Pitfalls and fallacies in foundation engineering design

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Amsterdam

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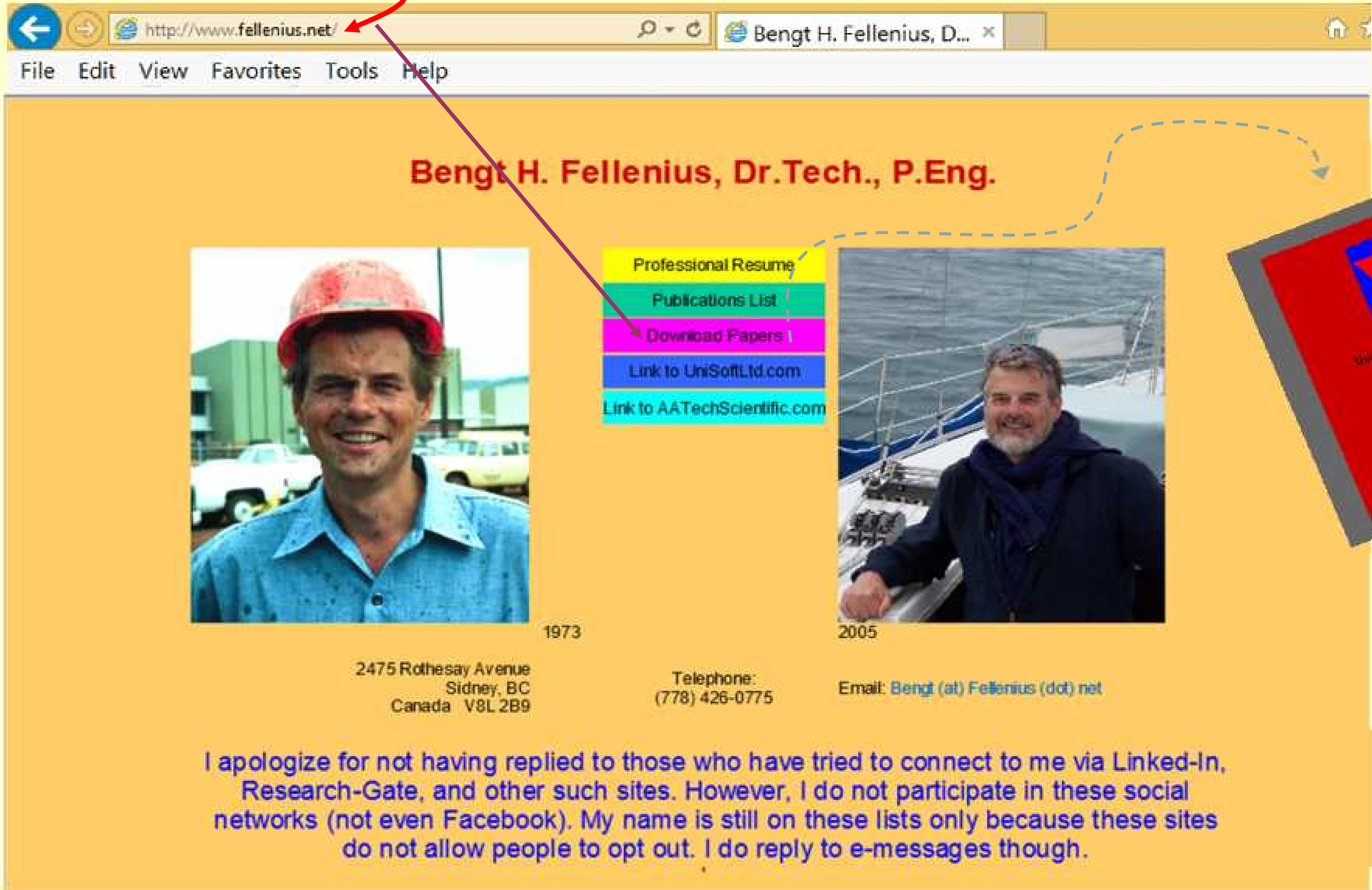
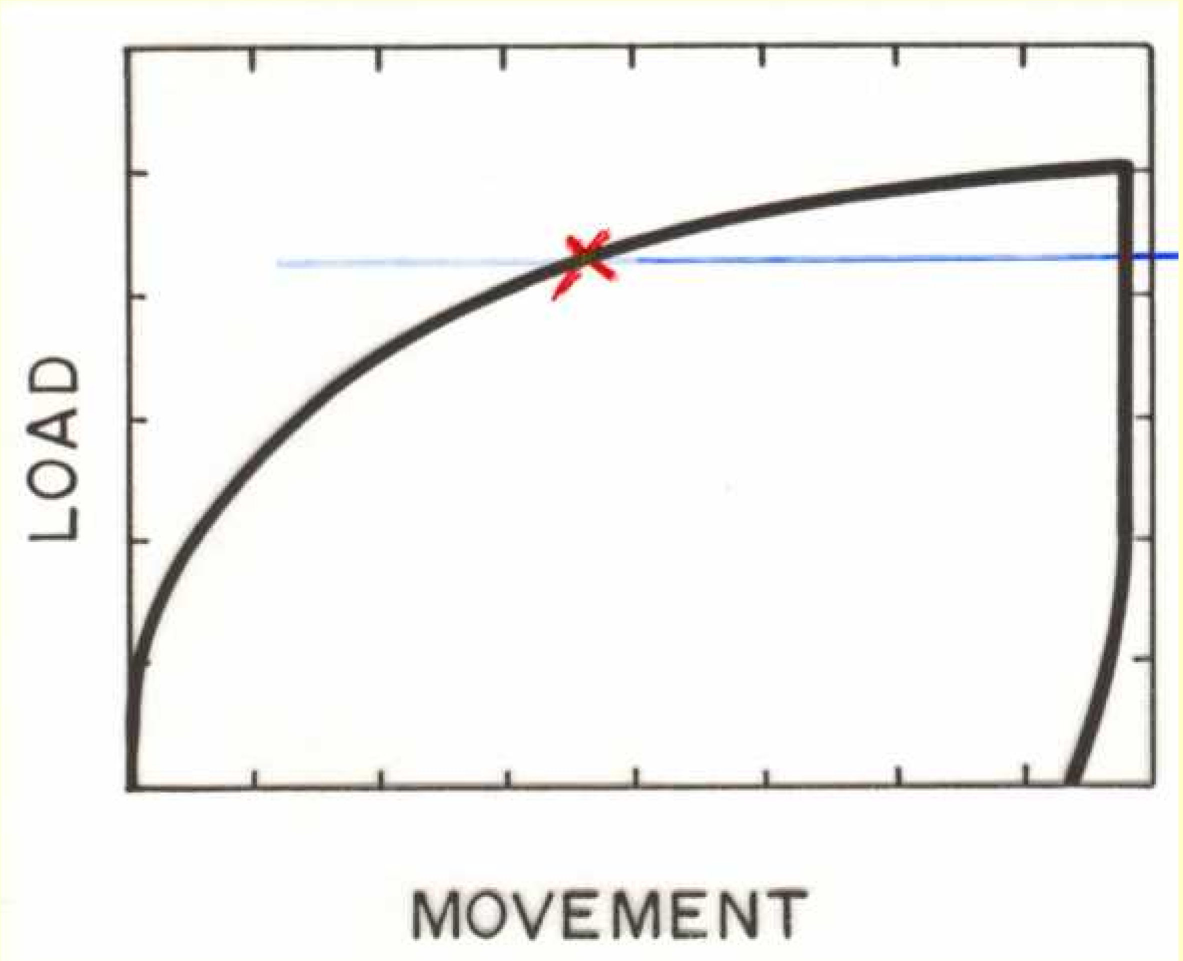
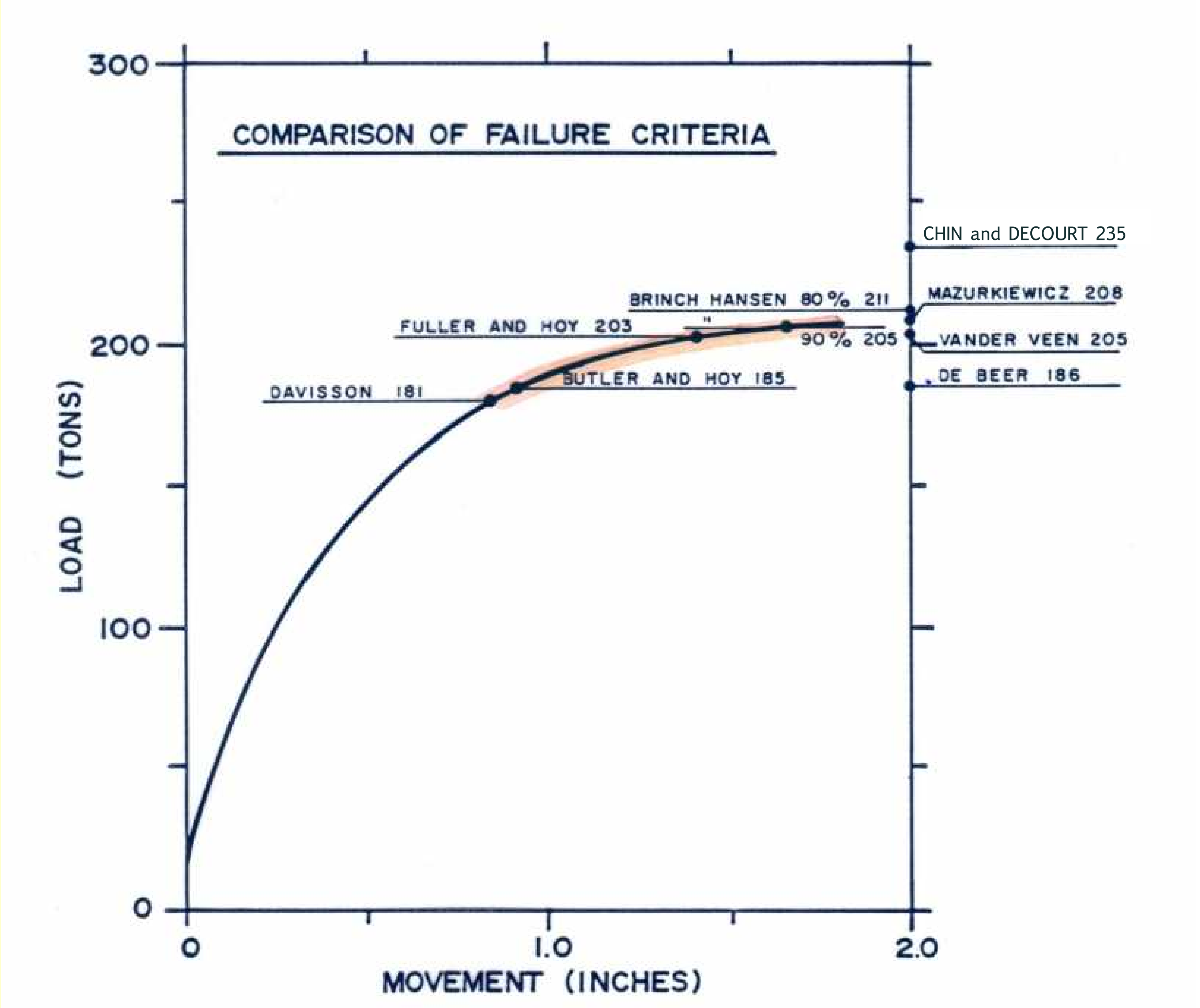


The primary base for the design of a piled foundation is the pile capacity as determined in a static loading test. A routine static loading test provides the load-movement of the pile head...

and the pile capacity?




COMPARISON OF FAILURE CRITERIA
CHIN and DECOURT 235
BRINCH HANSEN 80% 211
MAZURKIEWICZ 208
FULLER AND HOY 203
90% 205
VANDER VEEN 205
DAVISSON 181
BUTLER AND HOY 185
DE BEER 186
300
200
100
0
LOAD (TONS)
0
1.0
2.0
MOVEMENT (INCHES)

Some people consider the capacity to be the load applied to the pile head*) that caused a movement equal to 10 % of the pile head diameter. This is usually claimed to be recommended by Terzaghi. He knew better than that!

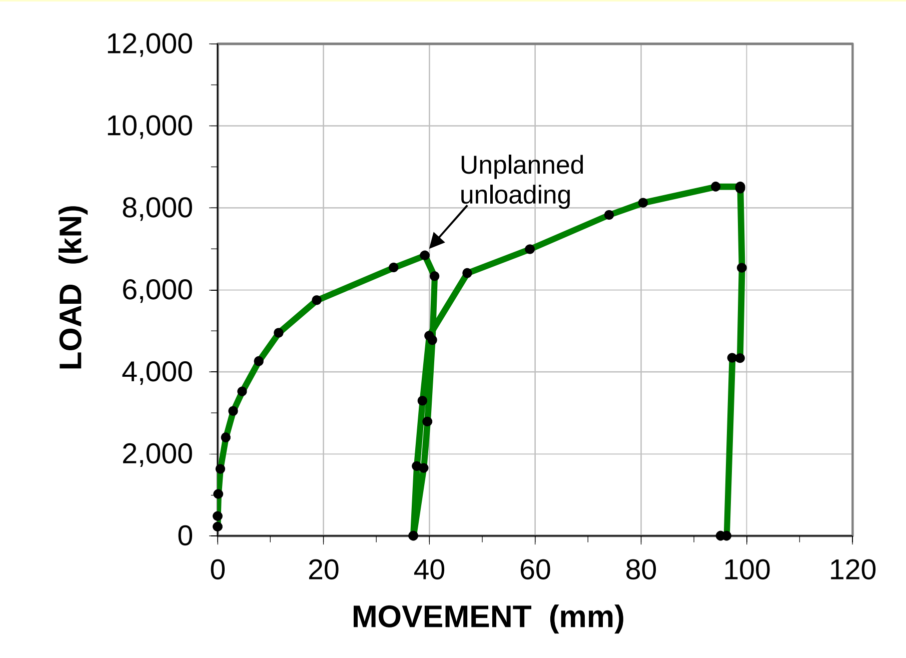
Terzaghis's recommendation, and he did give one (Terzaghi 1942), was this: ***Do not try to estimate pile capacity unless the pile toe movement is equal to at least 10 % of the pile toe diameter***. I think you agree that this is a very different recommendation. I have tried to find out where the misquote started and it seems to have been first put forward by E. DeBeer in the early 1950's and then picked up from there by several of the "Old Masters".

Terzaghi was referring to pile with diameters of about 300 mm. I have seen the misquoted definition applied to piles with a diameter larger than 1.5 m! At times even to footings! The pile diameter, let alone the pile head diameter, has nothing to do with a pile capacity. It is sad that a few codes and standards have designated this silly and ignorant definition as the one to use, notably the EuroCode.

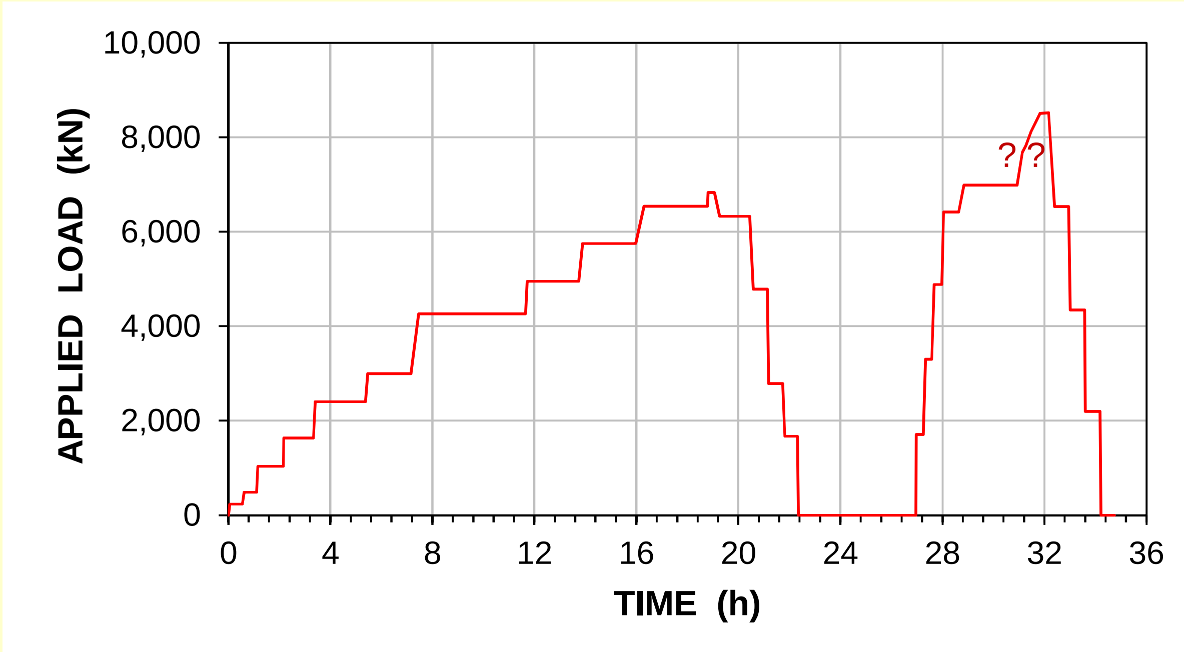
*) EuroCode has now (2016) changed to refer the 10-% movement to the pile toe diameter. I do not think this is an improvement.

