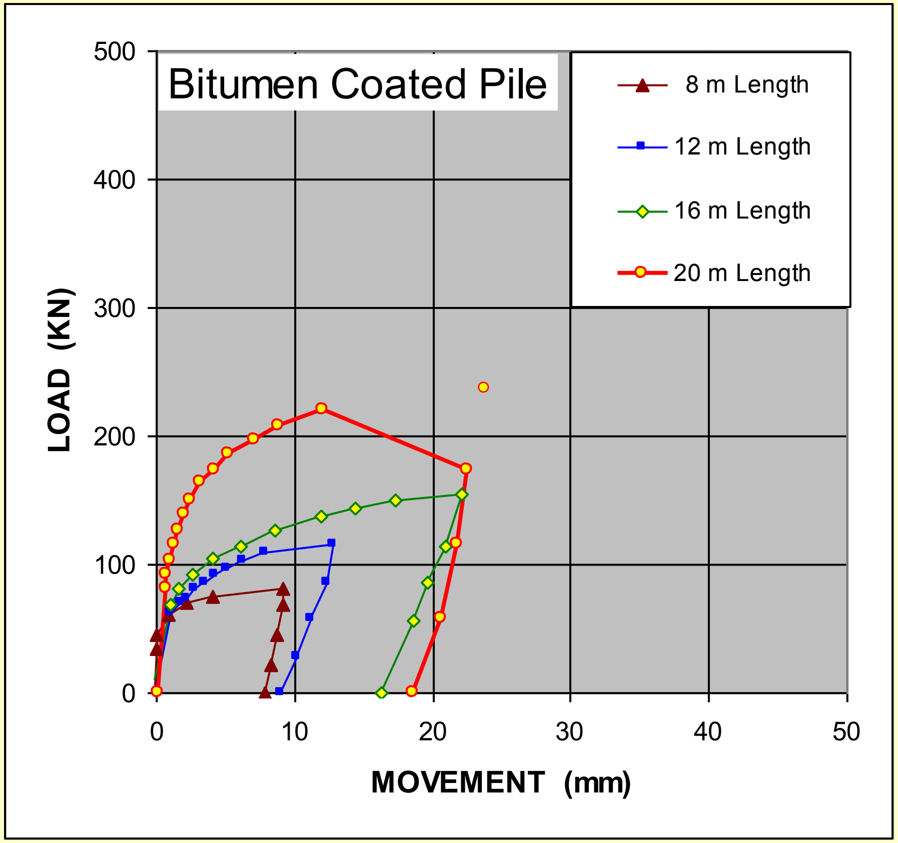
- Indraratna et al. (1992) reported results from full-scale field tests on instrumented 400 mm diameter, 25 m long cylinder piles driven into Bangkok clay. Tests were made in push and pull on bitumen-coated and uncoated piles.



Indraratna et al. 1992

Installation was made one 4m long segment at a time. Before splicing on the next segment and continuing, **pull tests** were carried out. The diagrams below show the measured load-movement curves.

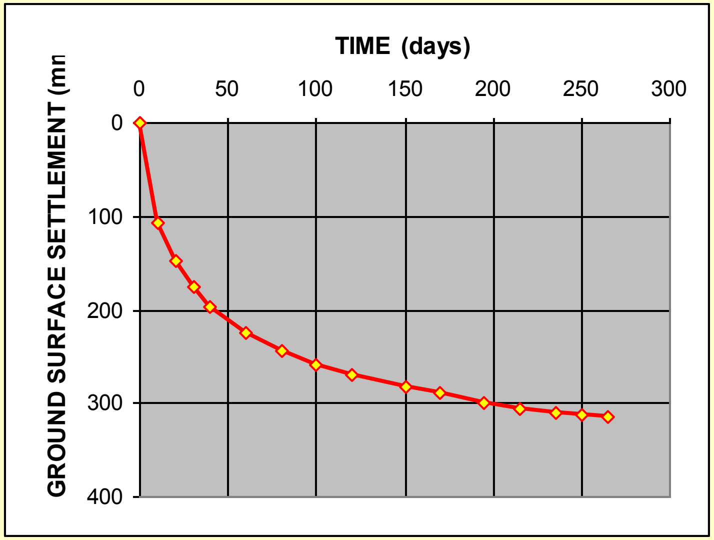




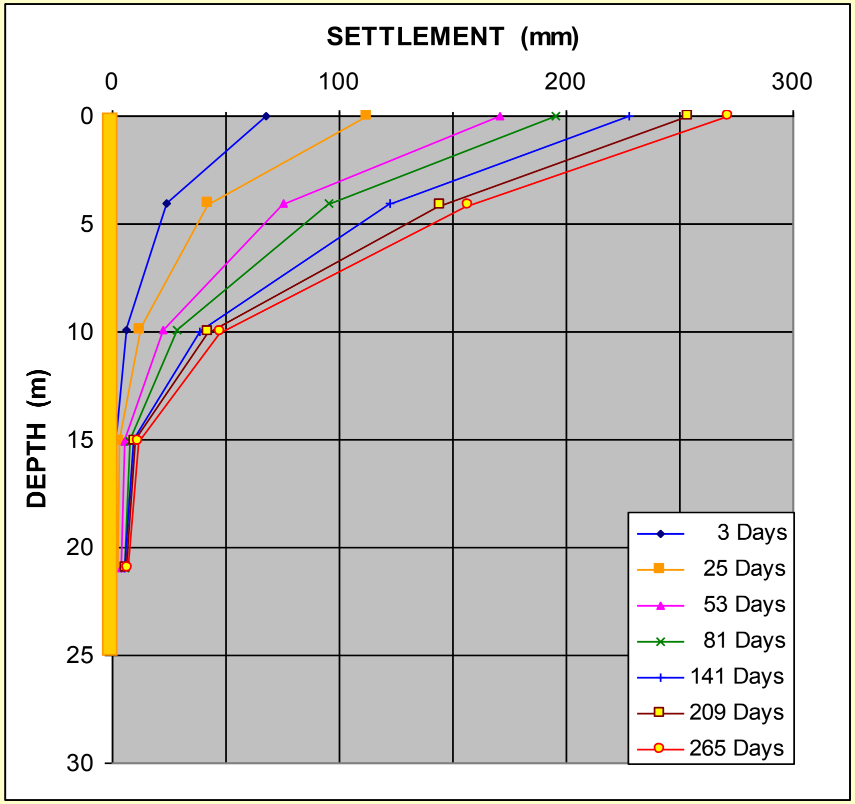
Pull Test Results

Data from Indraratna et al. 1992

After the driving of the piles, the embankment was placed around the piles and settlement, pile forces, and pore pressures were measured during an eight-month period.



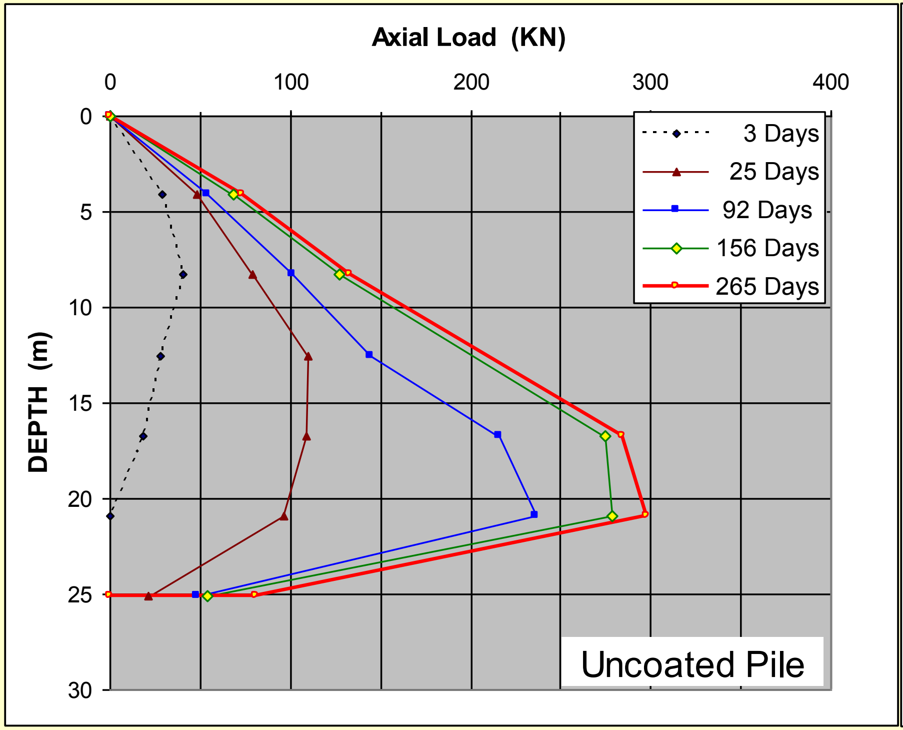
Ground surface settlement at center of embankment