Araquari Prediction Event, IFCEE 2015, Brazil. Test results

Actual pile-head Load-Movement
of the Araquari Prediction Event



Actual Load-Time Schedule of the
Araquari Prediction Event



Actual pile-head Load-Movement
of the Araquari Prediction Event

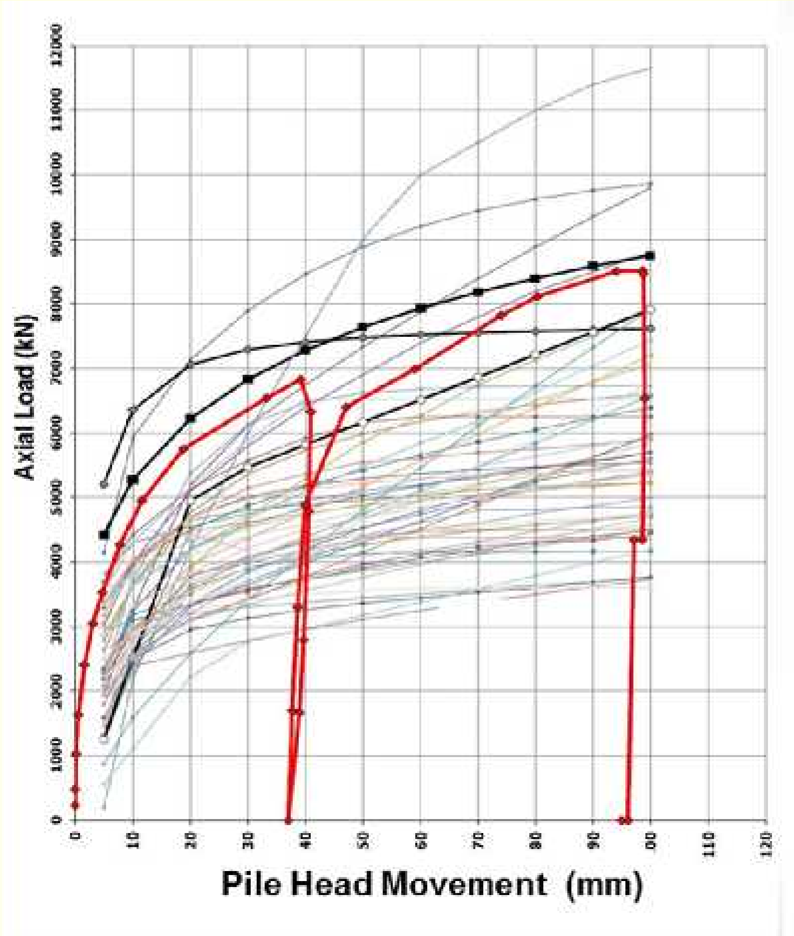
The participants in the prediction event were requested to provide the pile-head load-movement curve from start to 100 mm movement, i.e., the pile "capacity" of the 1,000 mm diameter pile, as defined by the Brazilian organizers.

The pile was instrumented with strain-gages at several levels. However, the uneven load-holding durations and unequal load increments combined with the (unintended, as it were) unloading/reloading will make the evaluation of the strain-gage records ambiguous and unconvincing.

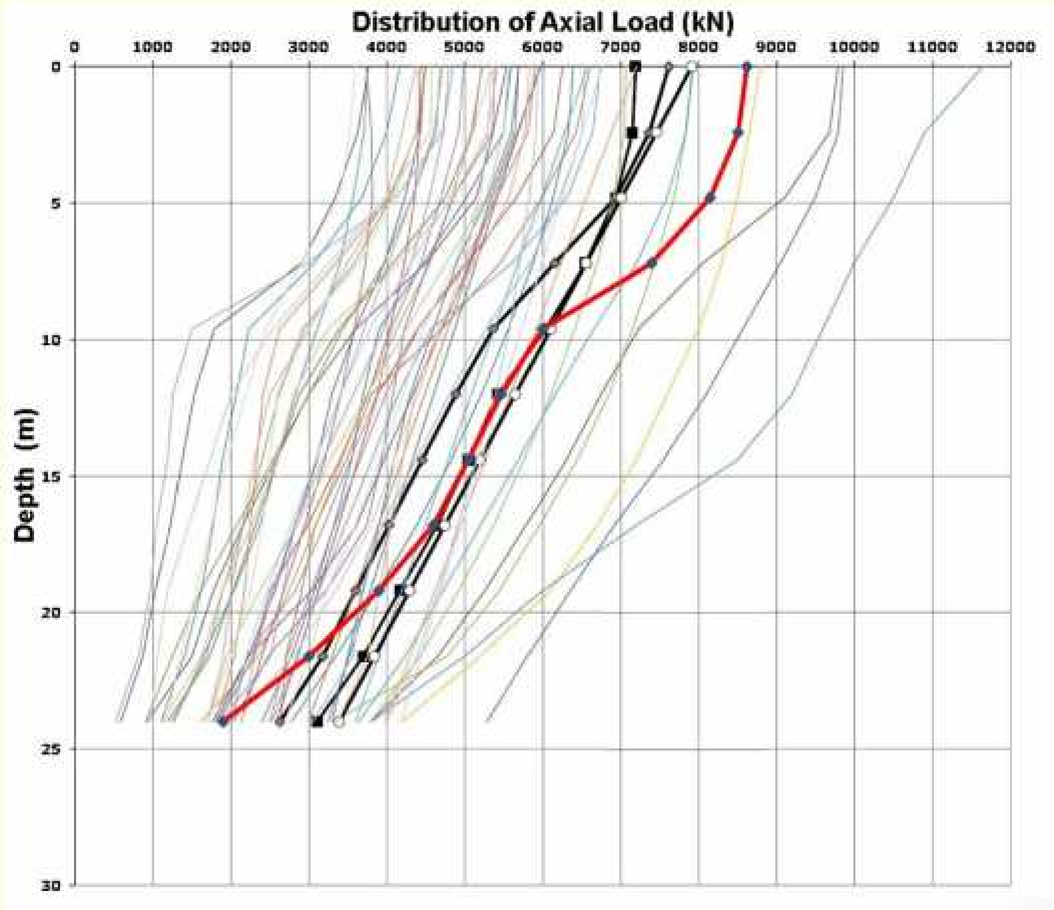
Cnt.

Araquari Prediction Event — As Predicted and As Compared to Actual Results

Pile-Head Load-Movements



Load Distributions

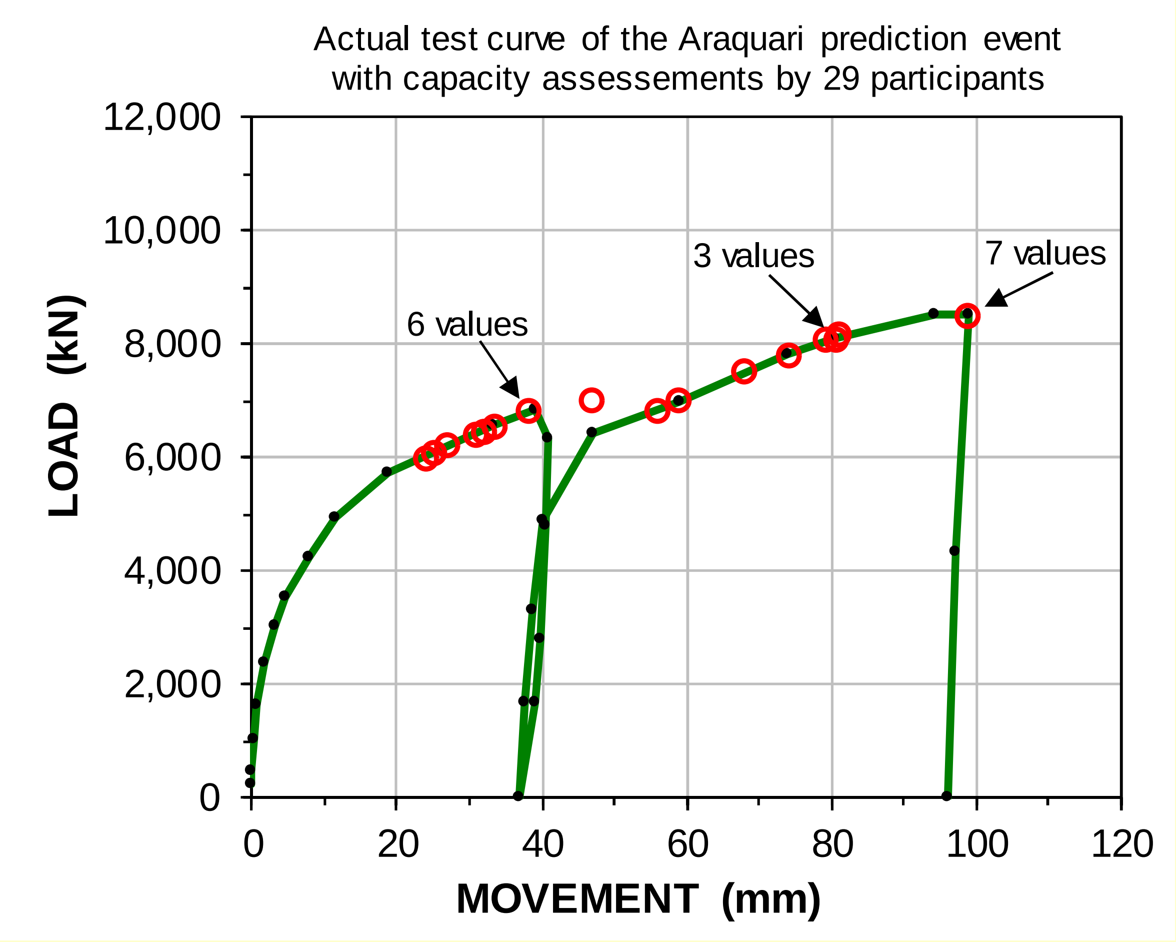


Araquari Prediction Event, IFCEE 2015, Brazil.
Assessments of Capacity

I was one of the participants in the prediction event. On receiving the actual test results after the conference, I wrote to all predictors and asked them to tell me what capacity they would asses from the actual load-movement curve.

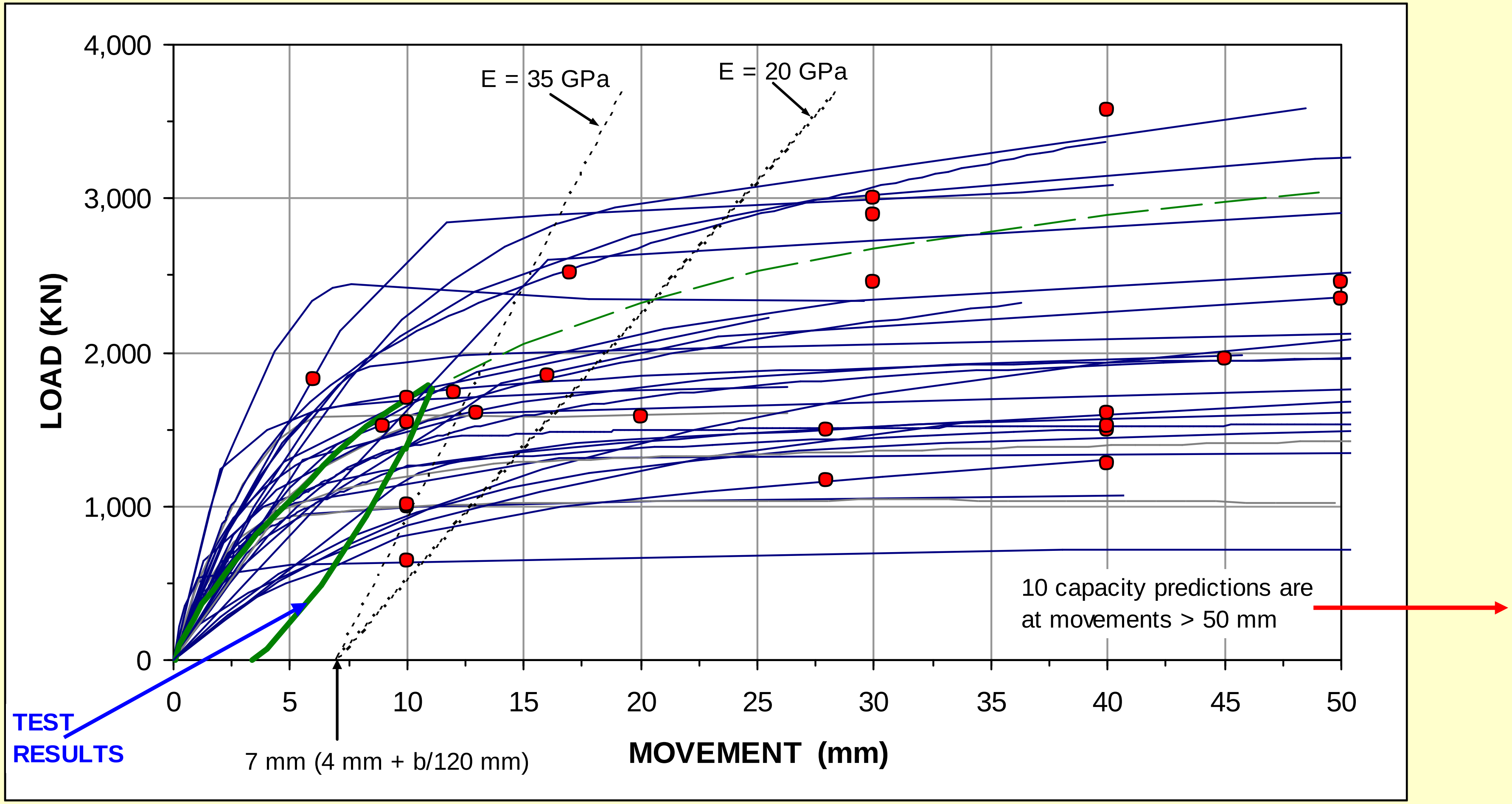
Note, that assessment is no longer a prediction, but an assessment of fact applying the participants usual method for determining capacity as achieved by a static loading test.

Twenty-nine of the participants replied giving me their capacity value.



Edmonton, Alberta, 2011

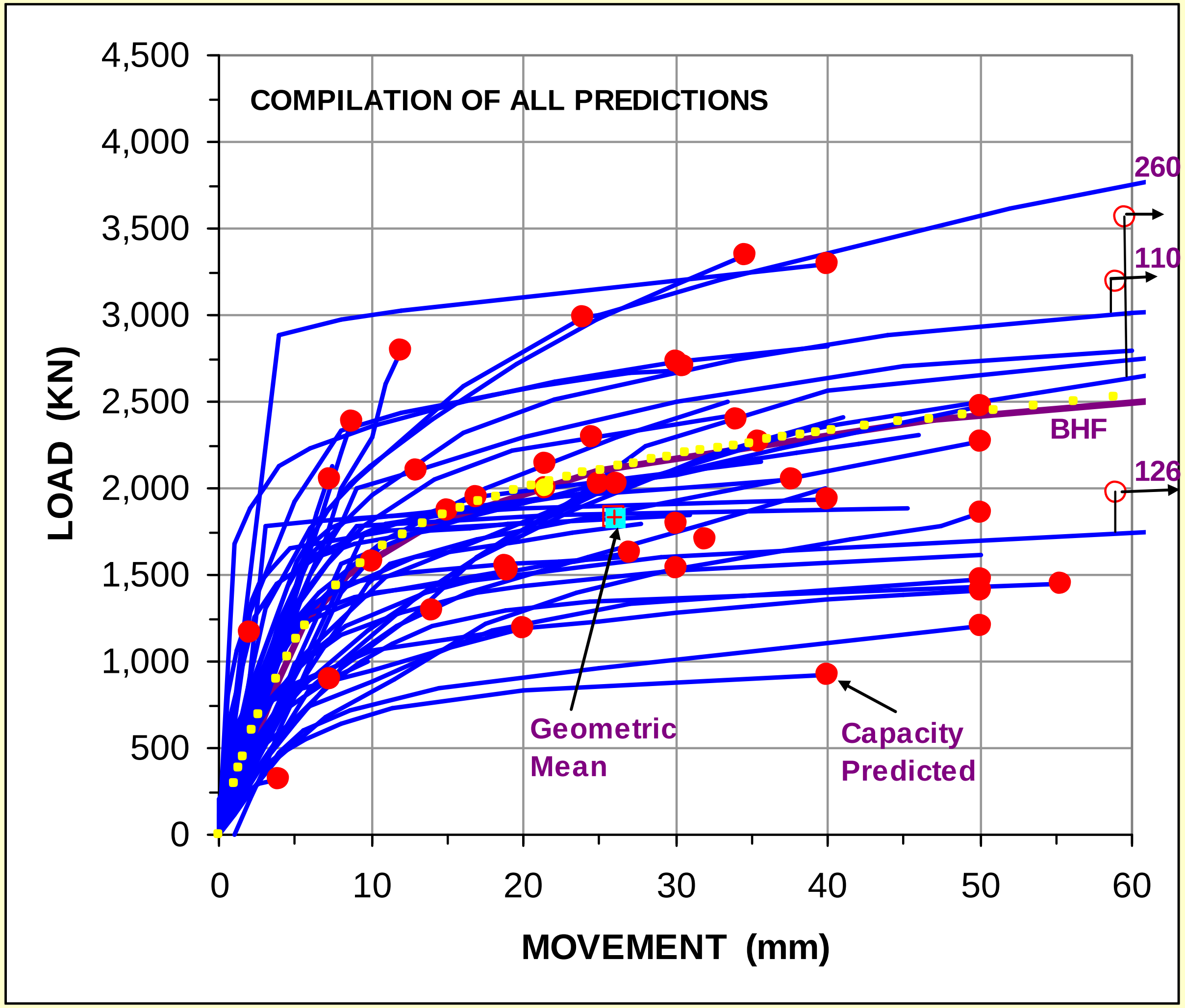
Prediction of load-movement and capacity of a 400-mm diameter, 18 m long, augercast pile constructed in transported and re-deposited glacial till.



Prediction Event in Bolivia April 2013

450 mm diameter, 17.5 m long, bored pile in fine sand

Compilation of predicted load-movement curves and capacities, TP1



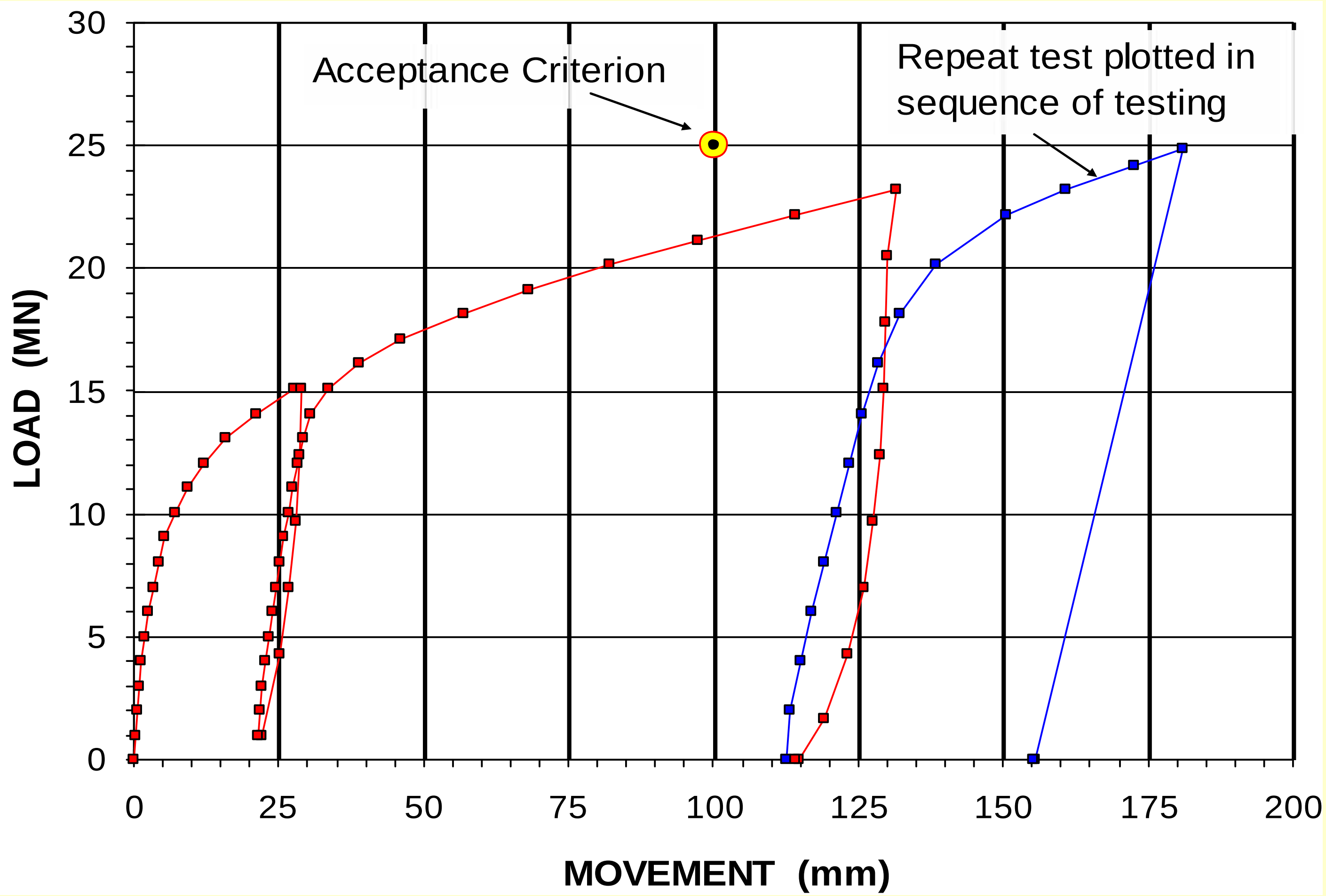
So, when designing foundations for capacity,

is applying a factor of safety or a resistance factor correct, safe, and economical, or do we by this approach open ourselves for litigation and demands on our liability insurance?

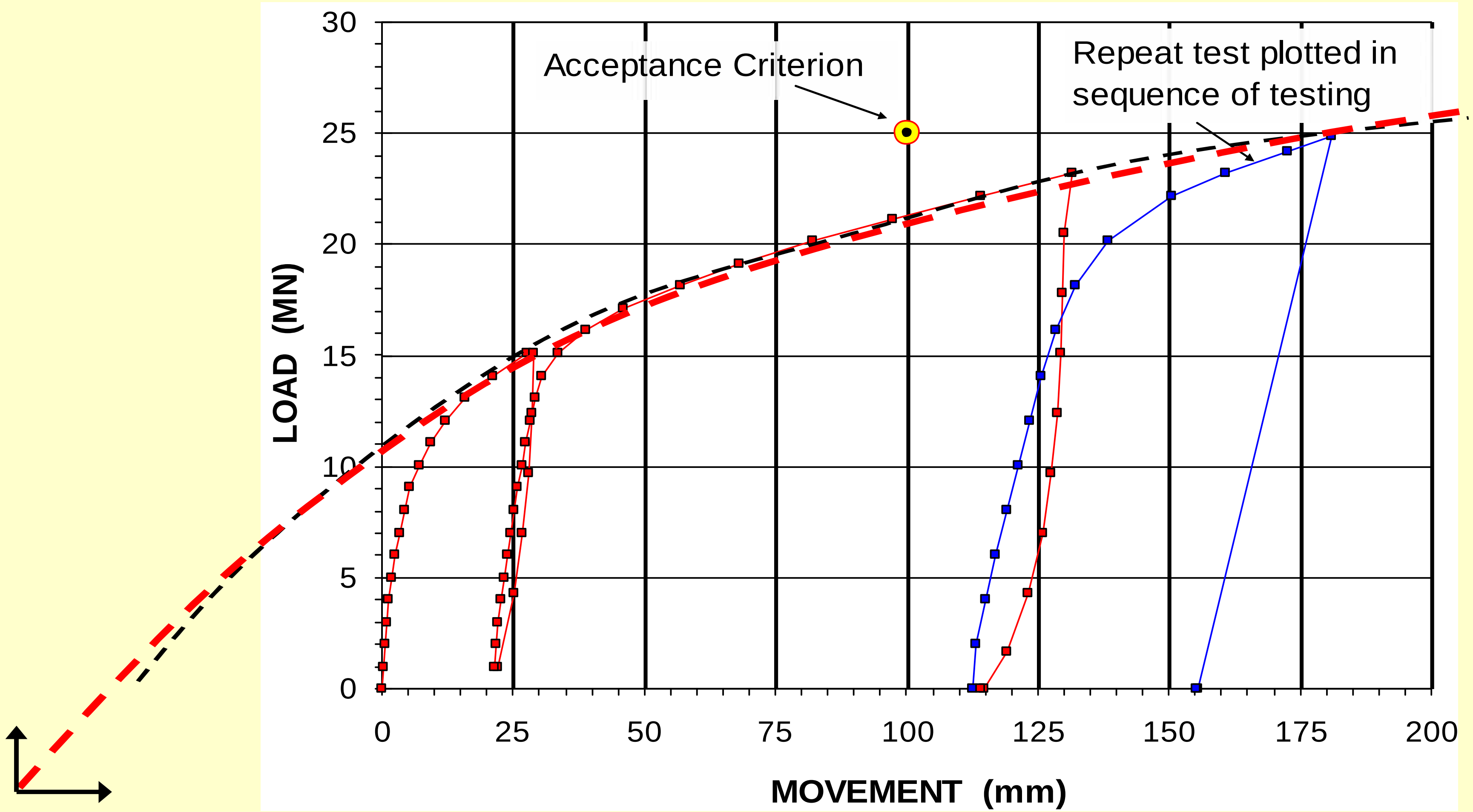
What really do we learn from unloading/reloading and what does unloading/reloading do to the gage records?

Does unloading/reloading add anything of value to a test?

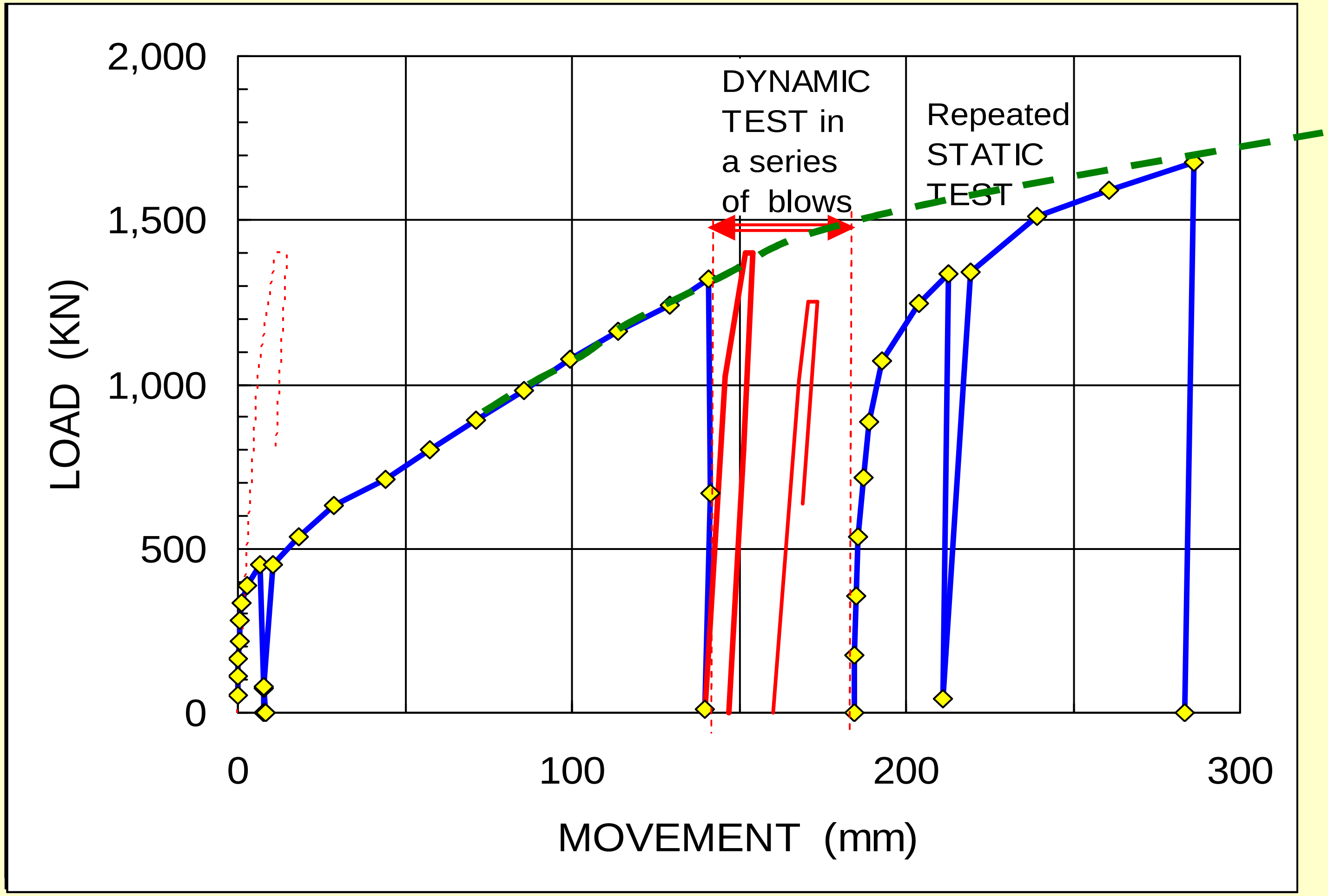
A test on a 2.5 m diameter, 80 m long bored pile



Plotting the repeat test in proper sequence



Also the best field work can get messed up if the analysis and conclusion effort loses sight of the history of the data



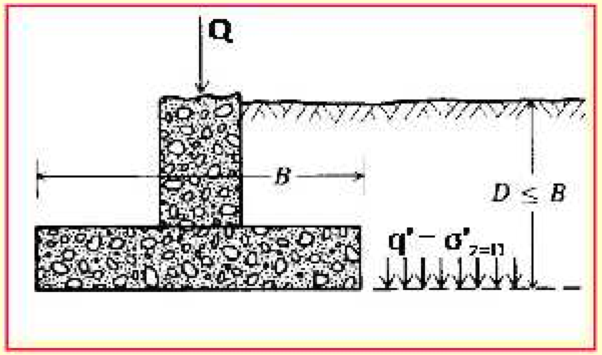
The dynamic test (CAPWAP) was performed after the static test.

The redriving (ten blows) forced the pile down additionally about 45 mm.