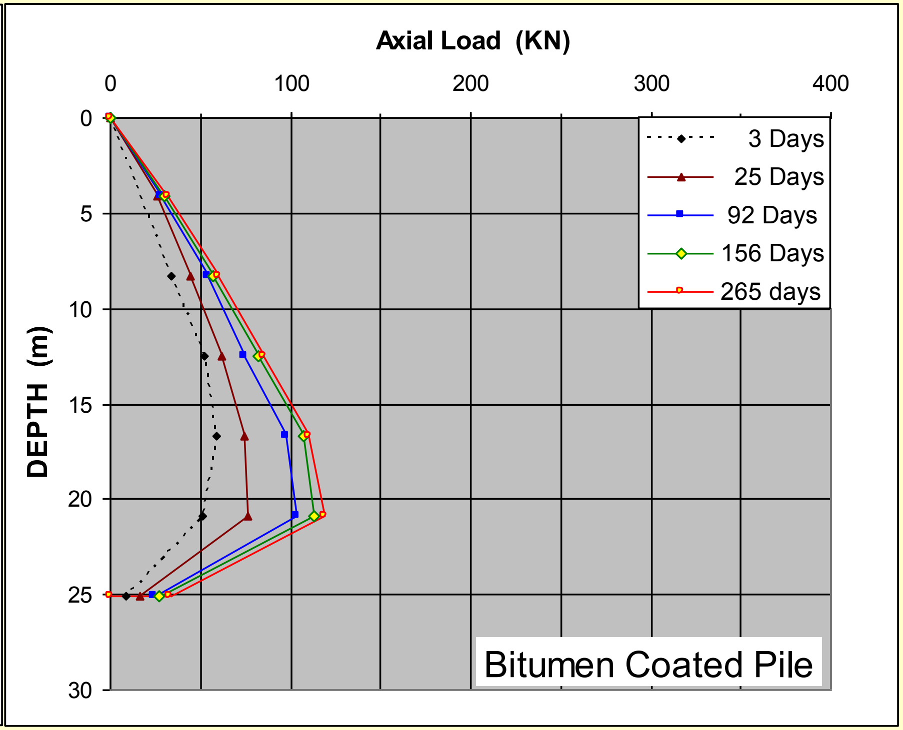


Vertical distribution of the settlement

Data from Indraratna et al. 1992

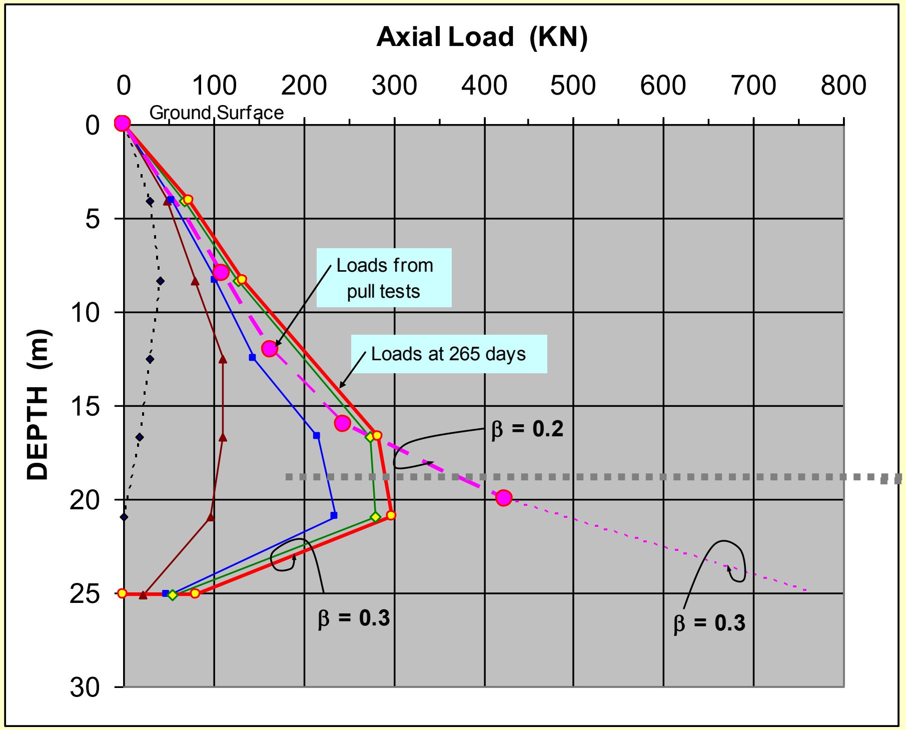
Measured distributions of axial load



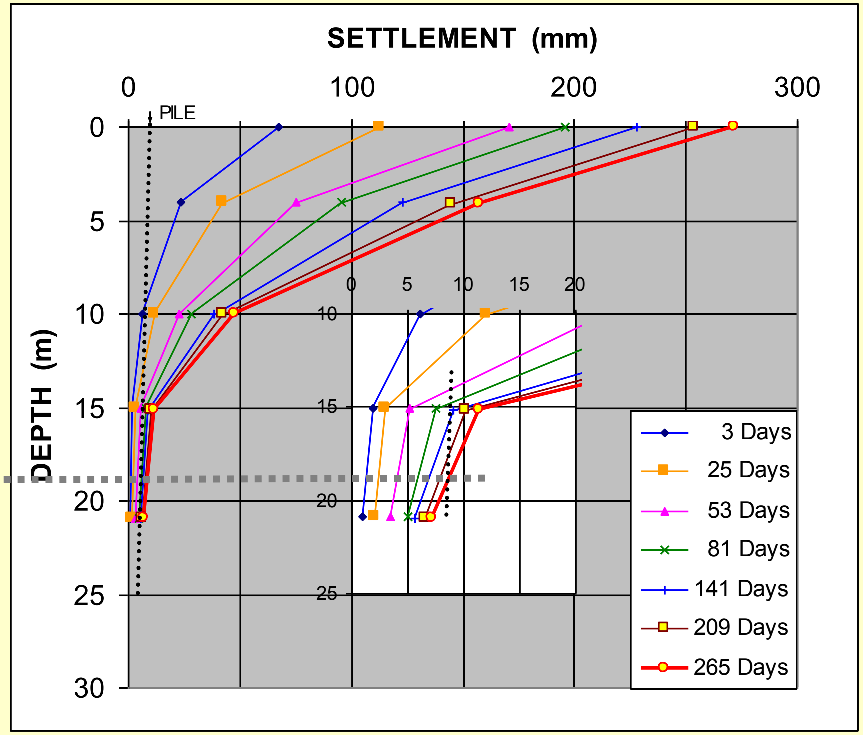


Data from Indraratna et al. 1992

The measurements compiled and put into the context of the basic soil response.



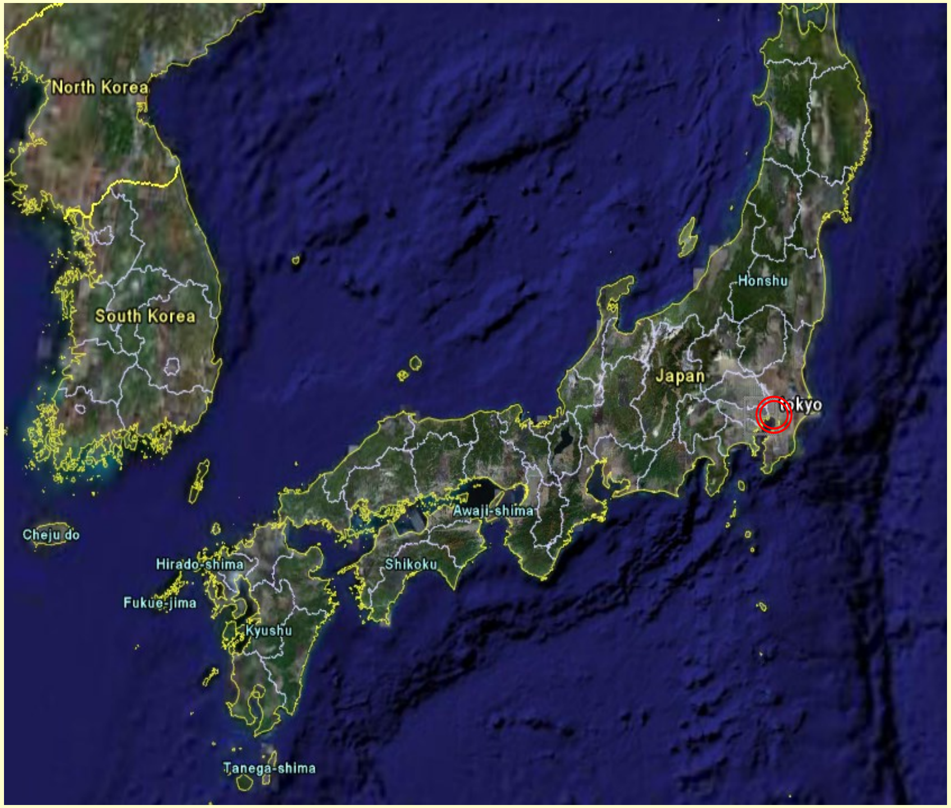
Measured and calculated load distribution



Settlement distribution

Data from Indraratna et al. 1992

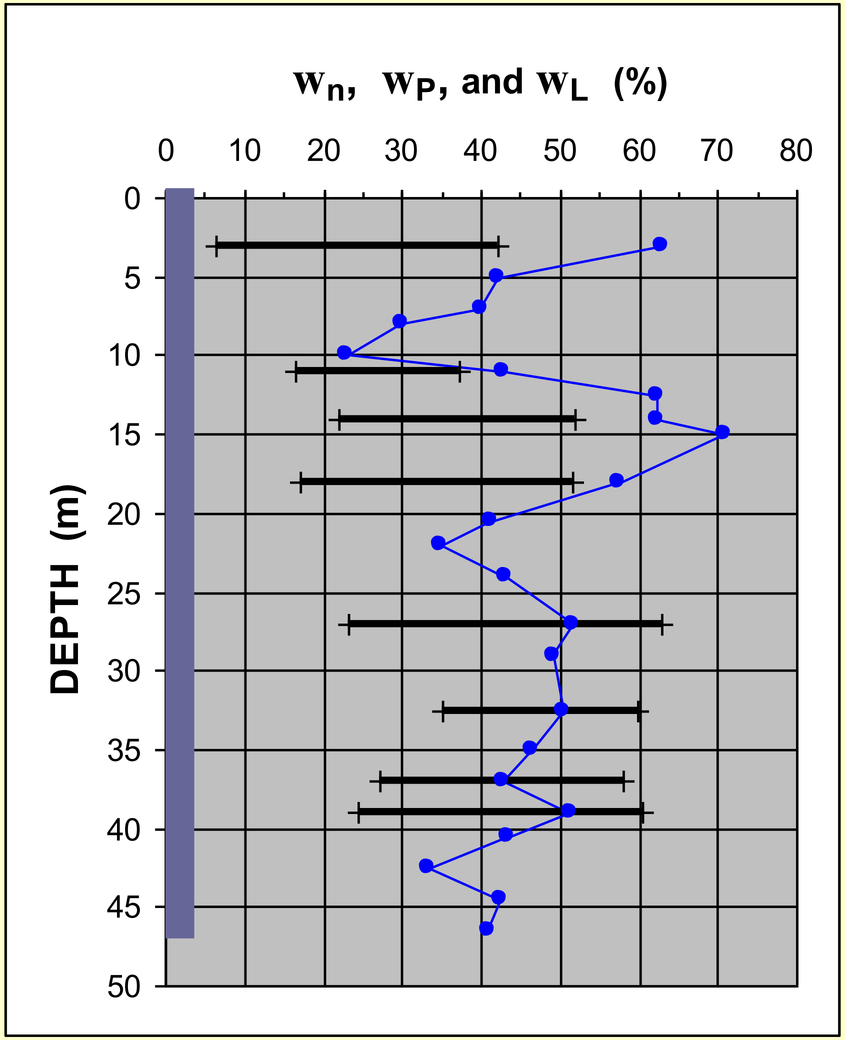
CASE #9



Okabe, T., 1977. Large negative skin friction and friction-free methods.

Proc. 9th ICSMFE, Tokyo, Vol.1, pp. 679 – 683.

CASE #9



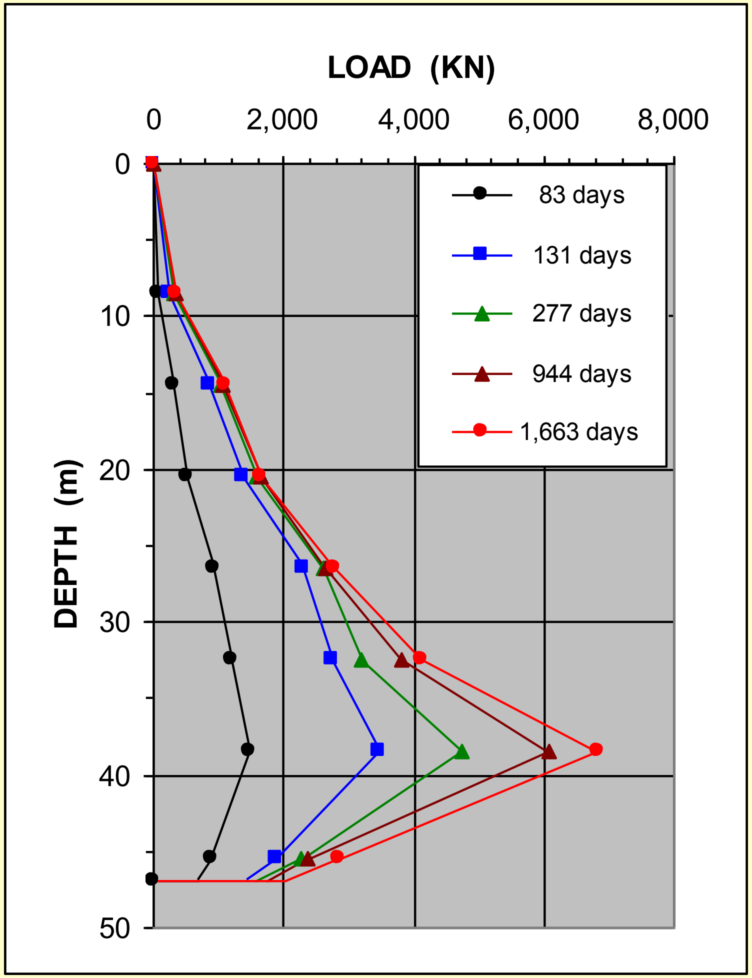
Strain-gage instrumented, 600mm diameter, pipe piles driven through silty clay and silt with silty sand.

A fill was placed on the ground (no information of fill height) over a vast area of the site and pumping of water at depth lowered the pore pressures at depth (no information on pore pressure distribution).

Data from Okabe (1977)

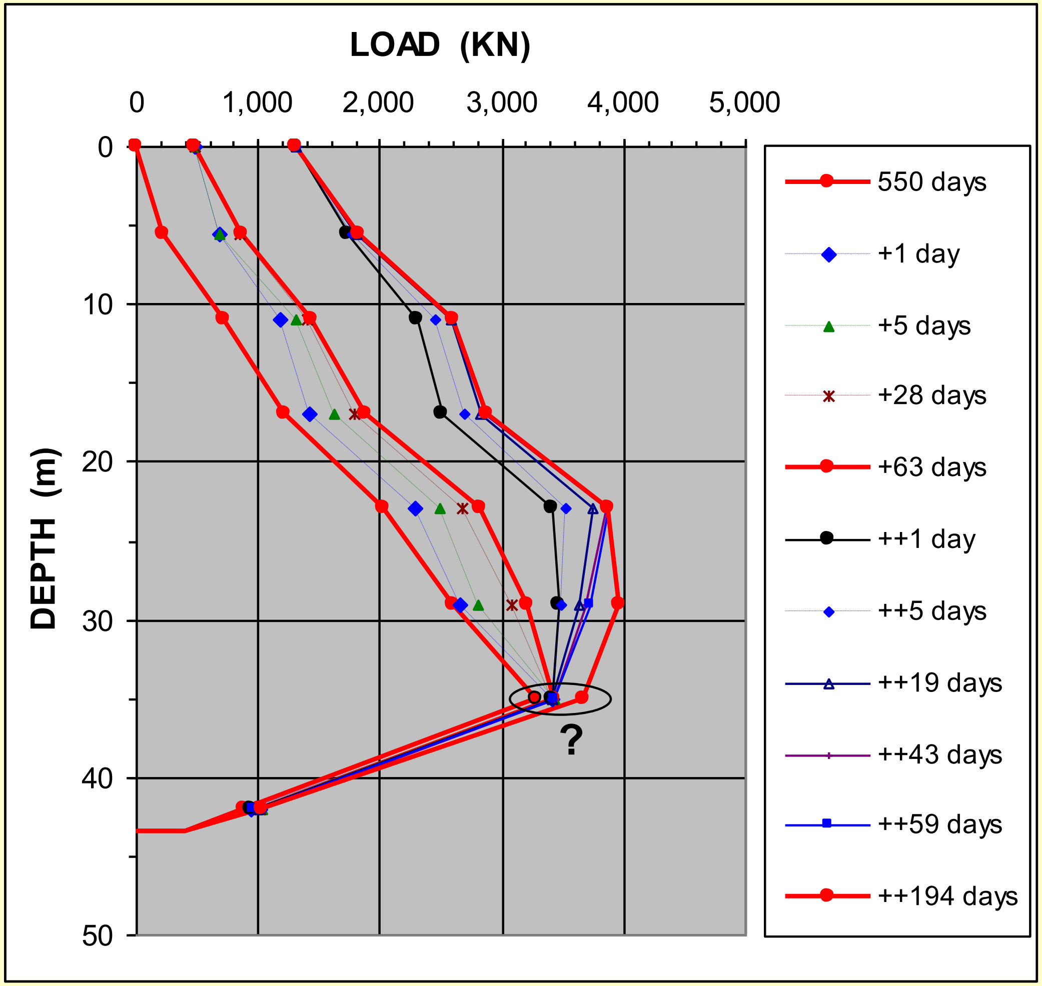
Test Pile #1 — Single Pile

Load distribution with time after driving



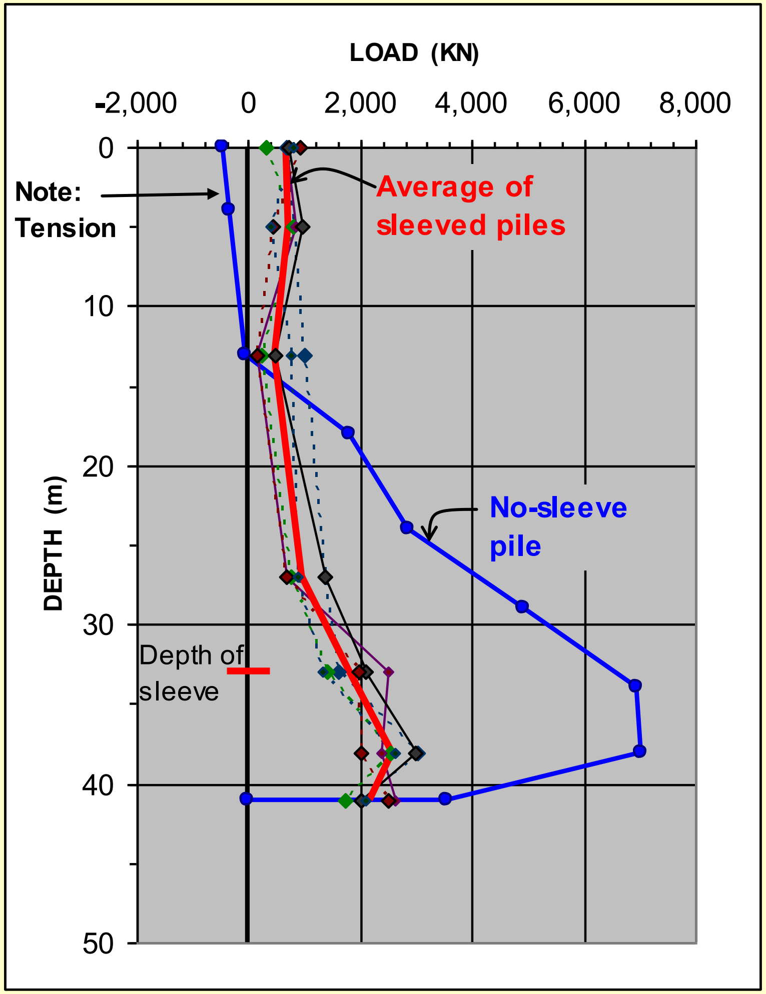
Test Pile #2 — Single Pile

Load distributions as loads (700 KN and 1,000 KN have been placed on the pile head

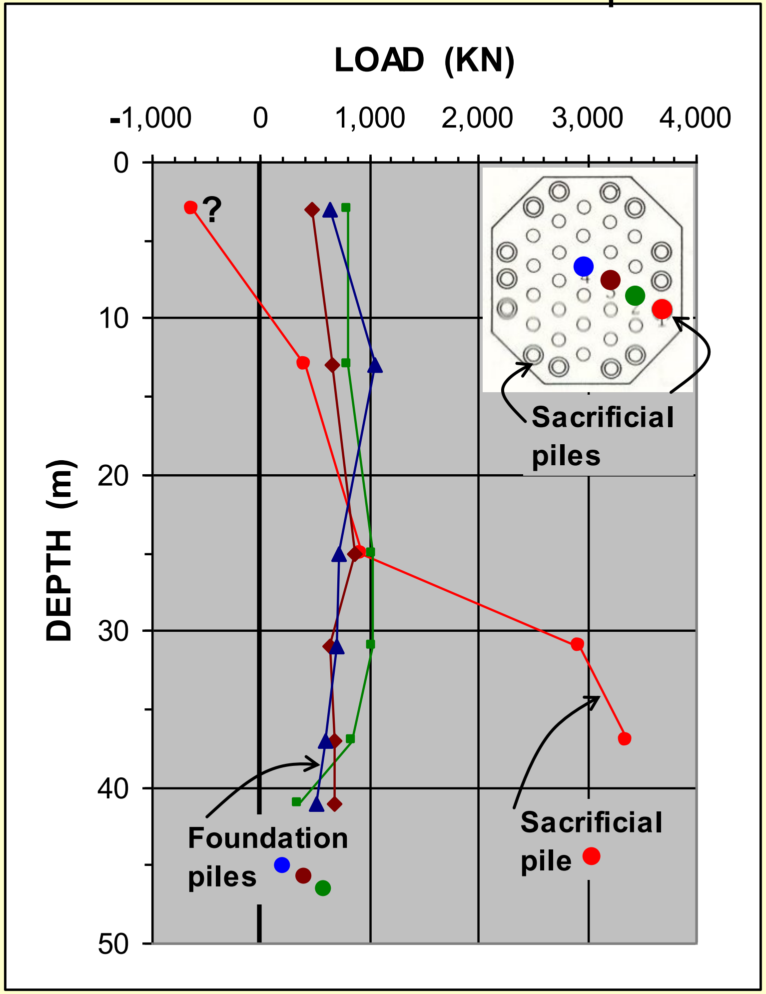


Data from Okabe 1977

Group of piles connected to a common cap
Four piles are "sleeved" one pile is not

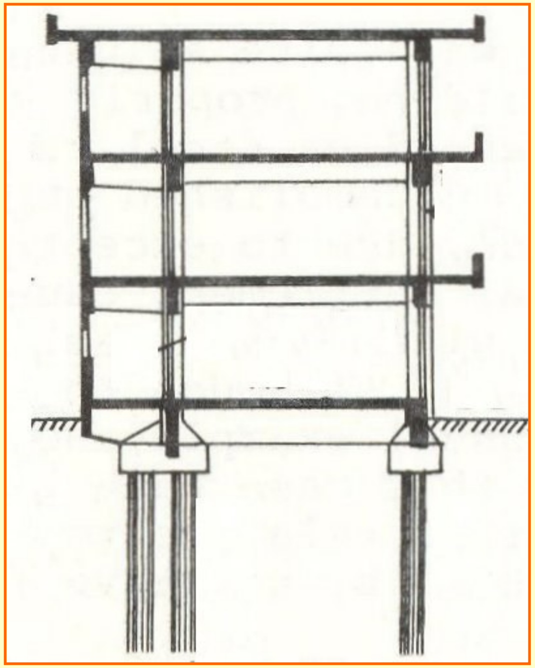


Group of piles connected to a common cap surrounded by "sacrificial" piles supposedly(?) not connected to cap