The Pile Toe is Really a Footing:

The Bearing Capacity Formula

$$r_u = c'N_c + q'N_q + 0.5b\gamma'N_\gamma$$



where r_u = ultimate unit resistance of the footing

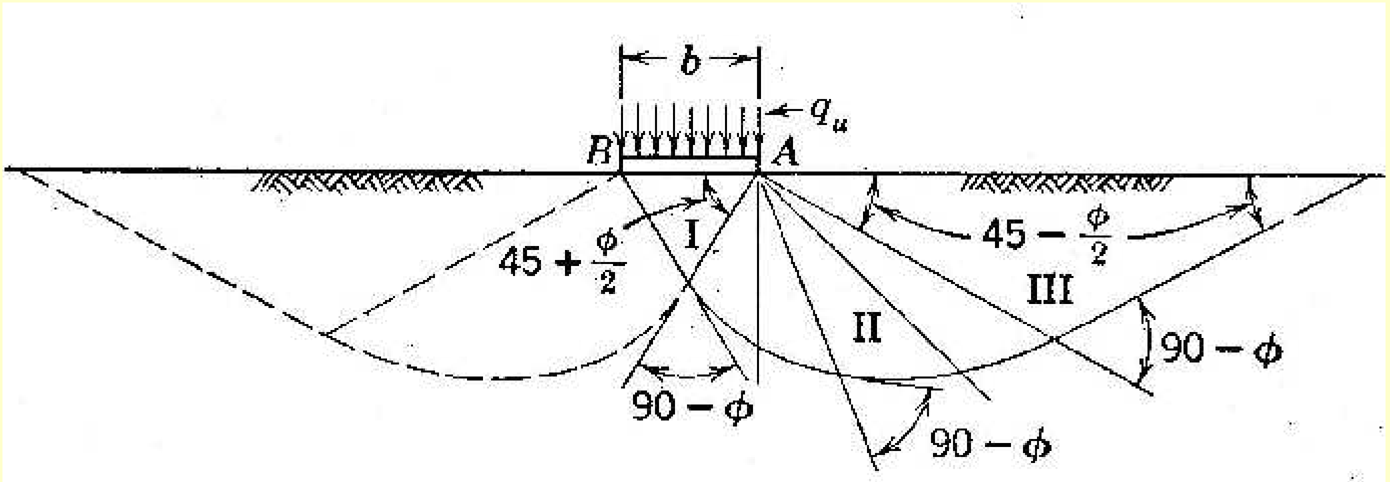
c' = effective cohesion intercept

B = footing width

q' = overburden effective stress at the foundation level

γ' = average effective unit weight of the soil below the foundation

N_c, N_q, N_γ = non-dimensional bearing capacity factors

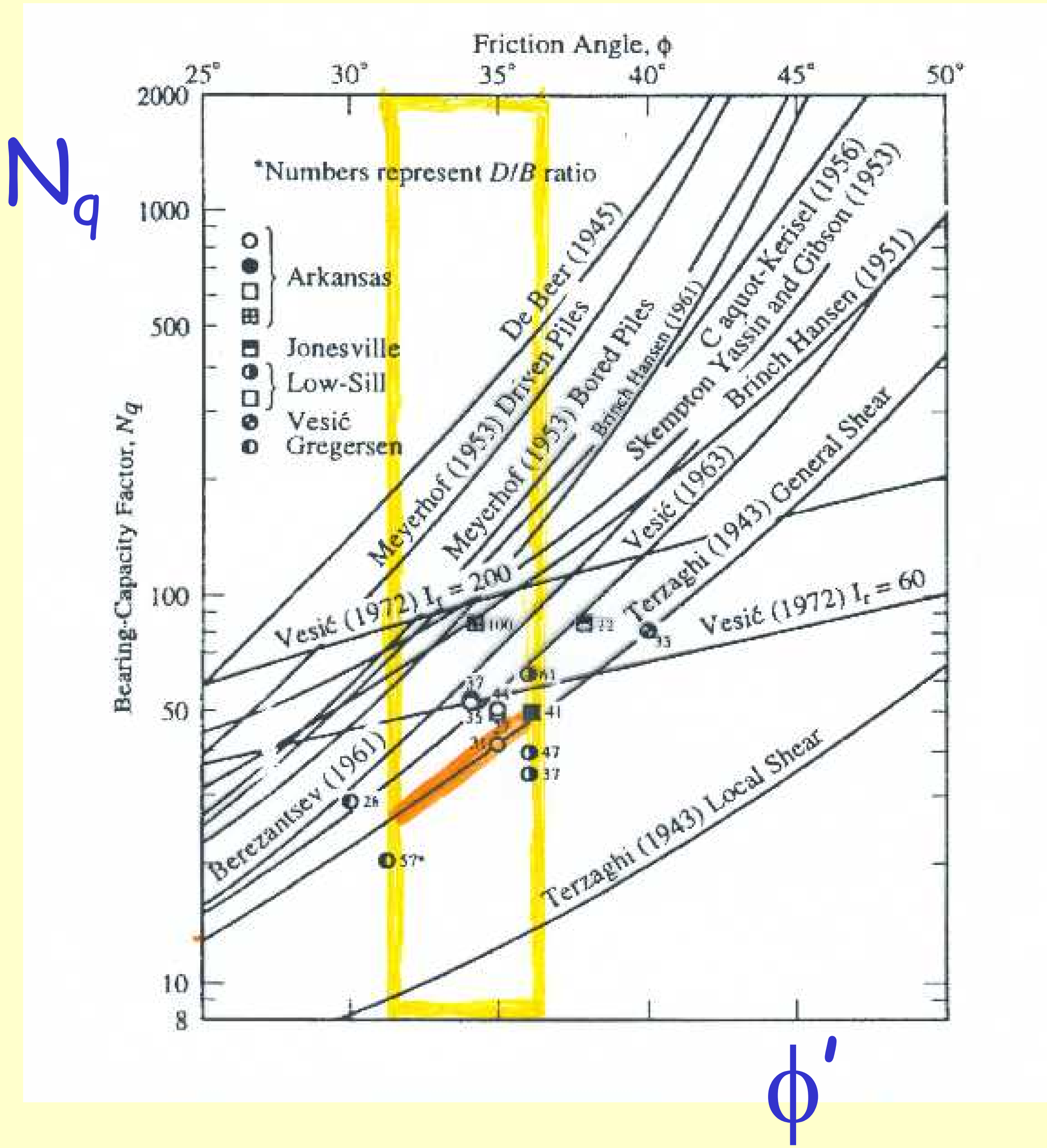


Factor of Safety, F_s

$F_s = r_u/q$

$$r_t = q' N_q$$

N_q was determined in tests—model-scale tests



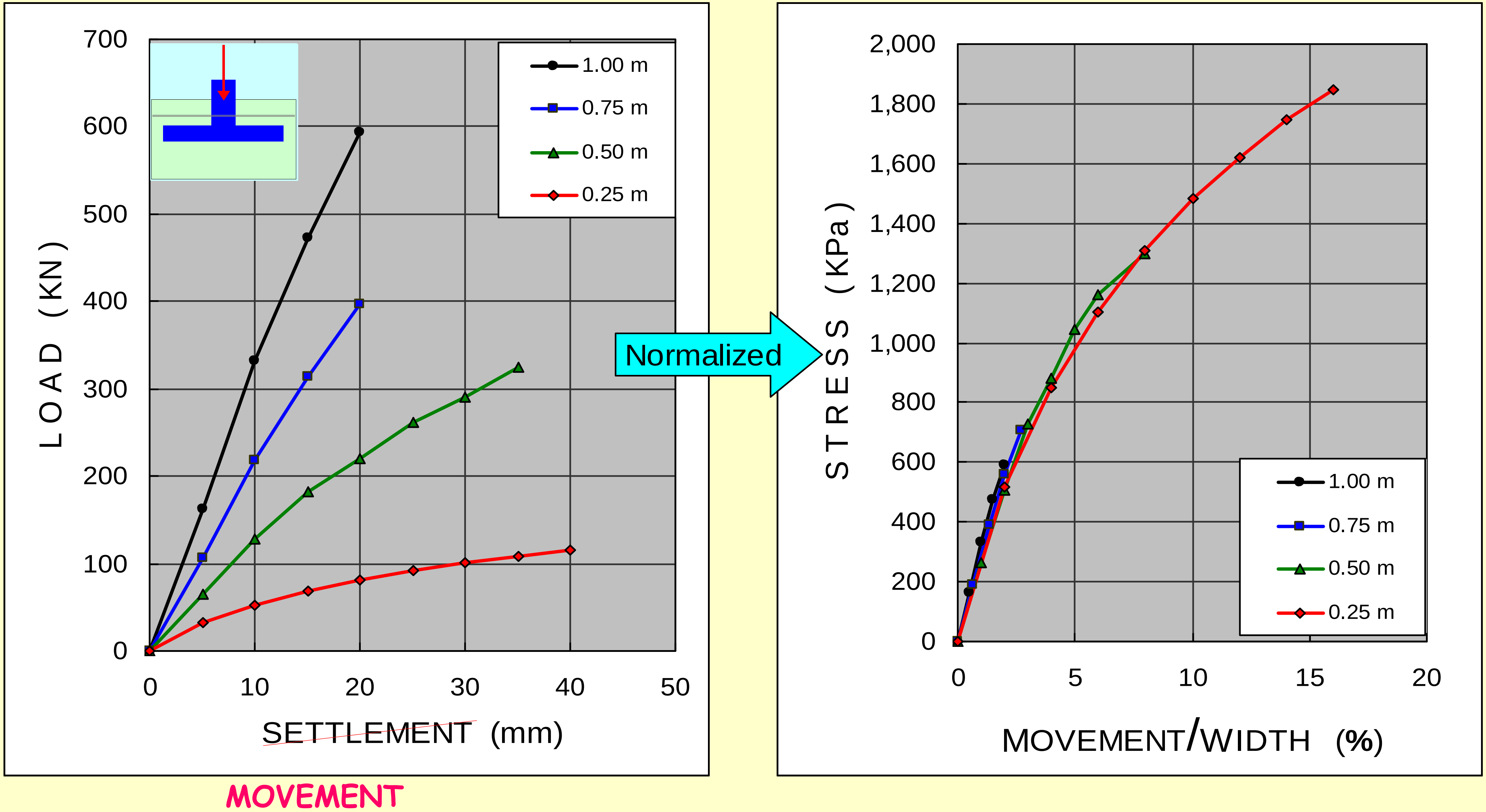
Min to max N_q ratio is up to ≈200 for the same φ'!

The log-scale plot is necessary to show all curves with some degree of resolution.

Why is it that nobody has realized that something must be wrong with the theory for the main factor, the N_q, to vary this much?

Let's compare to the reality?

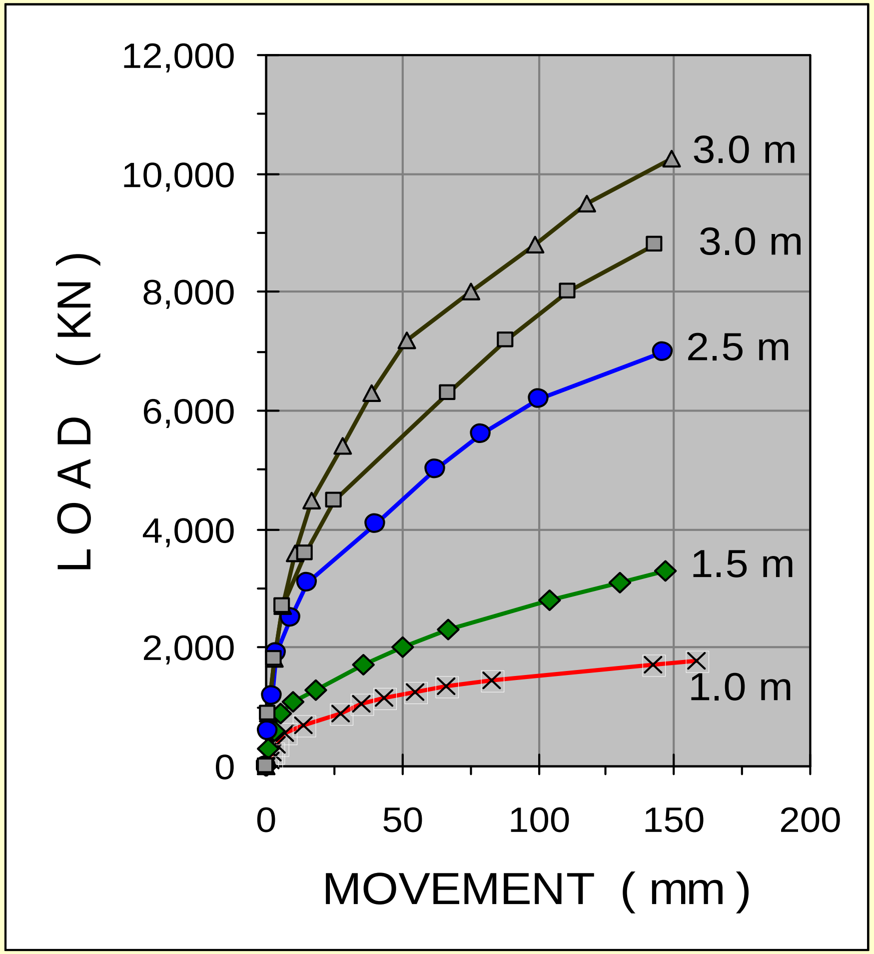
Results of static loading tests on **0.25 m to 0.75 m** square **footings** in well graded **sand** (Data from Ismael, 1985)

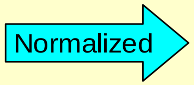


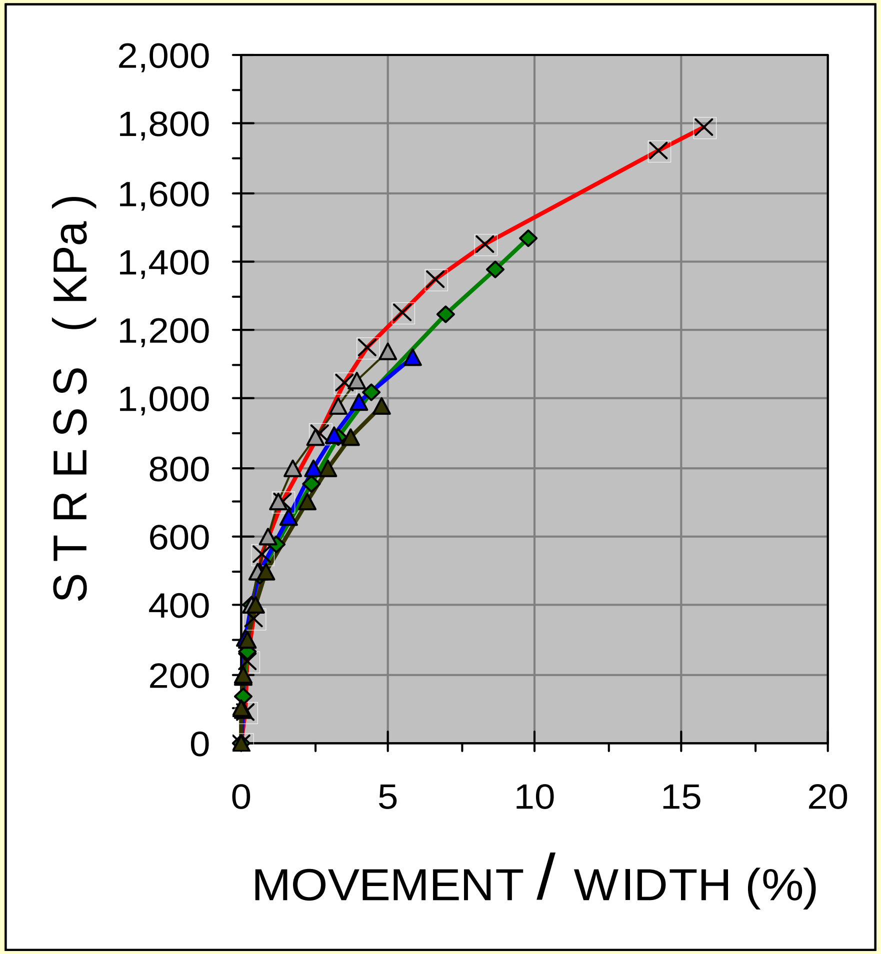
Texas A&M
Settlement Prediction Seminar

Load-Movement of Four Footings on Sand
Texas A&M University Experimental Site
J-L Briaud and R.M. Gibbens 1994,
ASCE GSP 41

As before, we divide the load with the footing area (to get stress) and divide the movement with the footing width, as follows.