Data from Okabe 1977

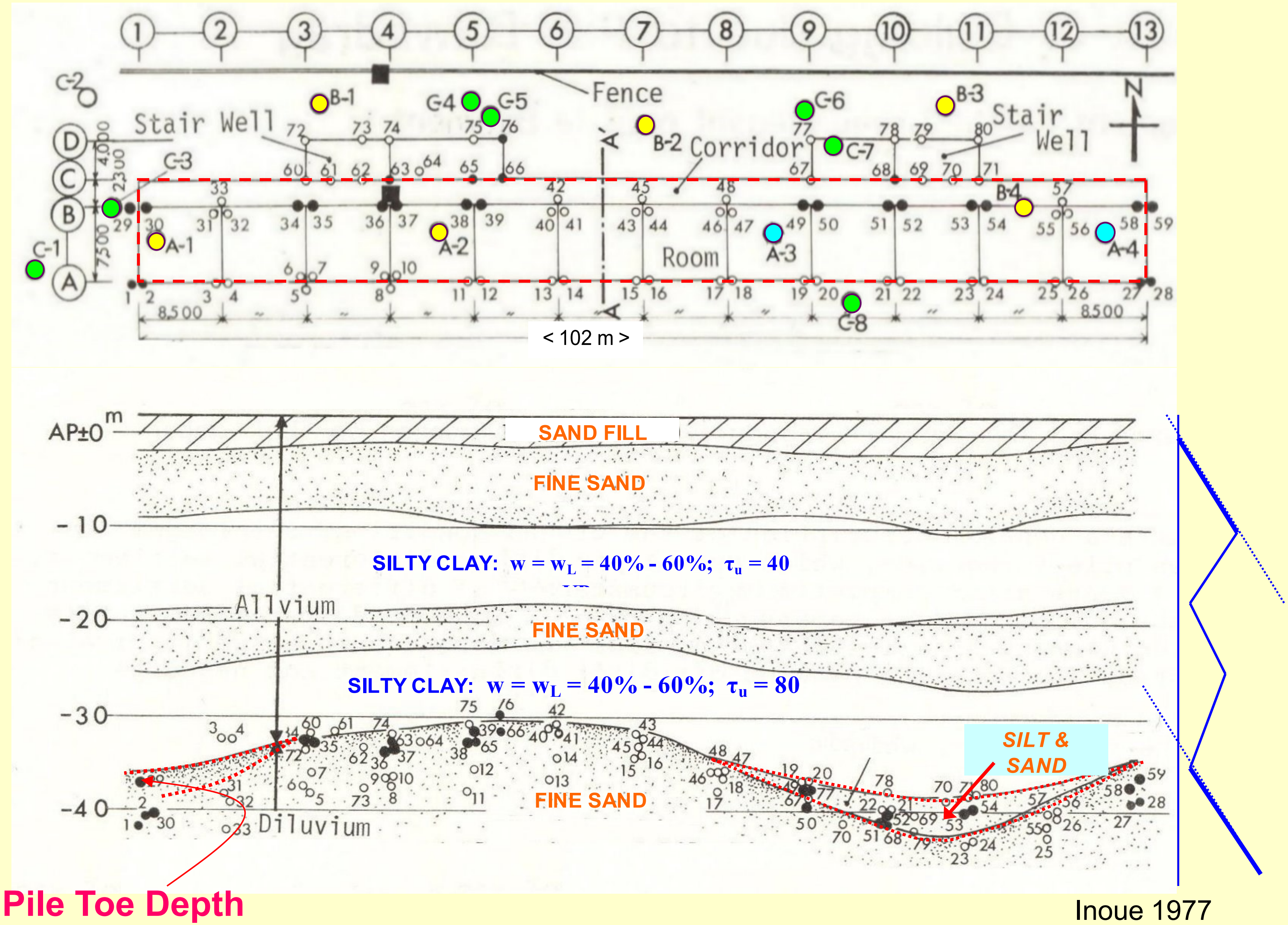
CASE #10



Inoue, Y., Tamaoki, K., Ogai, T., 1977. Settlement of building due to pile downdrag. Proc. 9th ICSMFE, Tokyo, Vol.1, pp. 561 – 564.

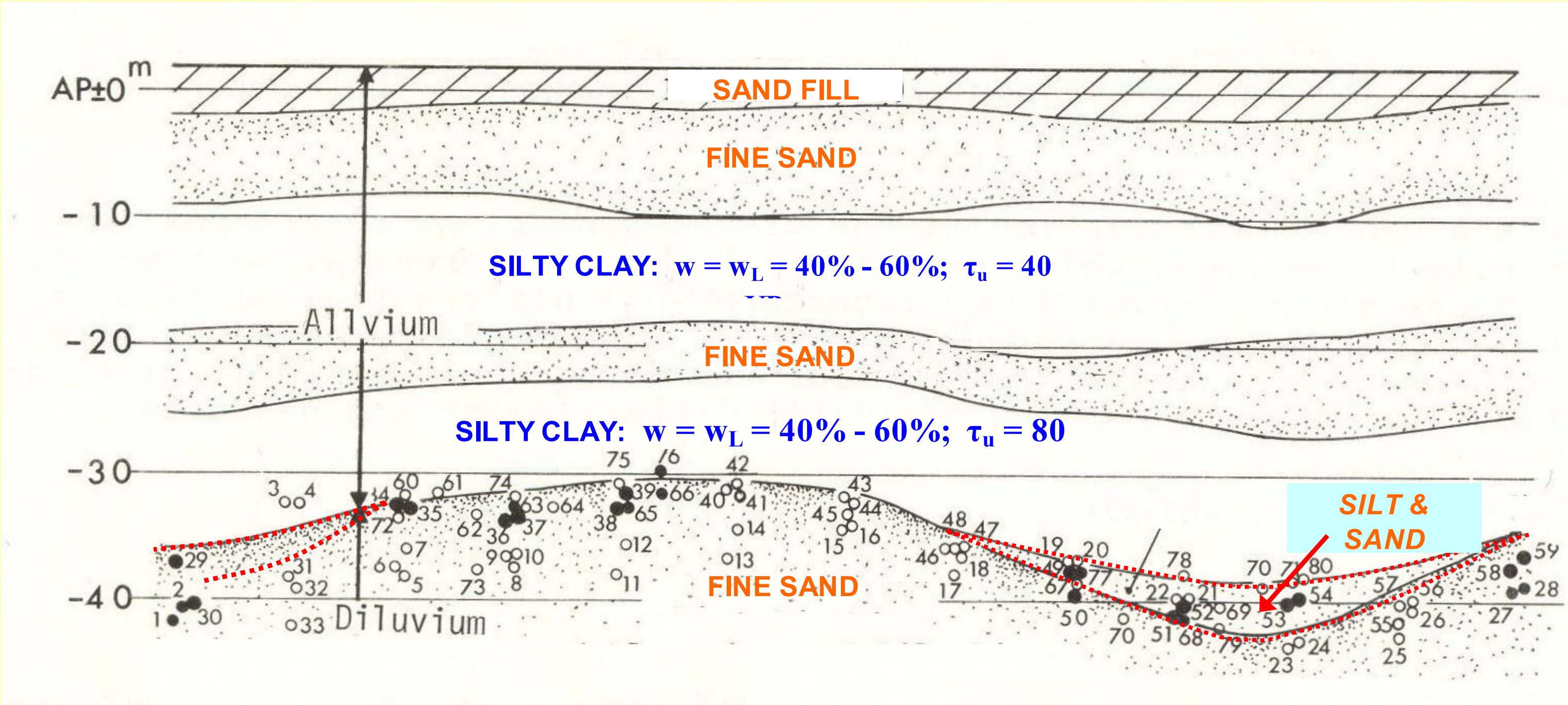
A Downdrag Case

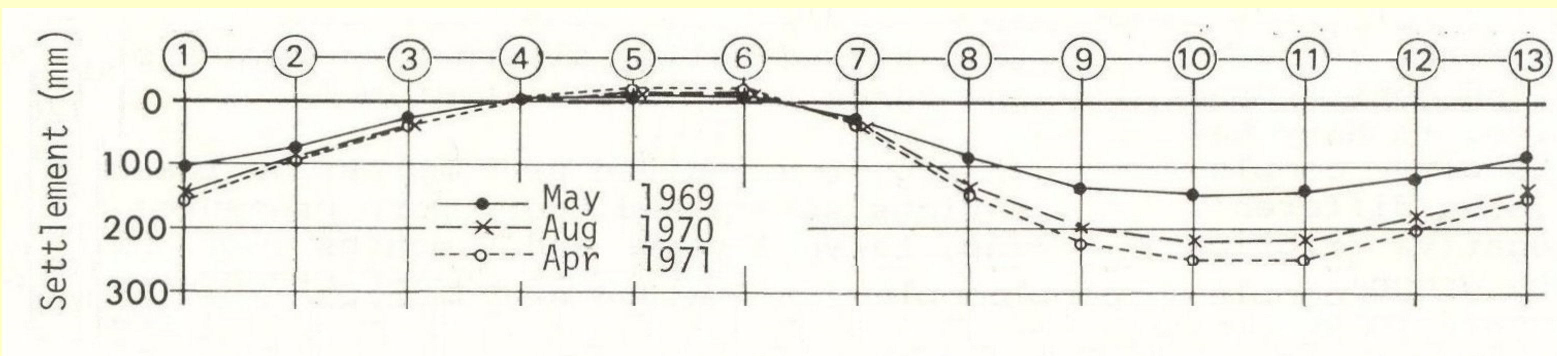
A three-storey building with a foot print of 15 m by 100 m founded on 500 mm diameter open-toe pipe piles driven through sand and silty clay to bearing in a sand layer at about 35 m depth. The piles had more than adequate capacity to carry the building. Two years after construction, the building was noticed to have settled some 150 mm. Measurements during the following two years showed about 200 mm additional settlement. The building was demolished at that time.



Fence
Stair Well
Corridor
Room
< 102 m >
AP±0 m
SAND FILL
FINE SAND
SILTY CLAY: w = w_L = 40% - 60%; τ_u = 40
Allvium
FINE SAND
SILTY CLAY: w = w_L = 40% - 60%; τ_u = 80
FINE SAND
Diluvium
SILT & SAND
Pile Toe Depth

Inoue 1977

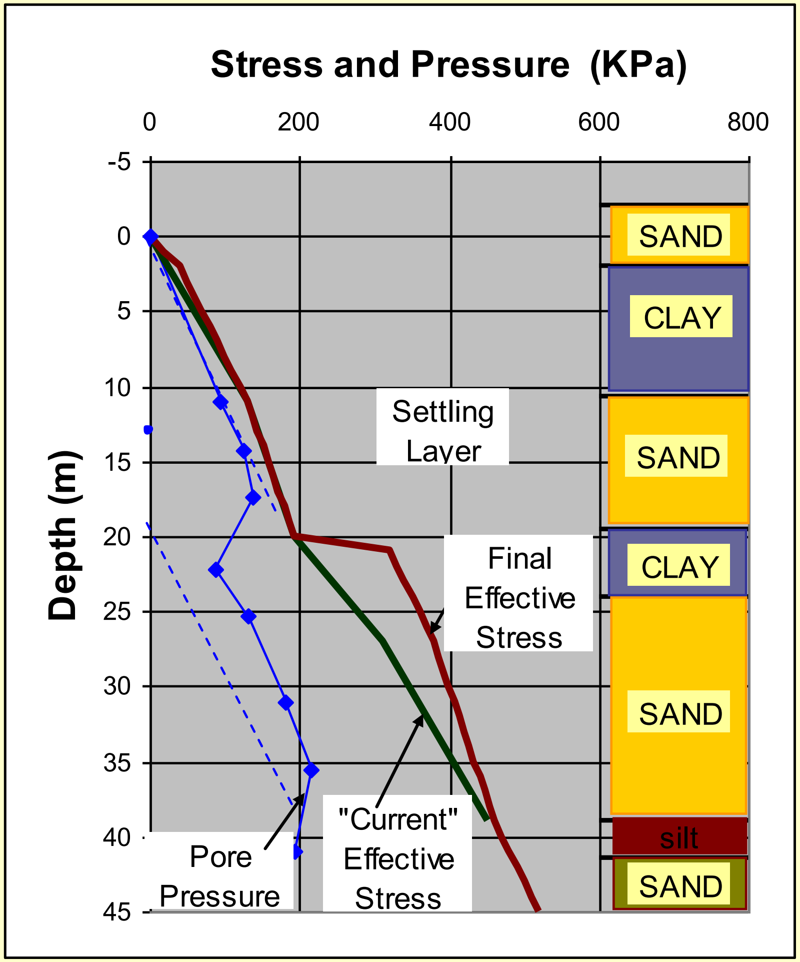


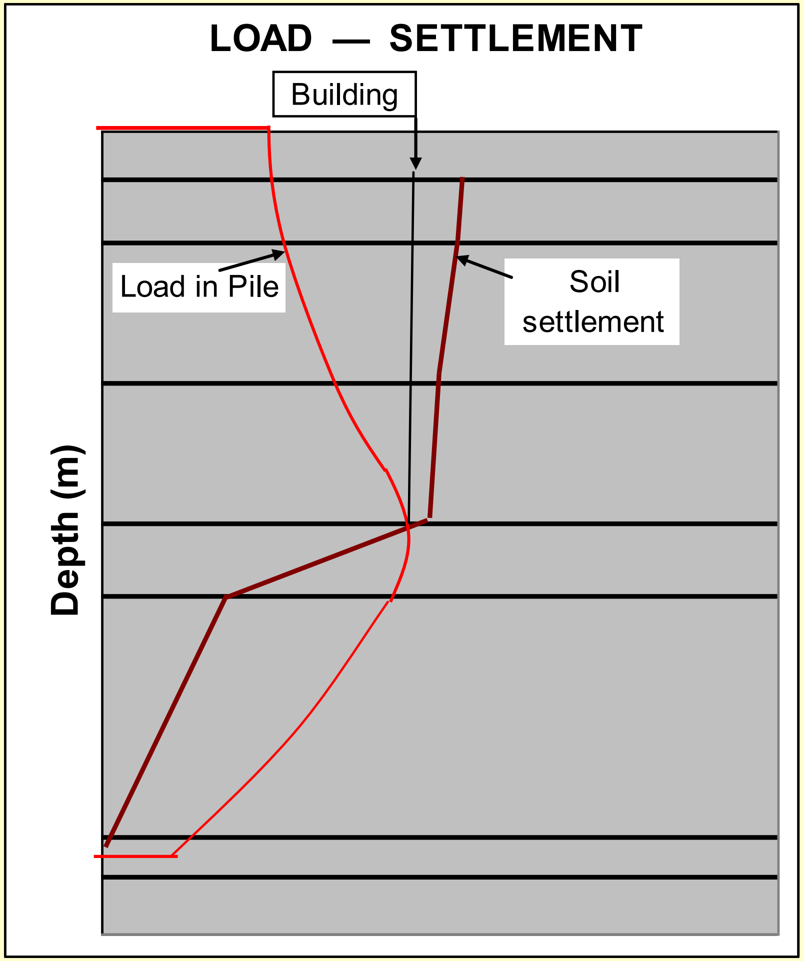


Settlement between piles in Row 6 and Row 10 from Sep. 1967 through May 1969 = 150 mm.

Slope ≅ 1 : 100 (Sep 67 Apr 71)