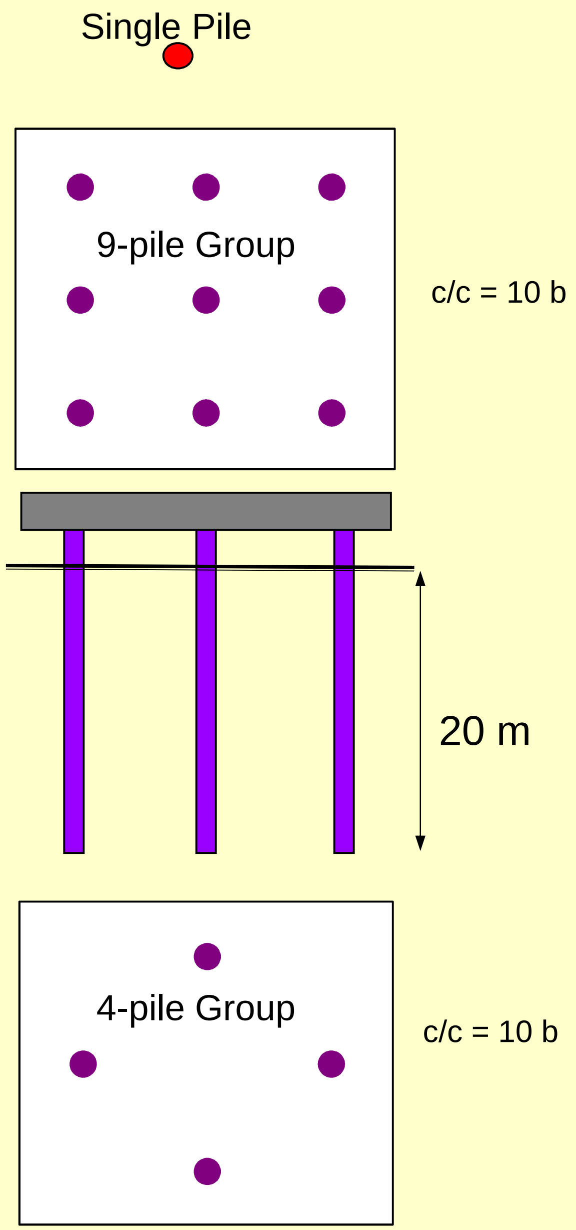


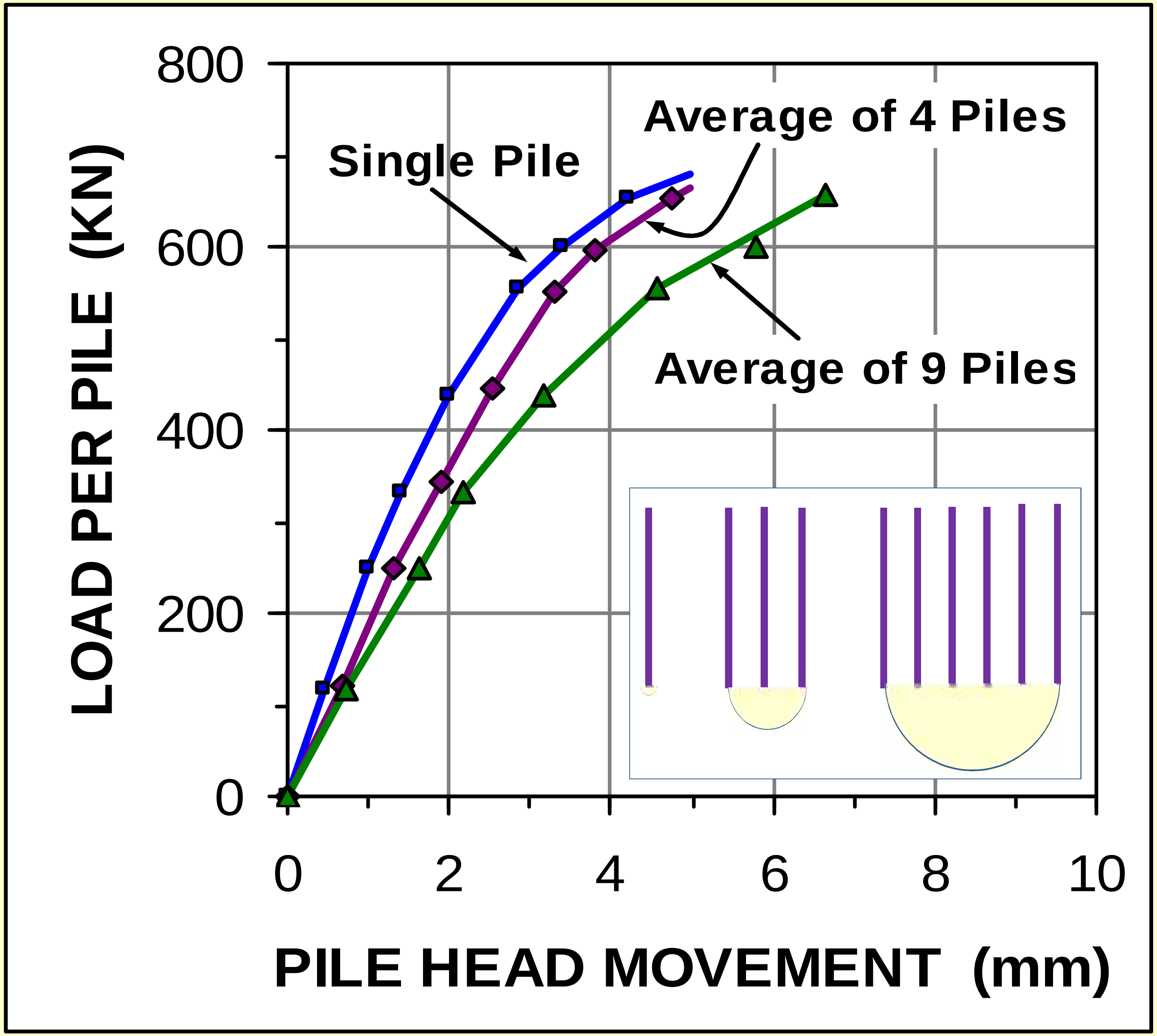




To repeat, when designing foundations based on capacity are we not basing our design on an illusion?

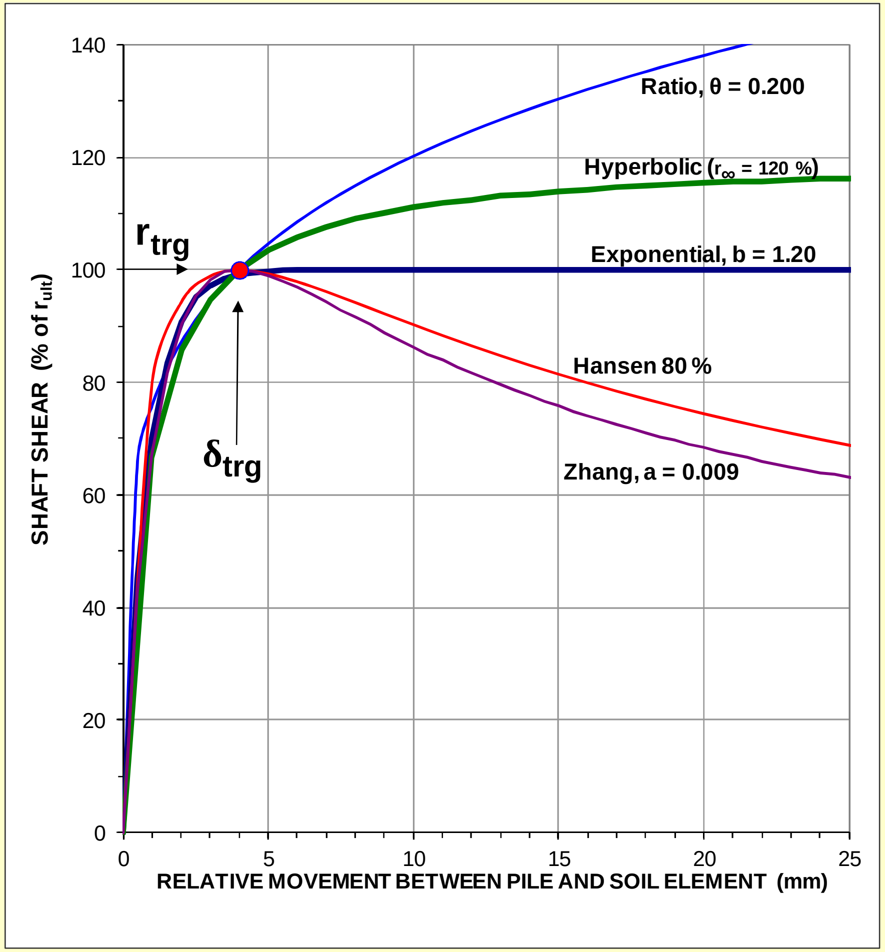
Group Effect





O'Neill et al. (1982)

Shaft Resistance and t-z and q-z functions



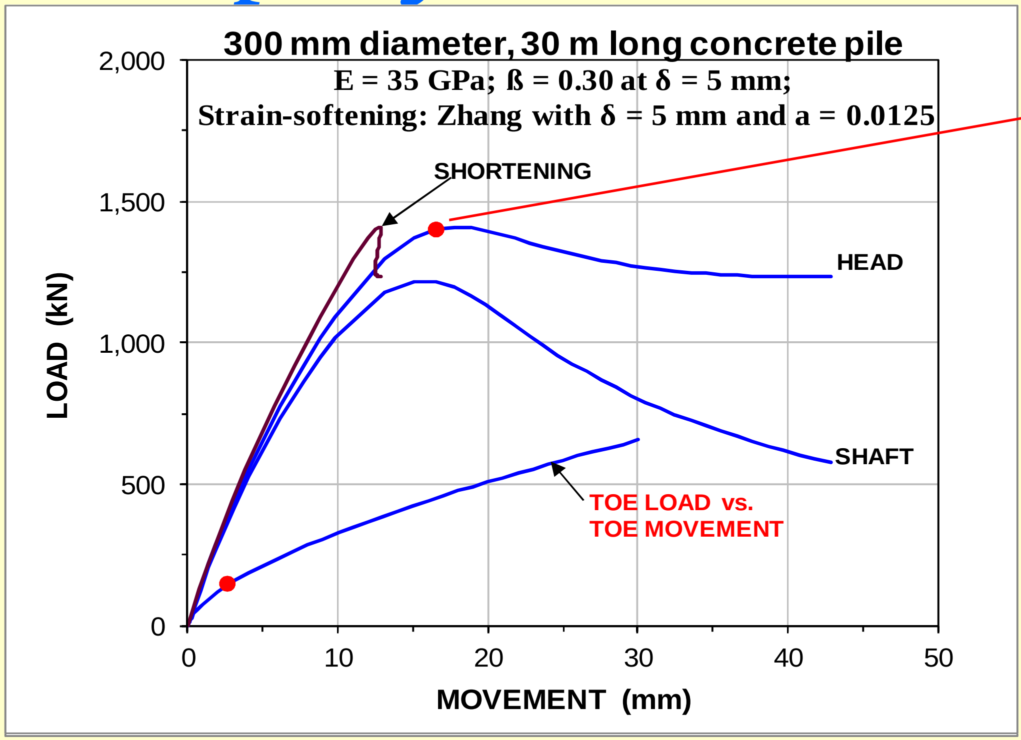
Strain-hardening

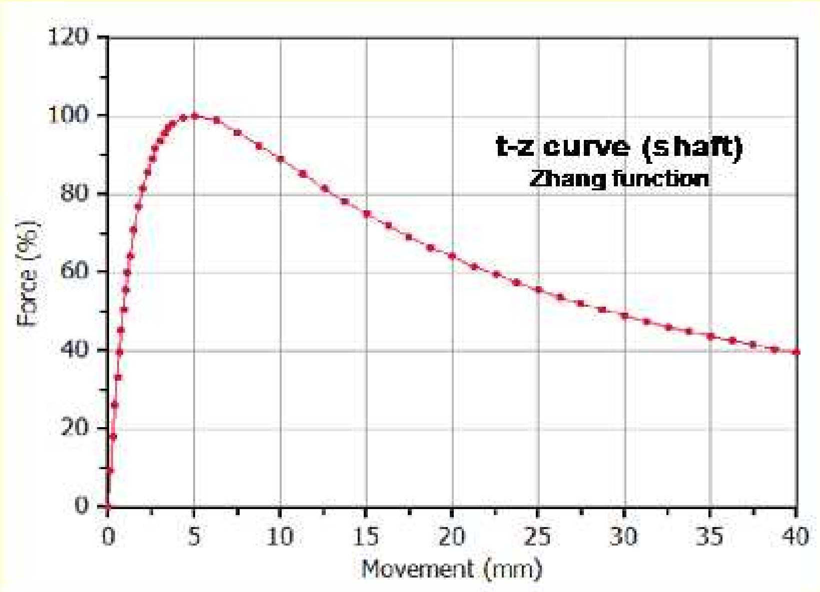
Elastic-plastic

Strain-softening

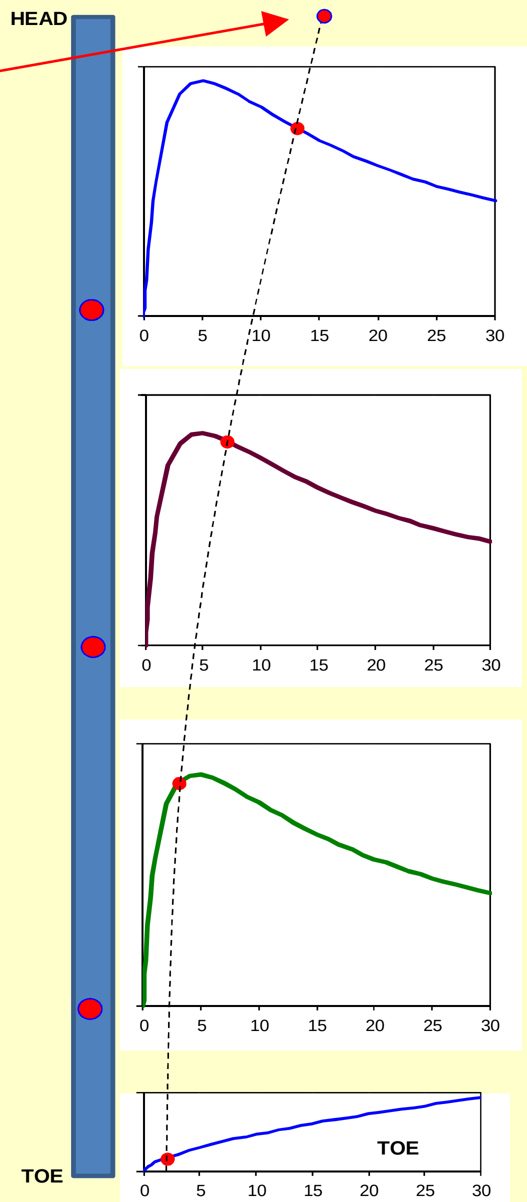
The t-z and q-z functions are fundamental to the analysis of pile response

"Capacity"?