Inoue 1977





Speculative distribution

Data from Inoue 1977

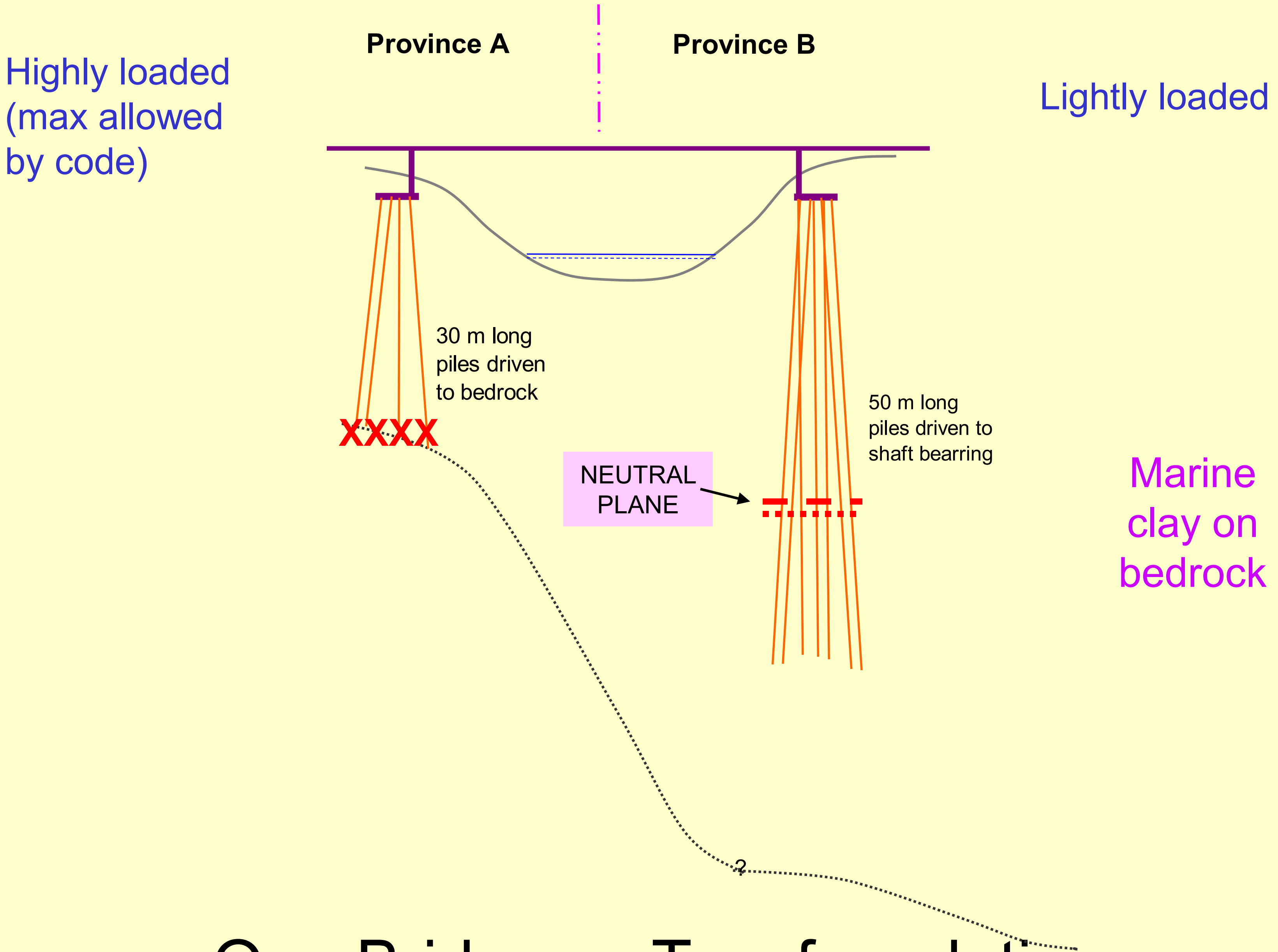
Two case histories

on

Damaging Drag Load

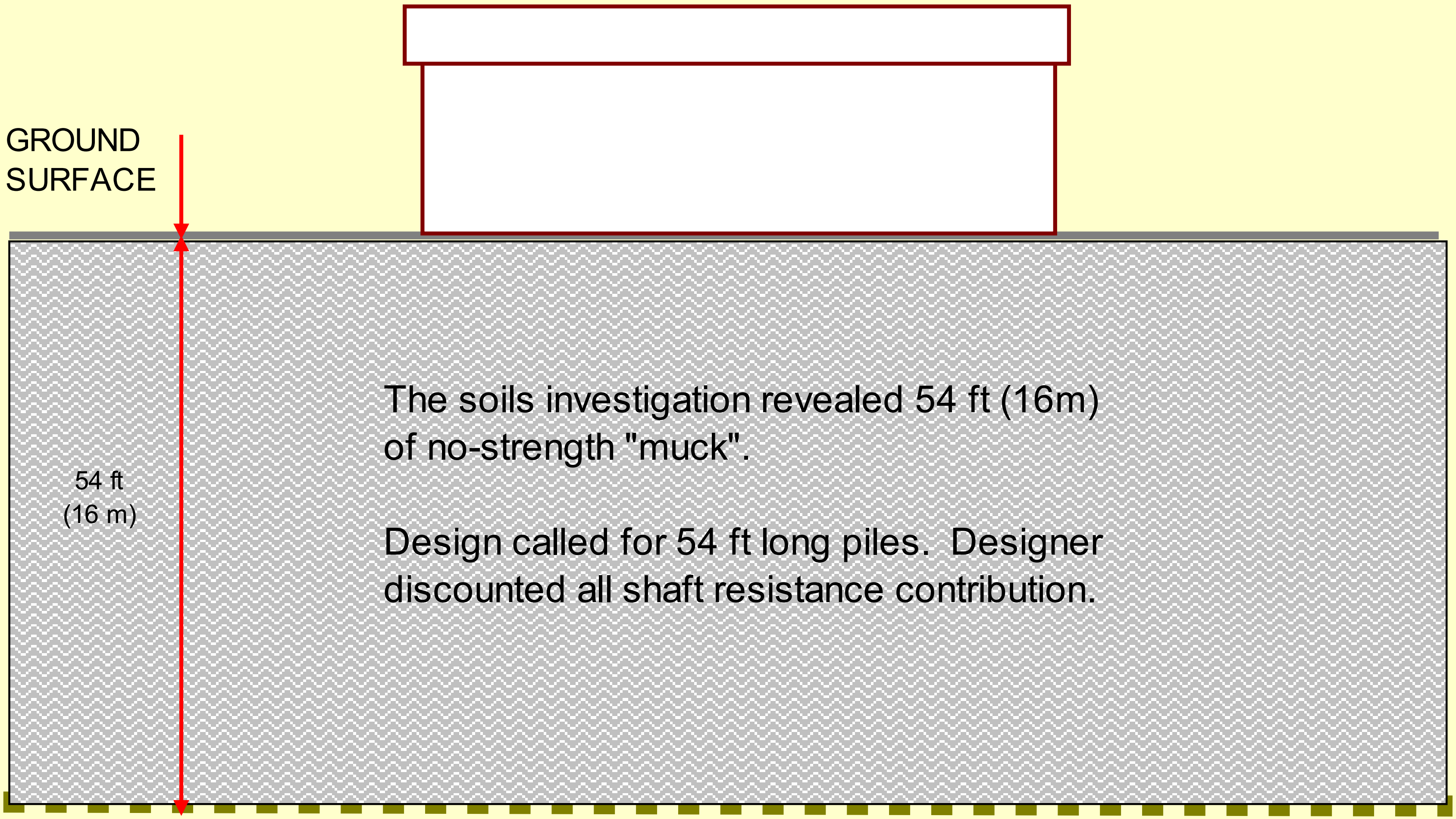
and

Damaging Downdrag



One Bridge — Two foundations

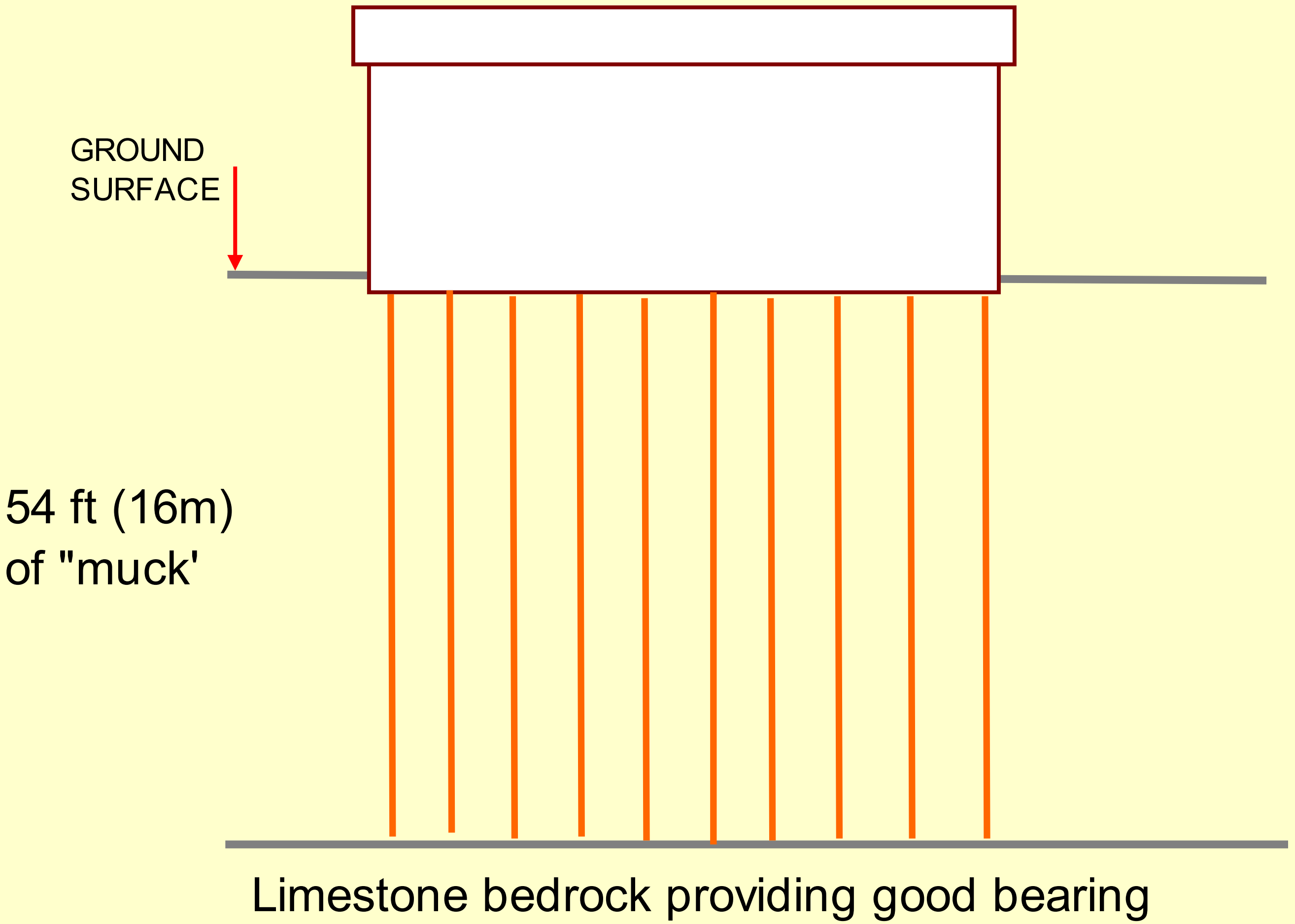
A CASE HISTORY OF A STRIP-MALL FOUNDED ON PILES



The soils investigation revealed 54 ft (16m) of no-strength "muck".

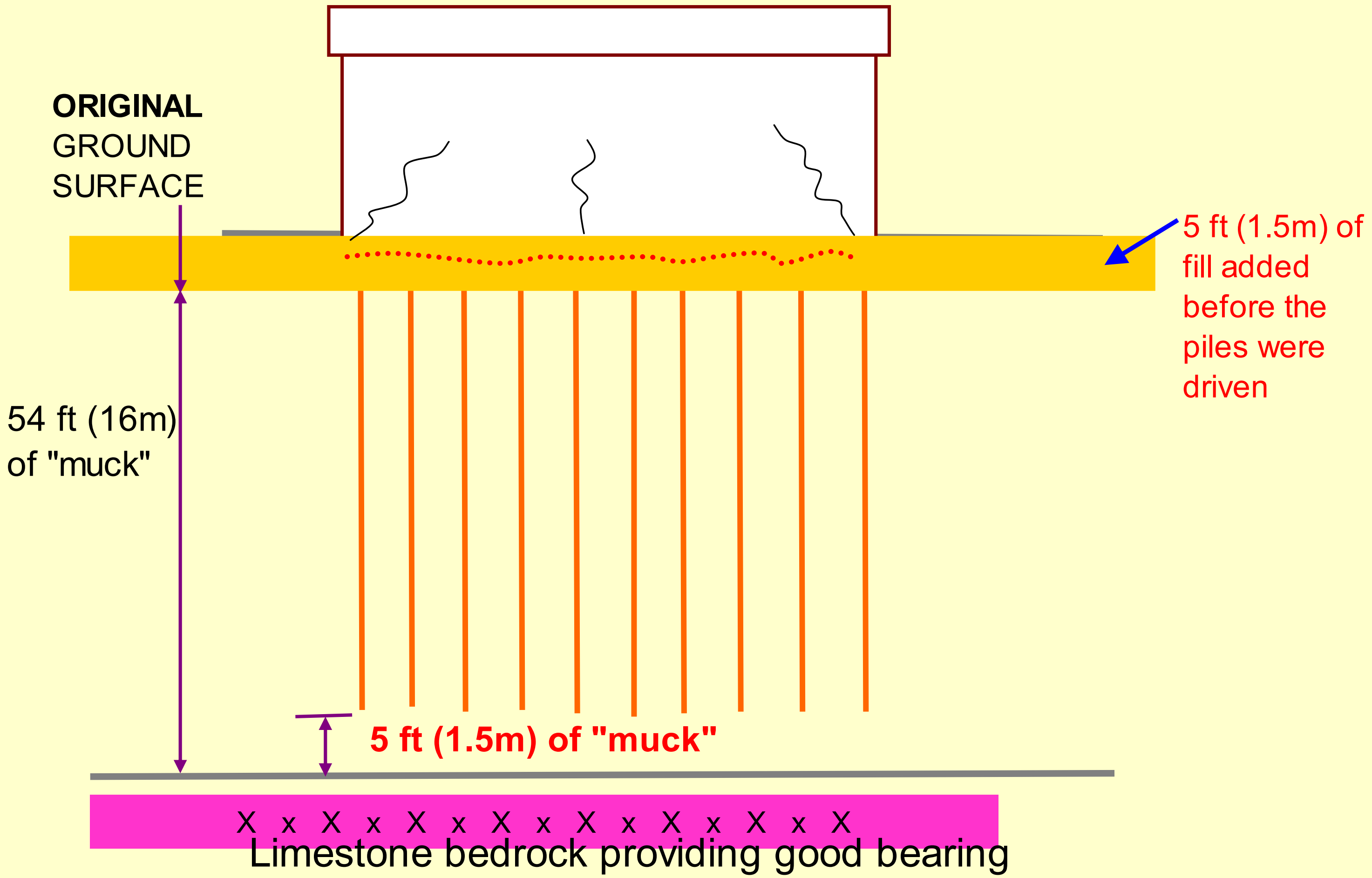
Design called for 54 ft long piles. Designer discounted all shaft resistance contribution.

Limestone bedrock providing good bearing



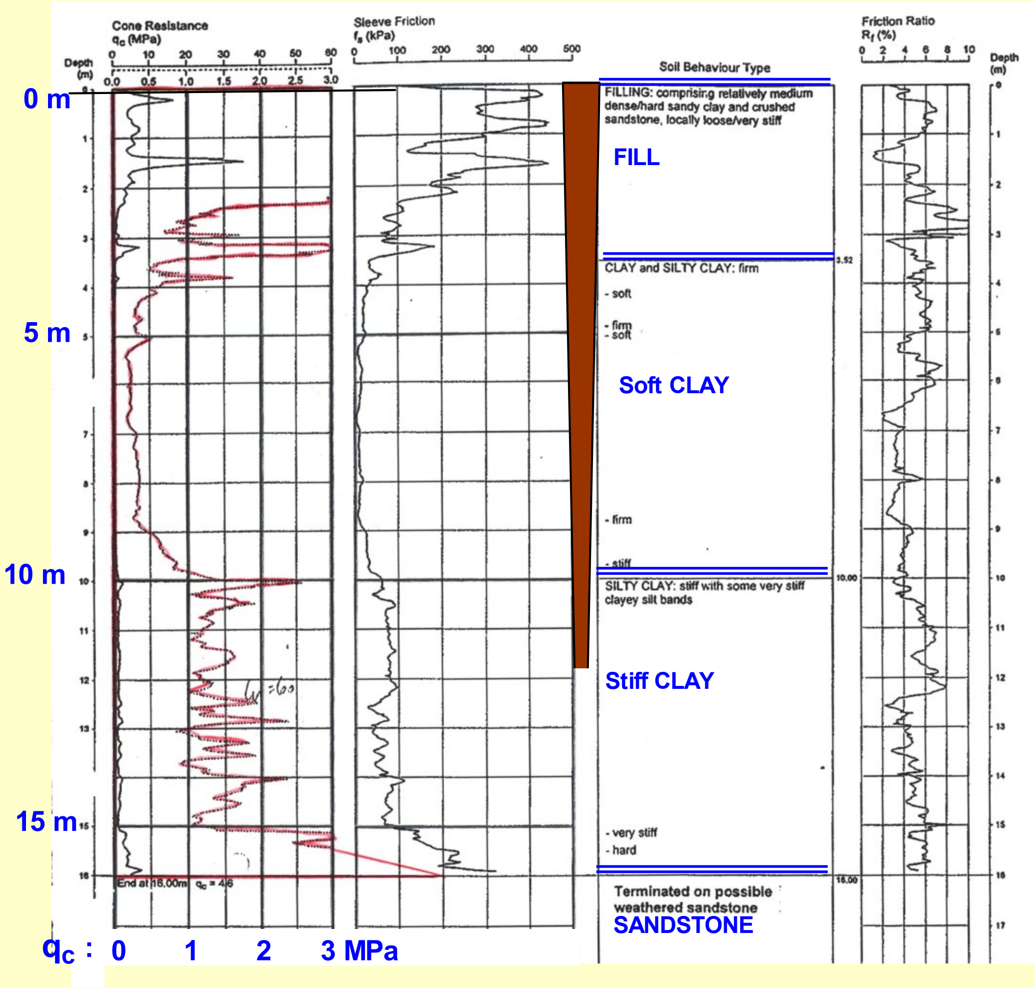
Strip-Mall as designed

A real DOWNDRAG case



The quoted case histories and several others have convincingly shown:

- Effective stress governs the mobilization of shaft shear between a pile surface and the soil (negative skin friction as well as positive shaft resistance).
- Very small movement is all that is needed to fully mobilize the shaft shear.
- A pile is a stiff member in a soft medium and there will always be a stress transfer between the two. That is, **a neutral plane will <u>always</u> develop, there will <u>always</u> be negative skin friction accumulating to drag load (residual load) in equilibrium with the positive shaft resistance, and the magnitude of the pile toe resistance will be as determined by the magnitude of movement-interaction between the pile and the soil**.
- The forces are often not fully developed or may take a long time to develop.
- Negative skin friction and drag load is a good thing, as it prestresses the pile (provided the pile structural strength can accept the dead load plus drag load).
- The matter of concern is "**downdrag**", that is the settlement of the pile from the soil settling around the pile dragging it down.



Office Building Extension in Brisbane, Australia

View toward the roof

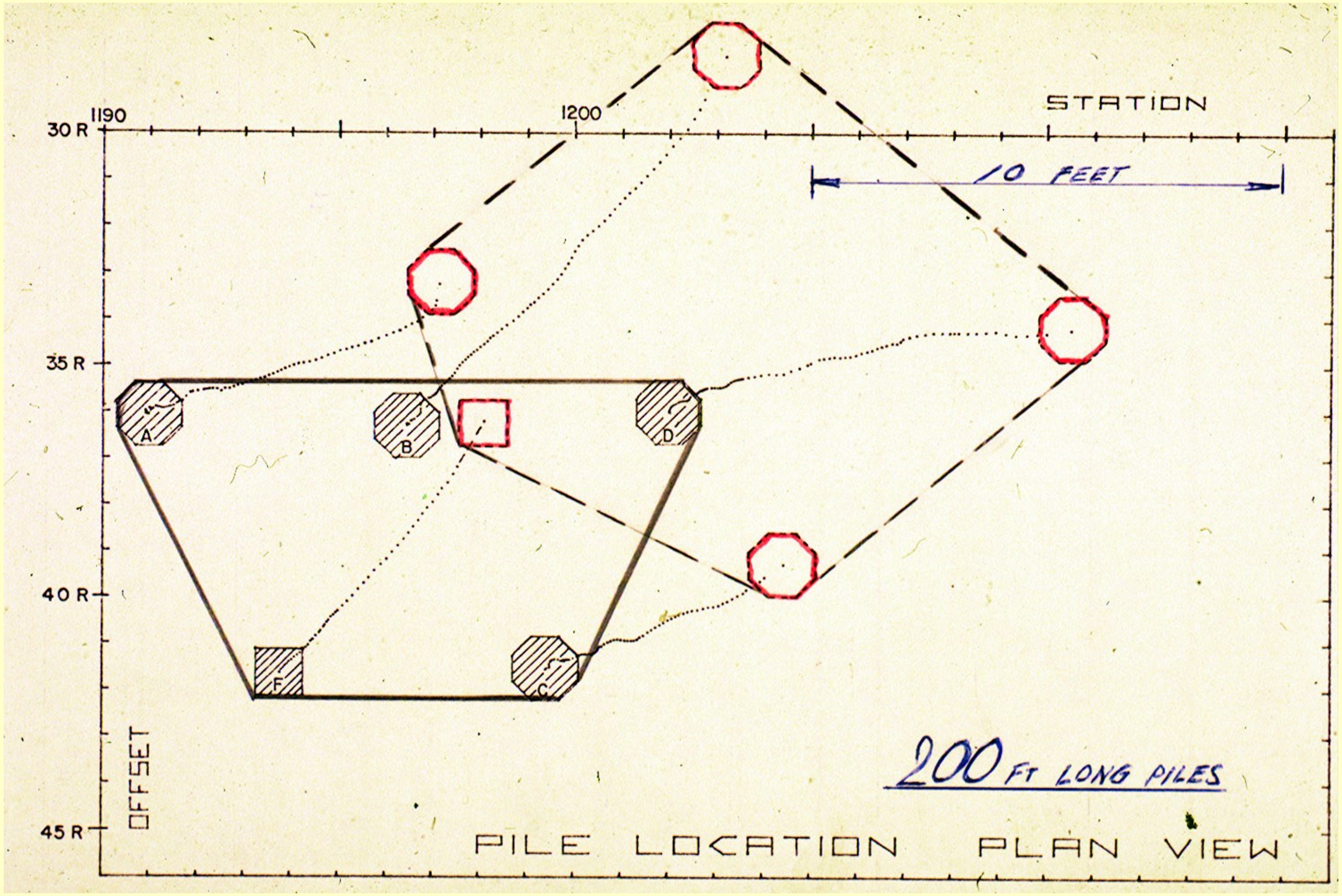
Foundations and underpinning

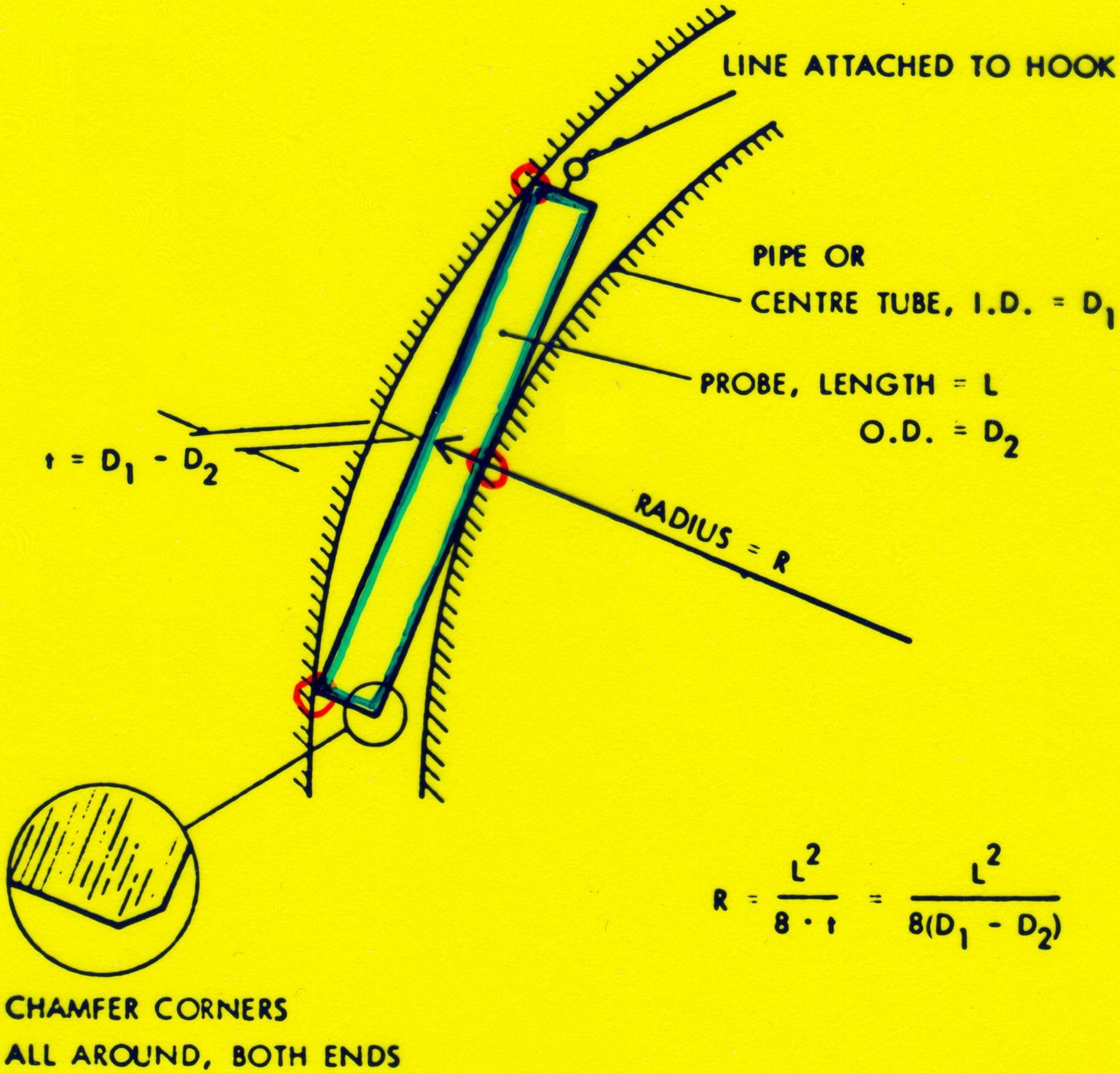
The original piles had a capacity about three times the applied load, but downdrag got the better of them. Luckily, new piles could be installed (driven to the sandstone; for some columns to a“four-for-one” ratio).