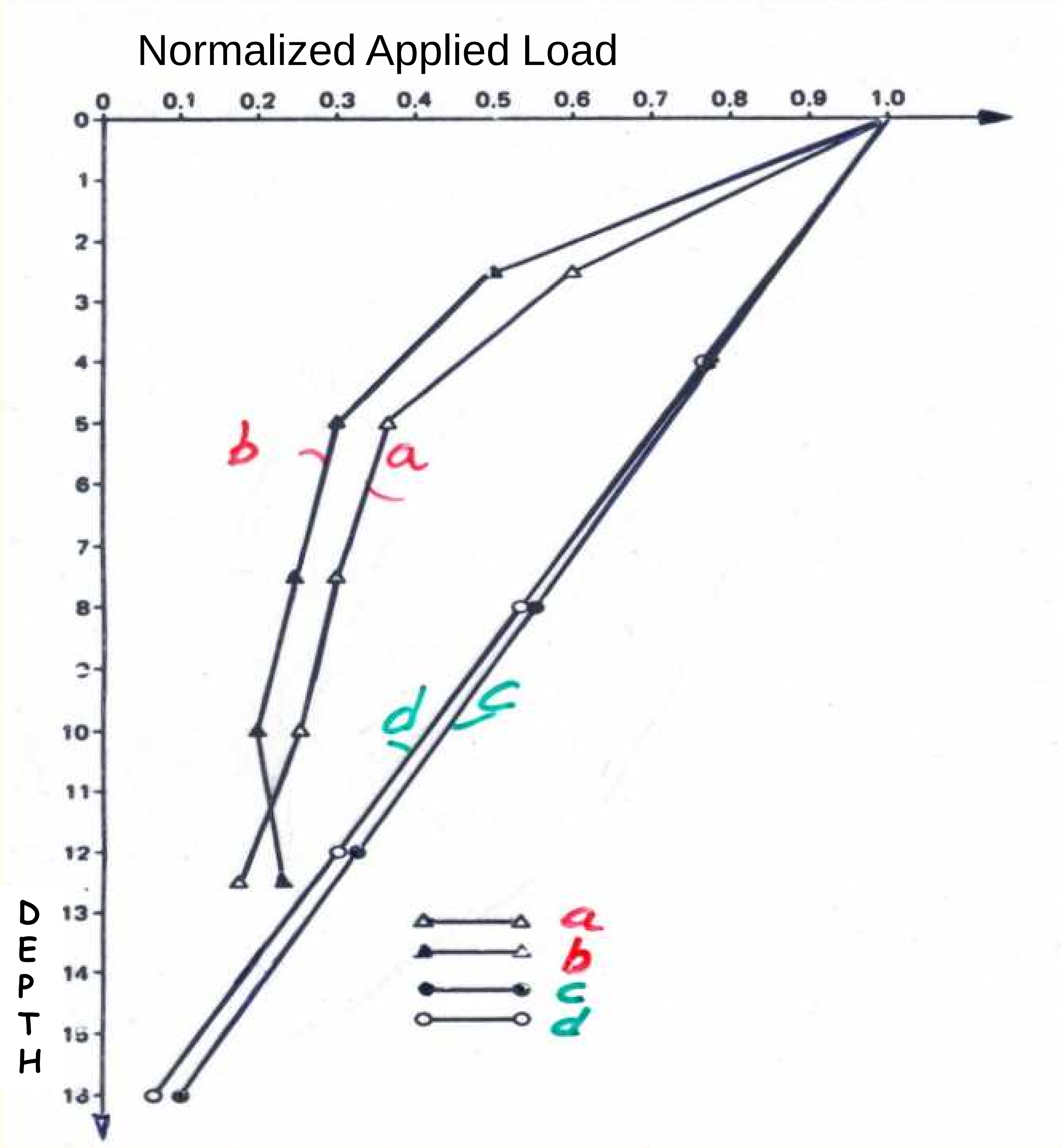
MOVEMENTS AT GAGE LOCATIONS



First shown important by Bram van Weele back in the 1960s

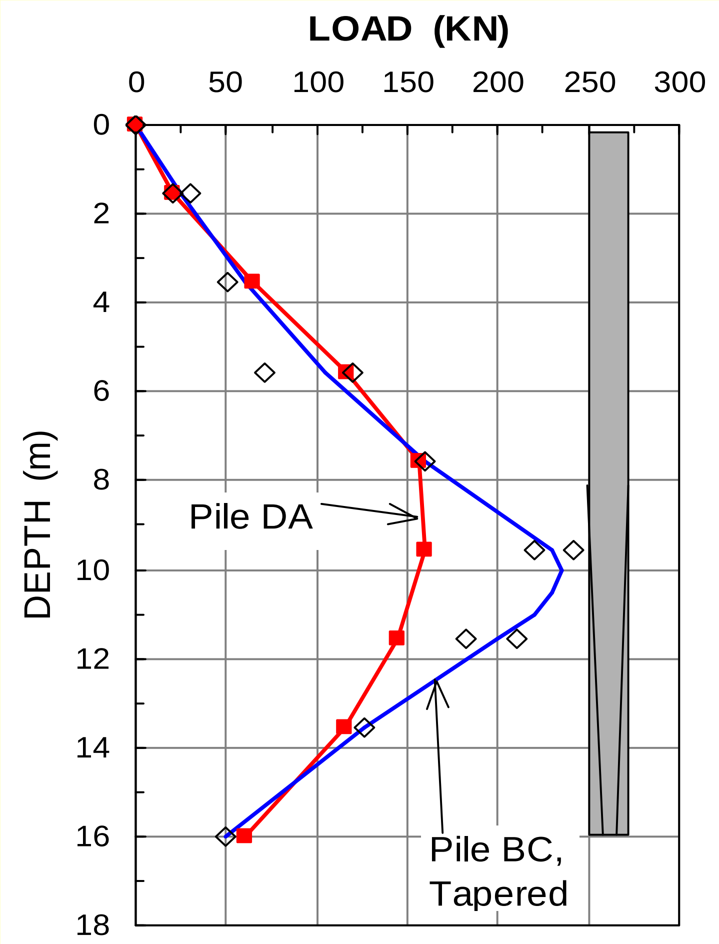
If we want to know the load distribution, we can measure it. But, what we measure is the increase of axial load in the pile due to the load applied to the pile head. What about the axial force in the pile that was there before we started the test?

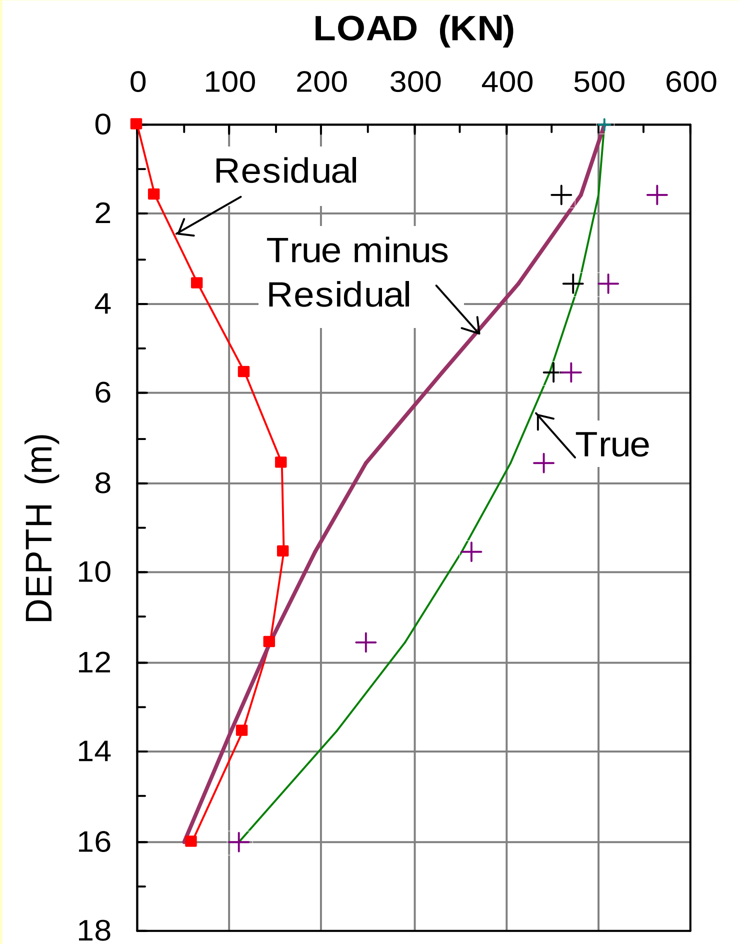
That is, the Residual Force



Load distributions in static loading tests on four instrumented piles in clay

Example from Gregersen et al., 1973





A. Distribution of residual force in DA and BC before start of the loading test

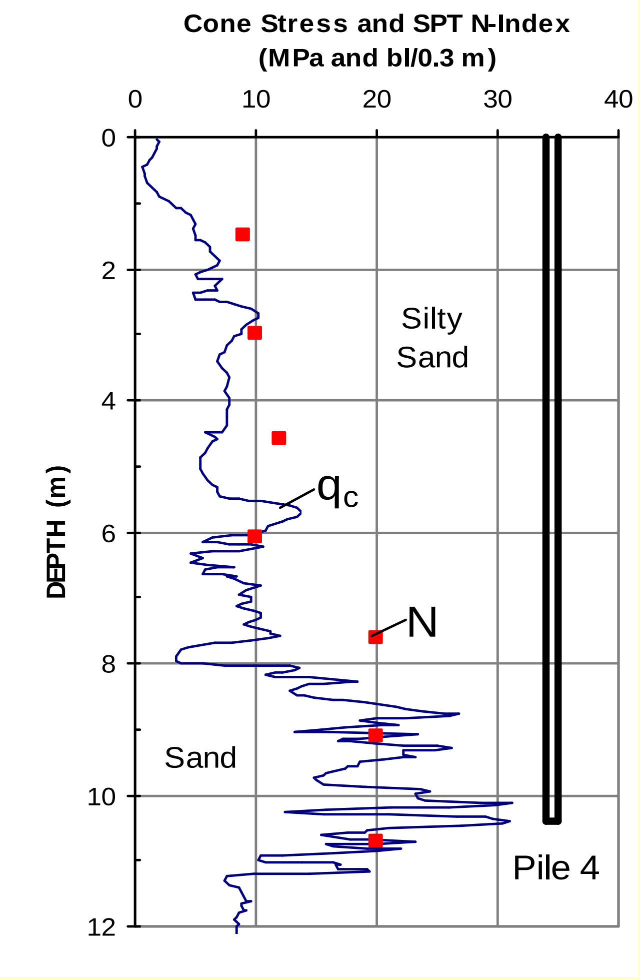
B. Load and resistance in DA for the maximum test load

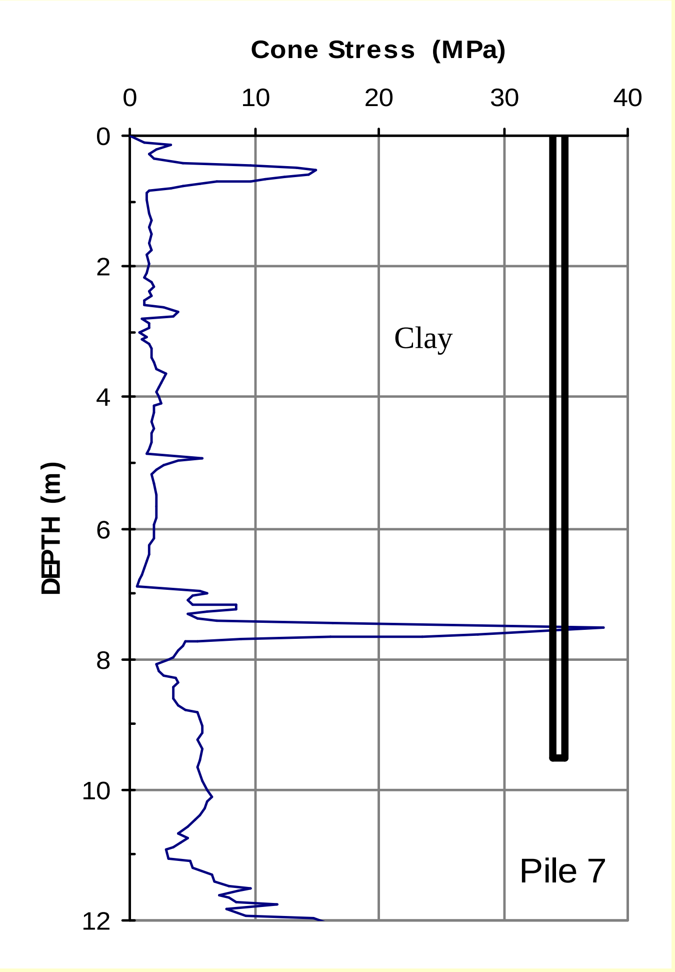
FHWA tests on 0.9 m diameter bored piles

One in sand and one in clay

(Baker et al., 1990 and Briaud et al., 2000)