$ $ $ $

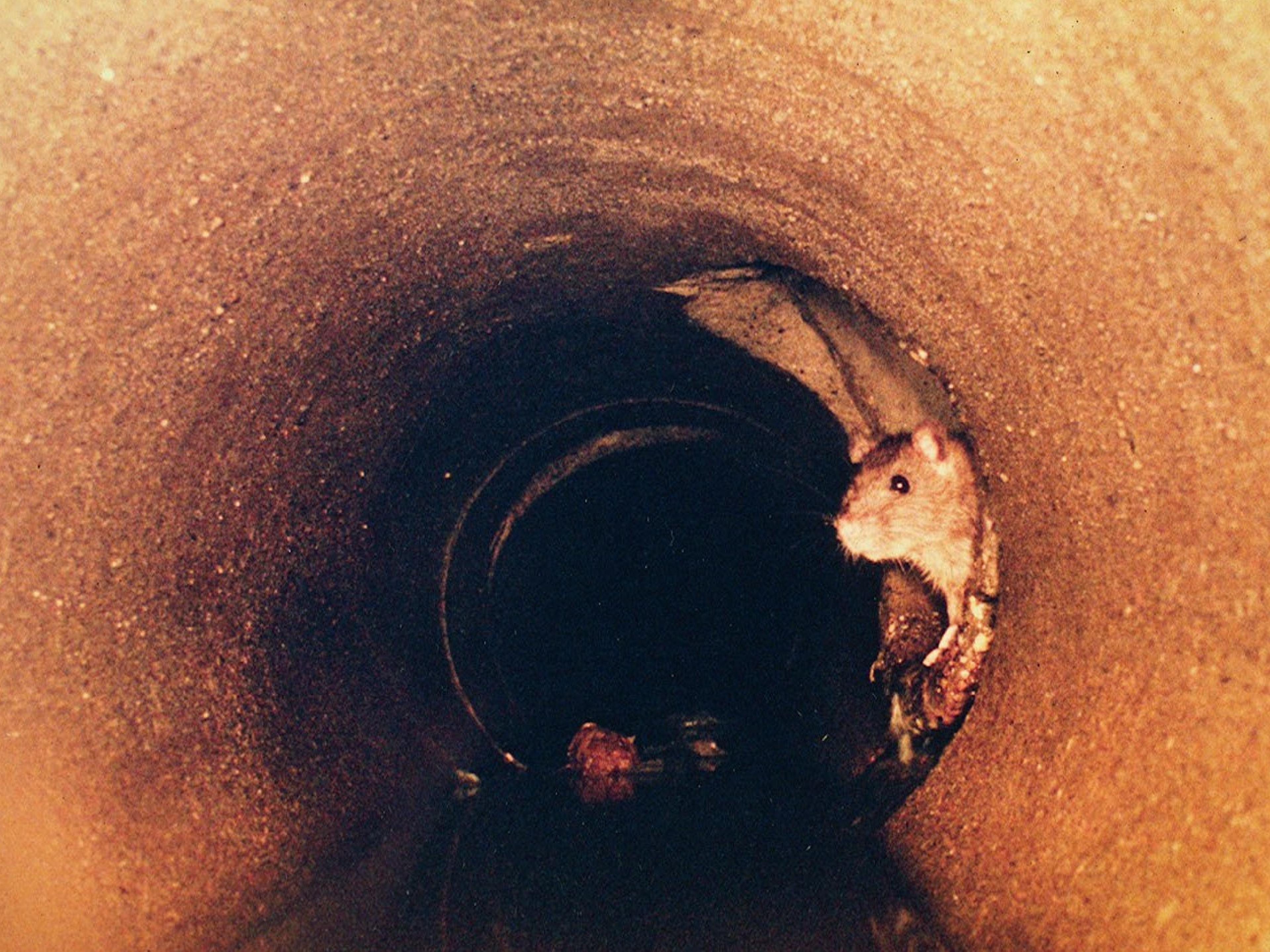

7.25.2003

Piles will bend and drift





$$R = \frac{L^2}{8 \cdot t} = \frac{L^2}{8(D_1 - D_2)}$$

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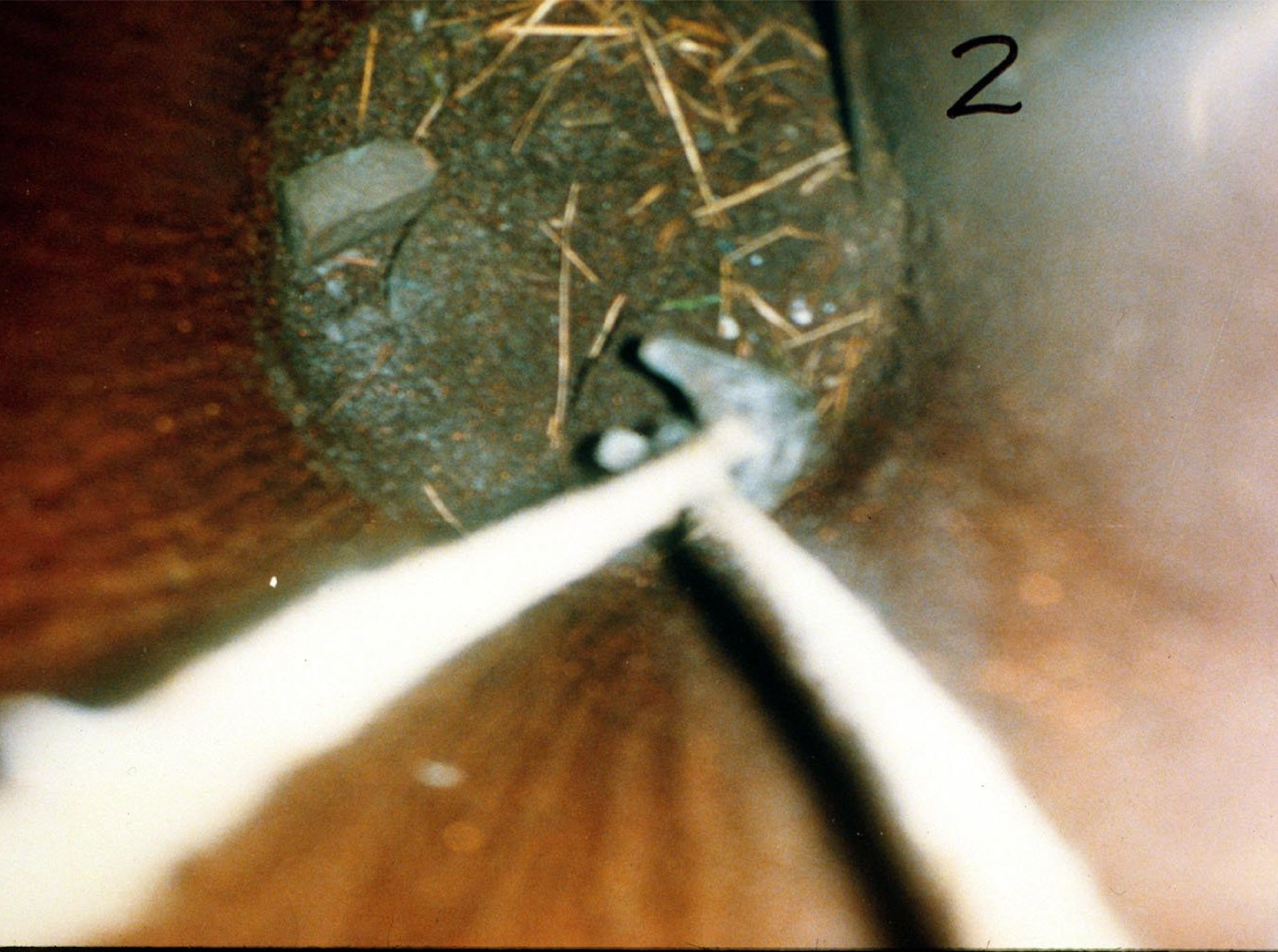
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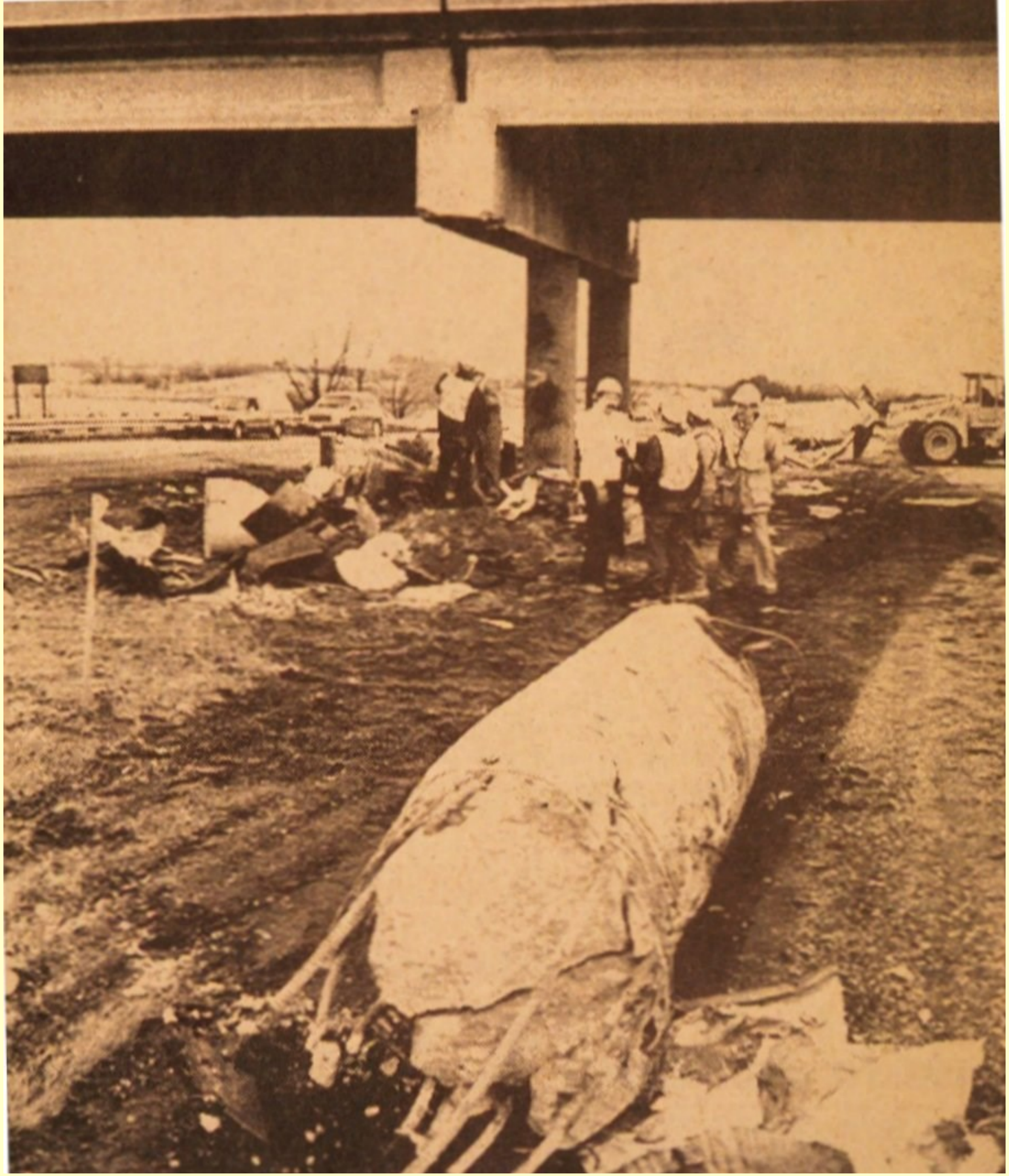
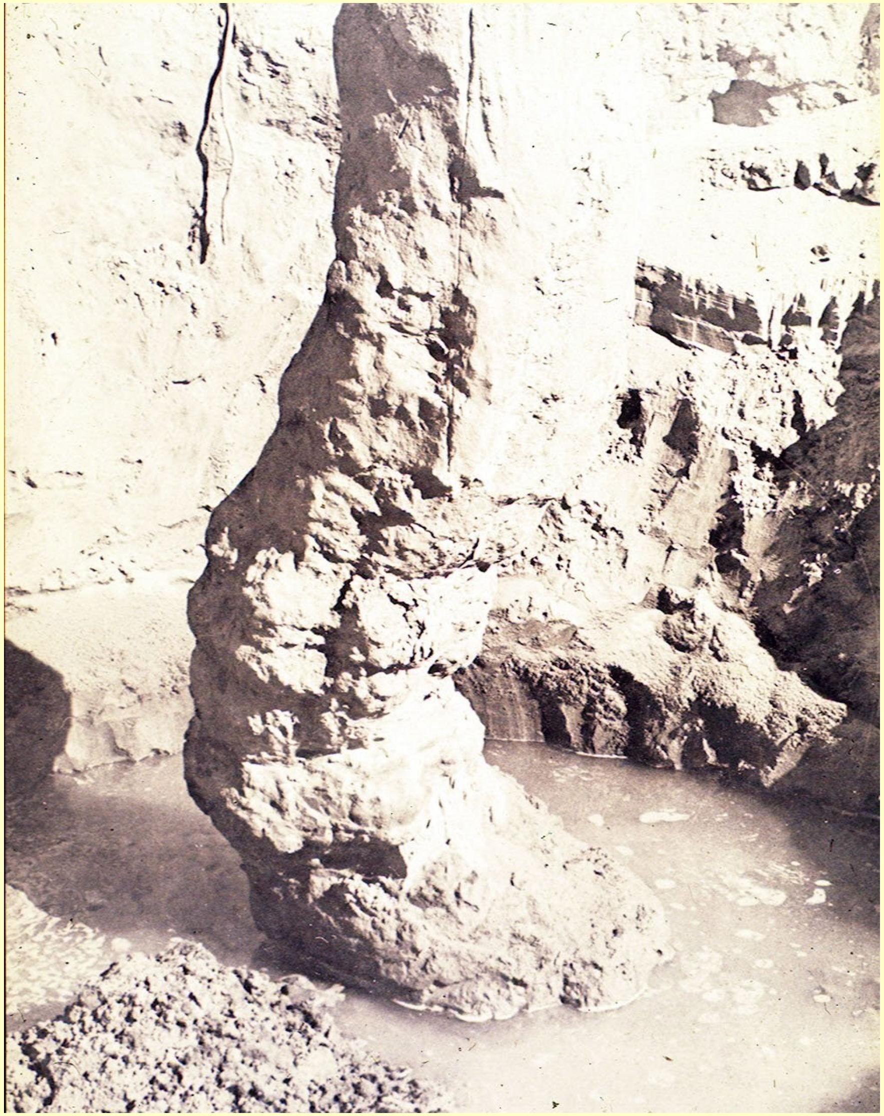


Bridge stops tractor-trailer at overpass near Milton on HWY401

The Ottawa Citizen
November 30, 1990



Lessons learnt

- Dragloads can be very large.
- Negative skin friction develops also at minimal relative movement between the pile and the soil. That is, piles will always experience drag load whether the settlement on the ground is large or small — or not noticeable.
- The shear force between a pile and the soil is governed by effective stress. Total stress (“undrained shear strength”) has little relation to the shear forces.
- A force equilibrium (neutral plane) develops between negative direction and positive direction forces (loads)
- Bitumen coat is effective in reducing the shear force.

Lessons learnt

- A Neutral Plane develops where the negative direction forces are in equilibrium with the positive direction forces, which is where the shaft shear changes from negative skin friction changed to positive shaft resistance.
- The Neutral Plane is also where the pile and the soil move (settle) equally, i.e., where there is no relative movement between the pile and the soil.
- The settlement of the pile head (the pile cap, the piled foundation) is the settlement of the pile(s) at the Neutral Plane plus the ‘elastic’ shortening of the pile for the load.

Lessons learnt

- Live load does not combine with drag load
- The negative skin friction (and positive shaft resistance) only needs at most a few millimeter of movement to become fully mobilized. That is, be the ground settlement large or small, the drag load will be about as large.
- Moreover, practically all piles will have drag load.