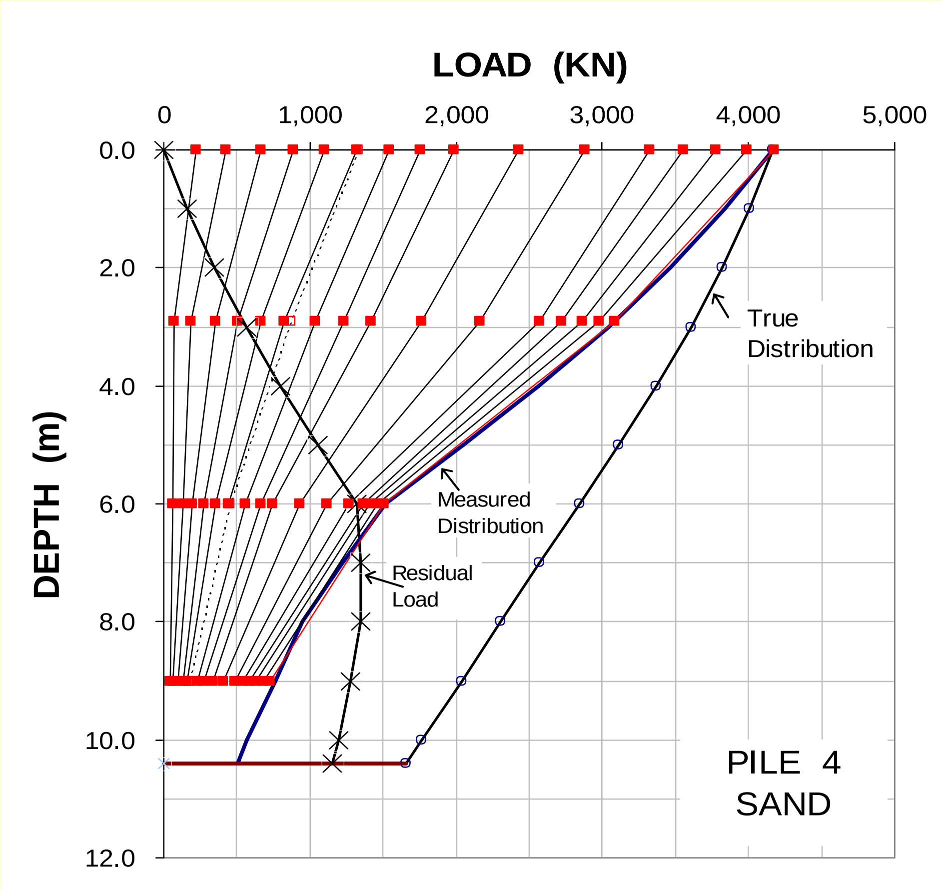
Silty Sand

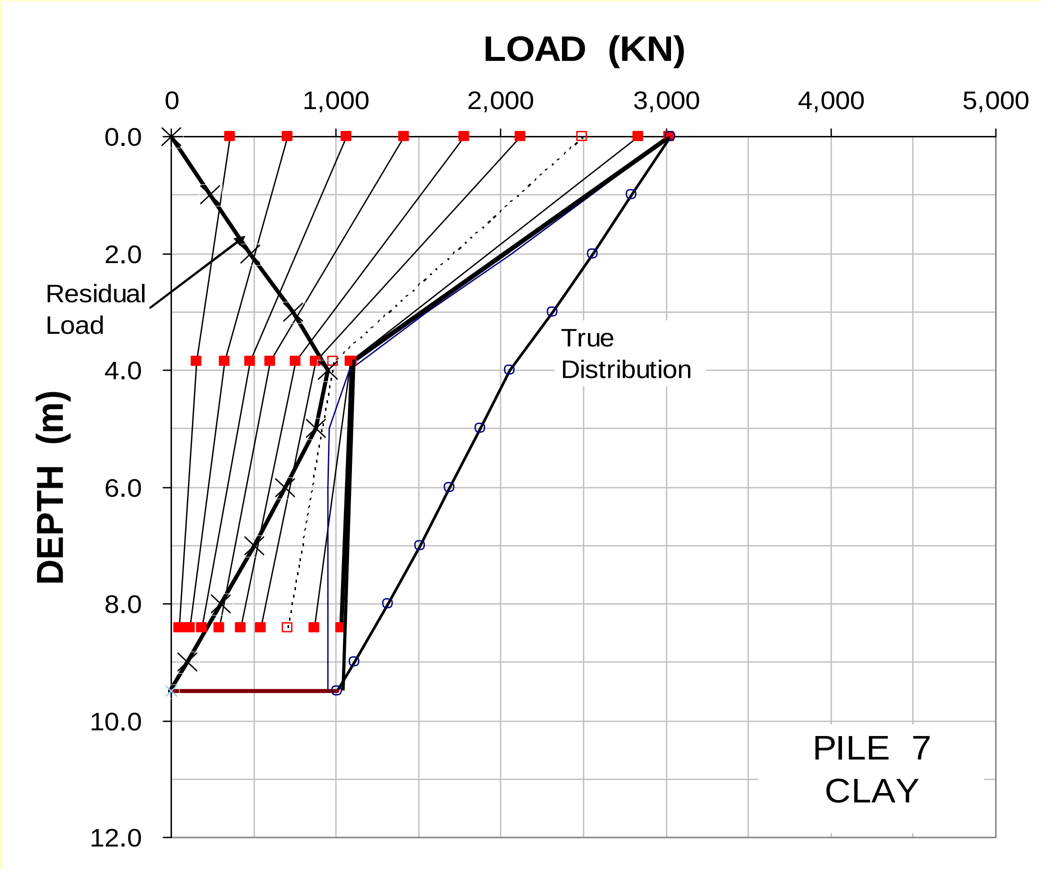




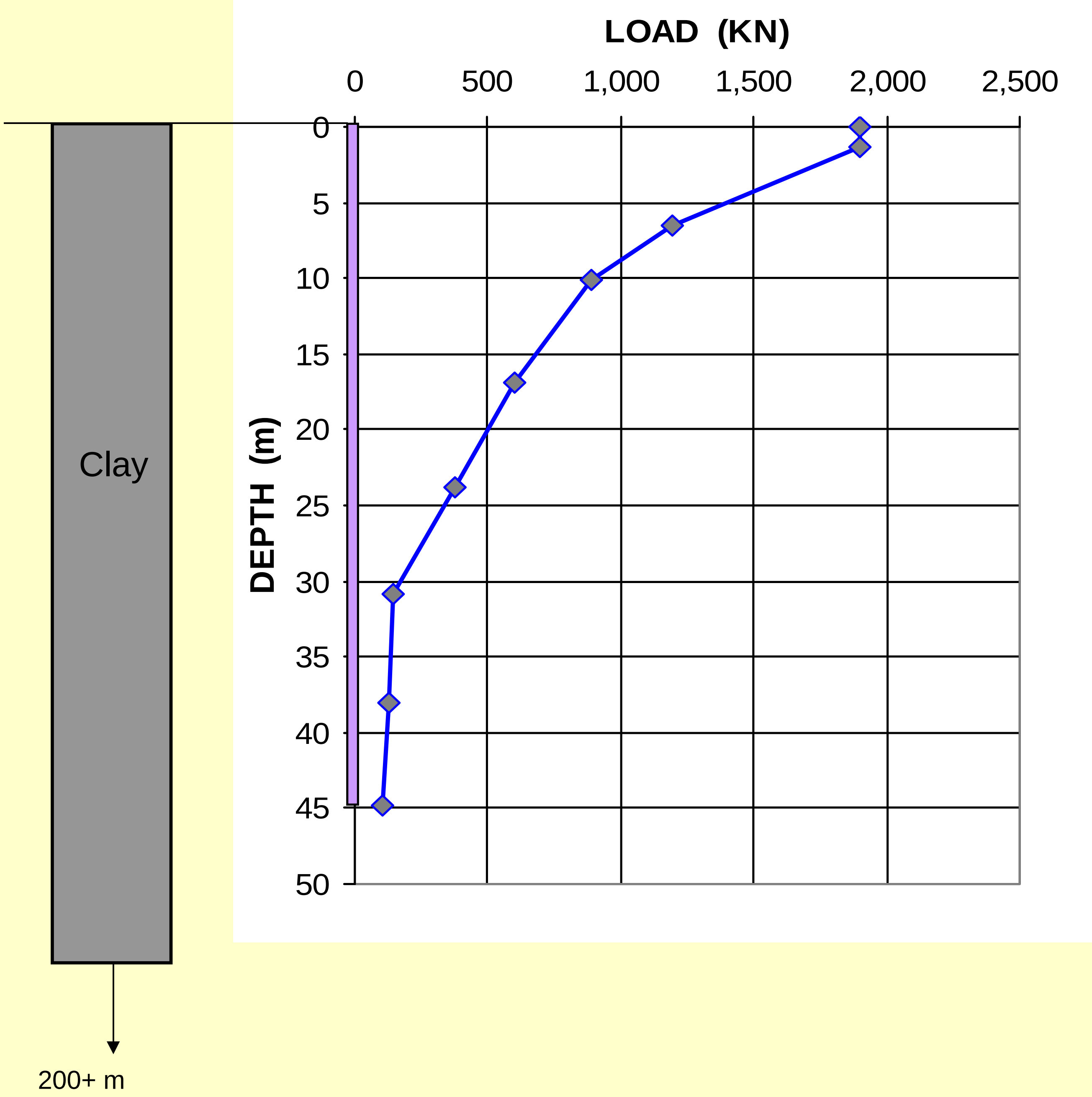
Clay

ANALYSIS RESULTS: Load-transfer curves





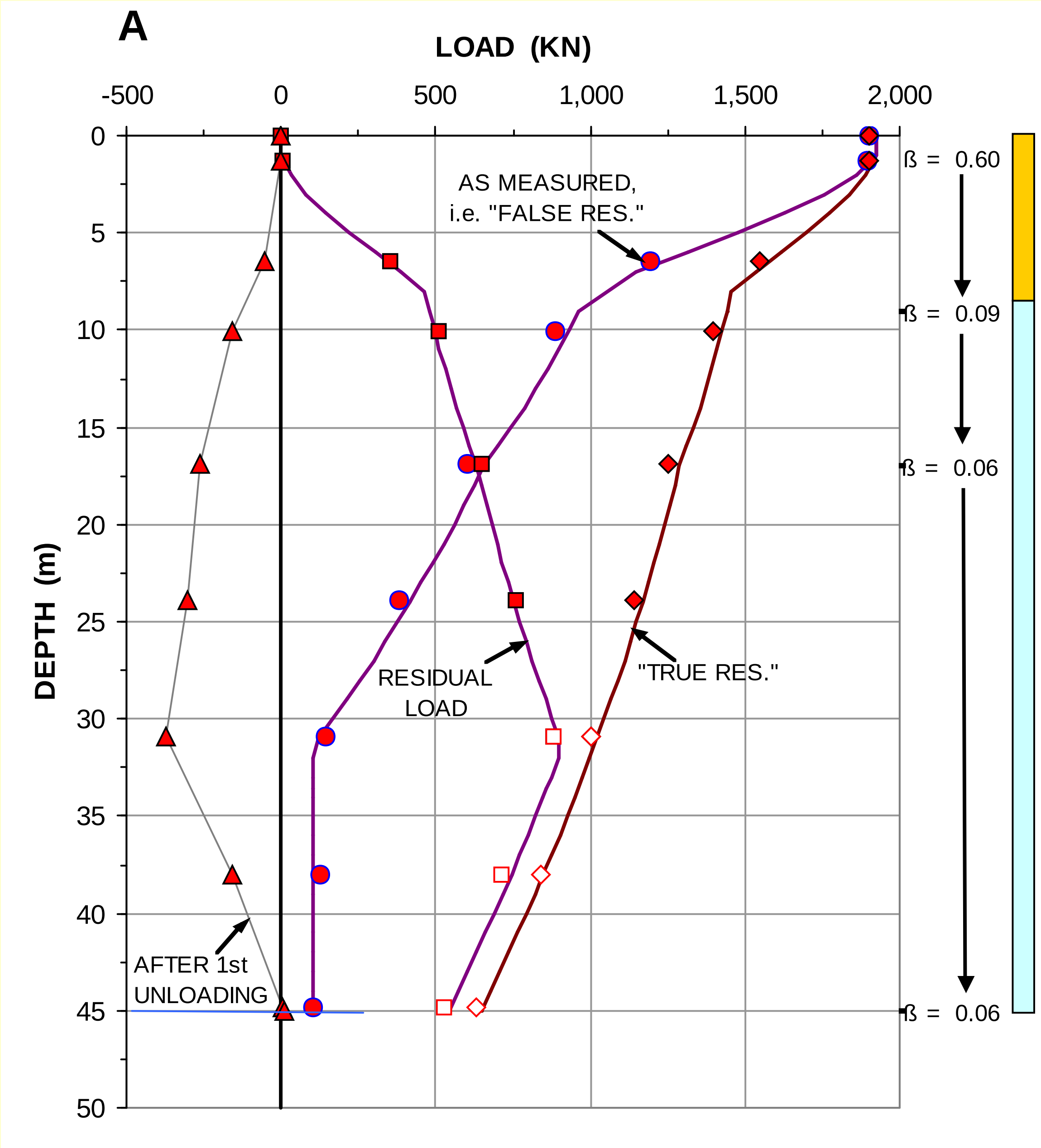
Determining True Resistance from Measured Resistance ("False Resistance")



Static Loading Test at Pend Oreille, Sandpoint, Idaho, for the realignment of US95

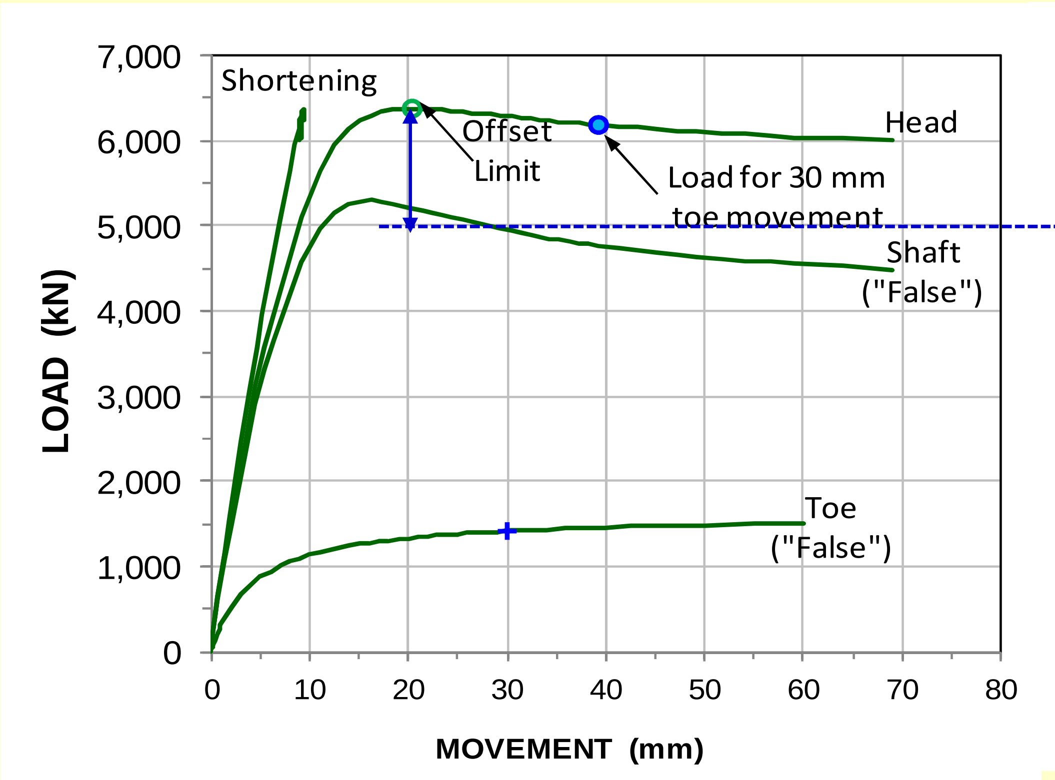
406 m diameter, 45 m long, closed-toe pipe pile driven in soft clay

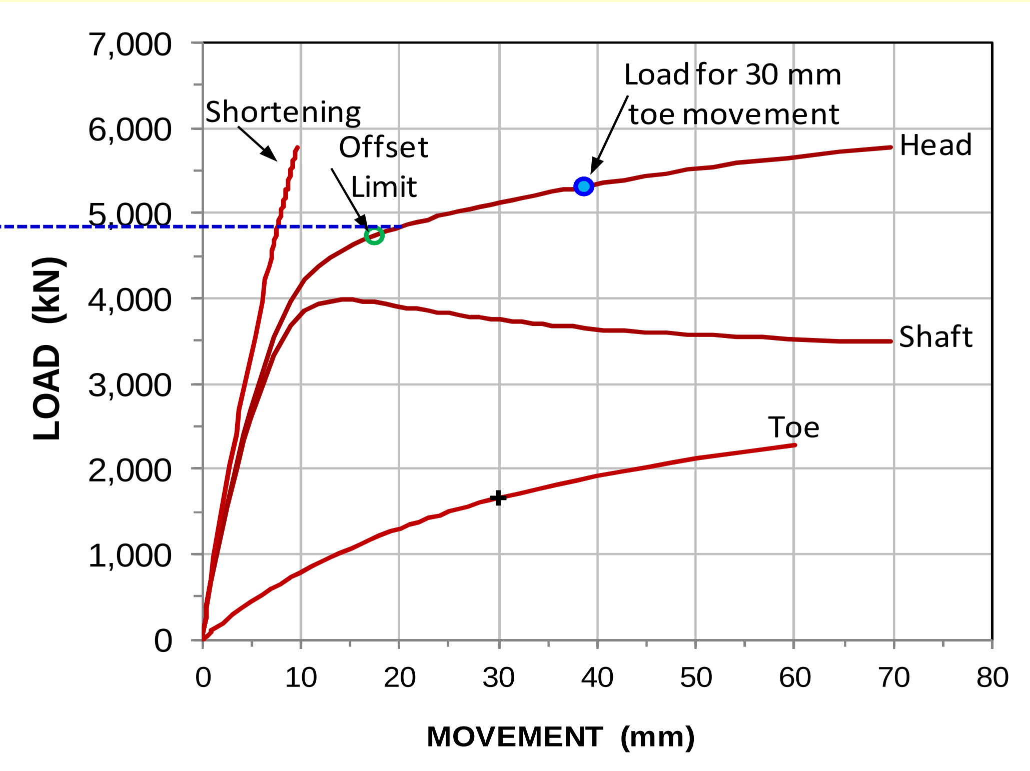
Fellenius et al. (2004)



Test on a strain-gage instrumented, 406 mm diameter, 45 m long pile driven in soft clay in Sandpoint, Idaho

Presence of residual force is not just of academic interest





Results from a test on a 15 m long, 600 mm diameter, jacked-in concrete pile (Fellenius 2014). The manner of testing built-in considerable residual force in the pile.

Same pile assumed tested without residual force being present. (The test results were first fitted to a UniPile t-z/q-z analysis, whereafter the analysis was converted to results for an identical pile and soil, but with no residual force present).

From *Recent experiences with static pile load testing on real job sites and General Report – Design methods based on static pile load tests.*

From Proceedings of ETC3, *Symposium on Design of Piles in Europe*
Leuven, Belgium, April 2016

Load-distribution from a 508 mm diameter driven cast-in-place pile installed to 15 m depth.

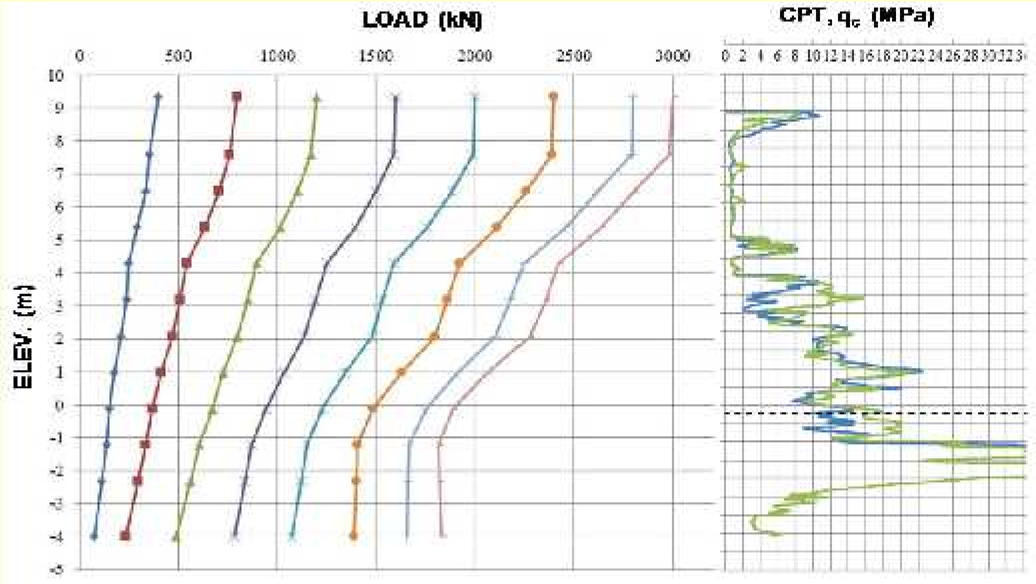


Figure 10 on Page 74

Load-distribution from a 620 mm diameter, 16.6 m long, screw pile installed to 15 m depth.

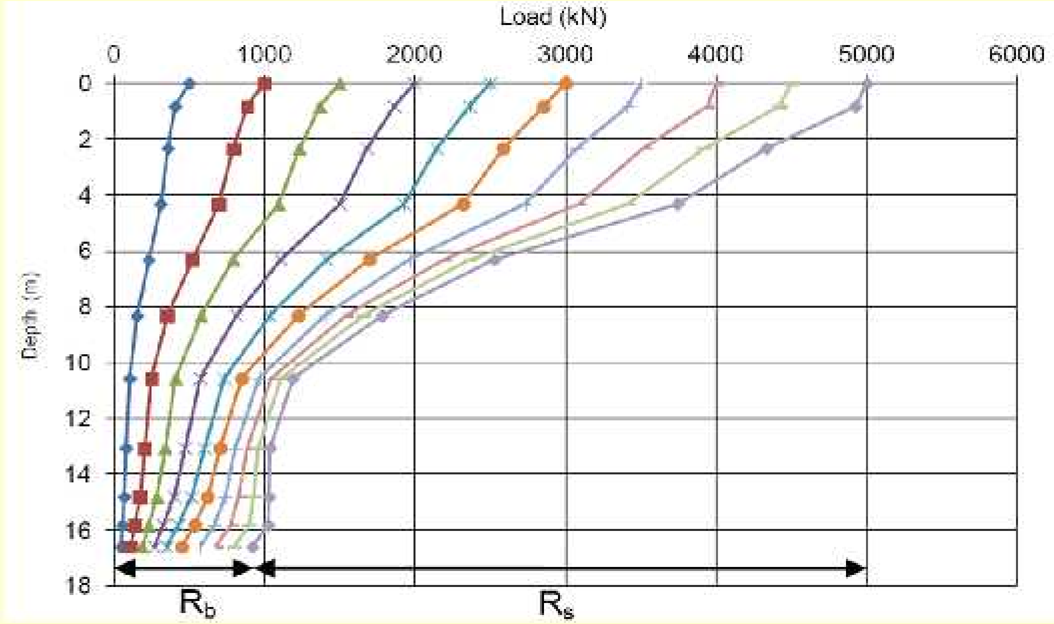


Figure 5a on Page 90

The authors mention various reasons for the lack of shaft resistance along the lower length, but do not address the main reason, which is that the piles have significant residual force, causing the shaft resistance along the upper length to appear too large and that along the lower length to appear too small or non-existent.

Load-movement curves from a 560 mm diameter "displacement screw pile" (CFA pile). Pile length and soil type not mentioned.
Residual toe force can be assumed small.

Load-movement curves from a 508 mm diameter driven cast-in-place pile. Pile length and soil type not mentioned. (Not the same pile as that used to show load distribution).
Residual toe force is usually considerable for driven piles.

Quote from the Proceedings "***The design based on the results of pile load tests is based on the results of the ultimate or pile capacity at 10%*** *) ***diameter pile base*** (toe) ***displacement***".

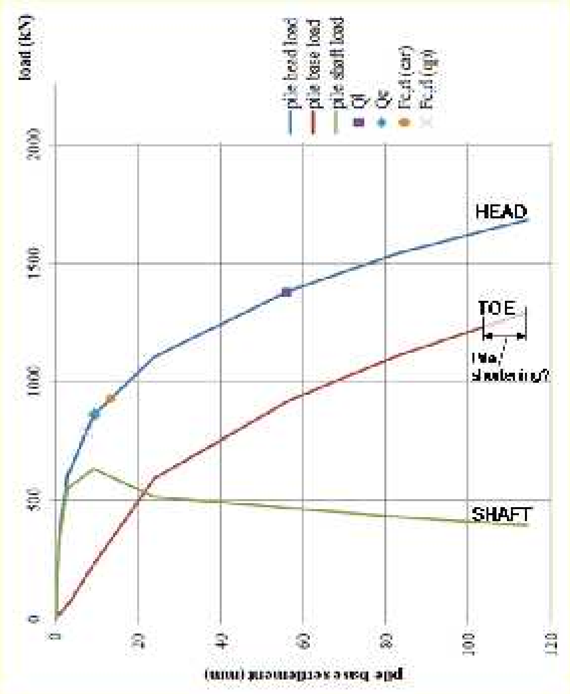


Figure 3 on Page 68

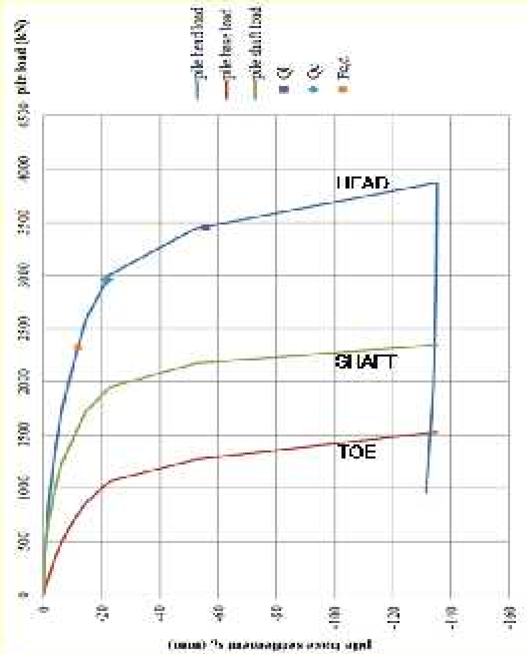
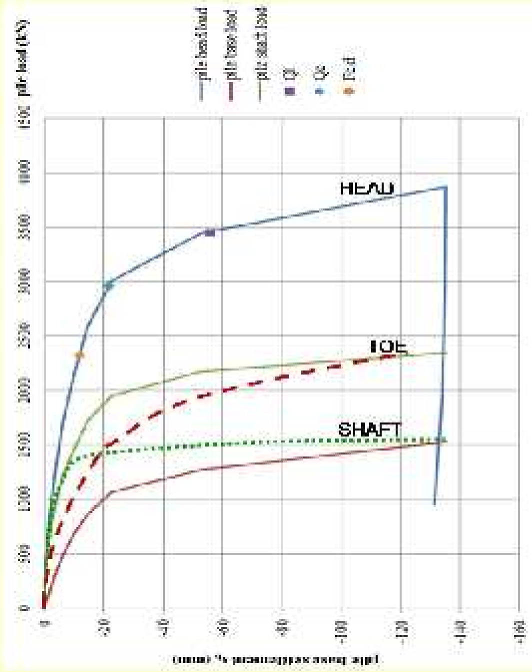


Figure 2 on Page 67



Same as Figure 2 on Page 67 with presumed "true" shaft and toe curves

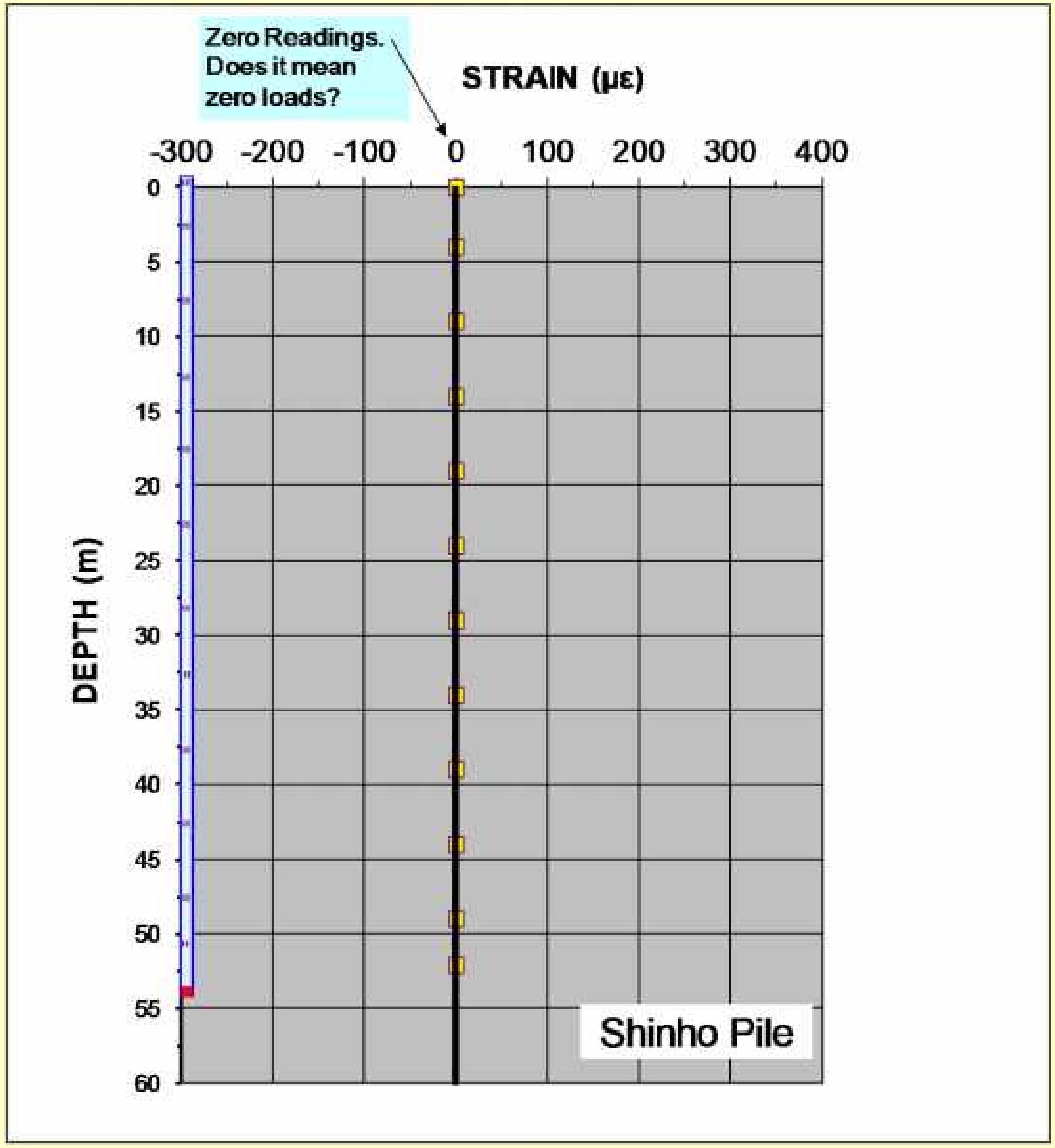
Toe and shaft movements are now shown after a moderate correction for residual force

Proceedings of ETC3
Symposium on Design of Piles in Europe
Leuven, Belgium, April 2016

*) **Triple mentioning of "base" will not negate the fact that the "10-%" definition of capacity is applied to a displacement that can be up to 100 % wrong!**

Residual Force

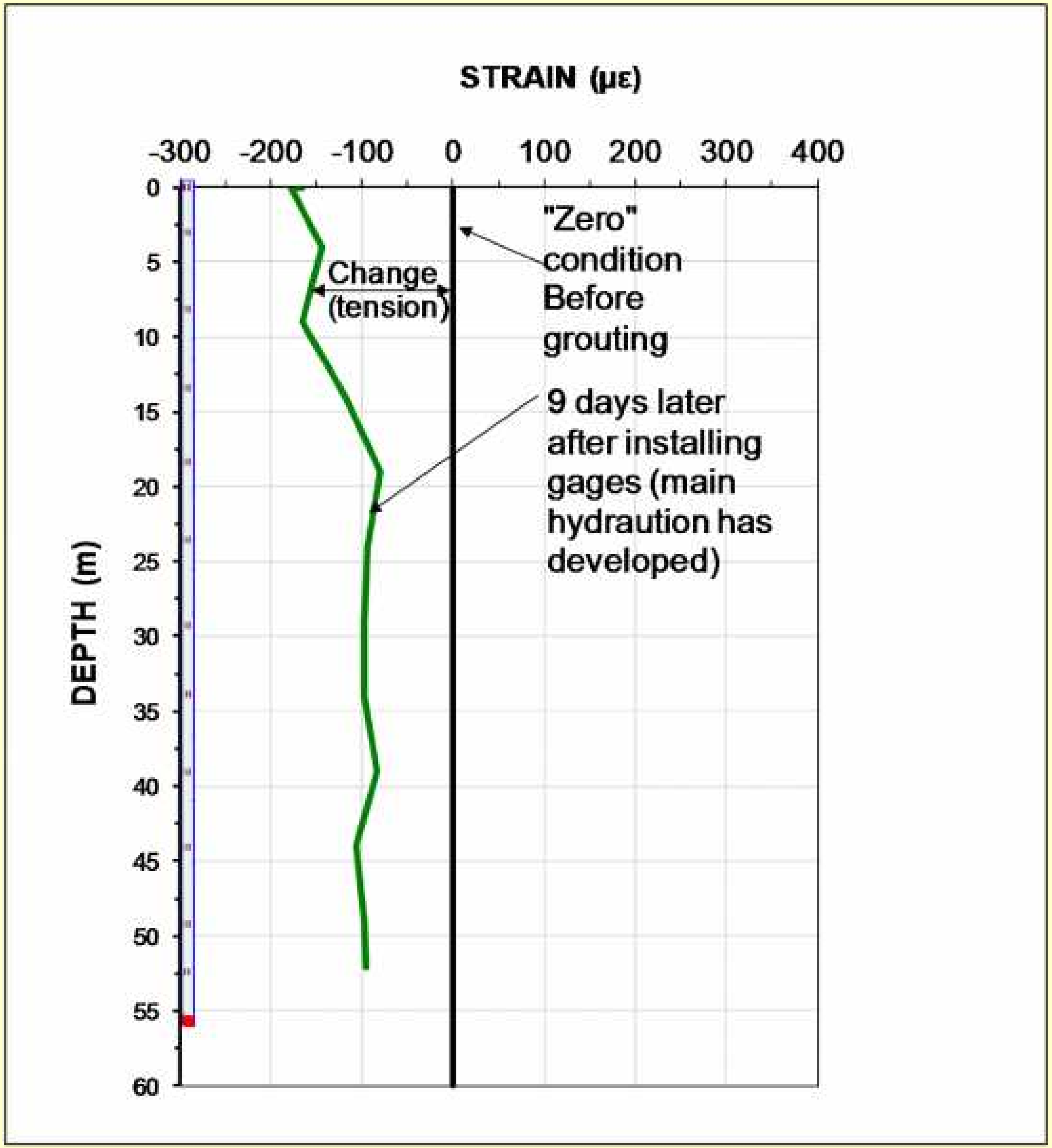
Head-down loading test on a 600 mm diameter, 54 m long cylinder pile in Busan, Korea, 2006.
(Kim, Chung, and Fellenius, 2011)



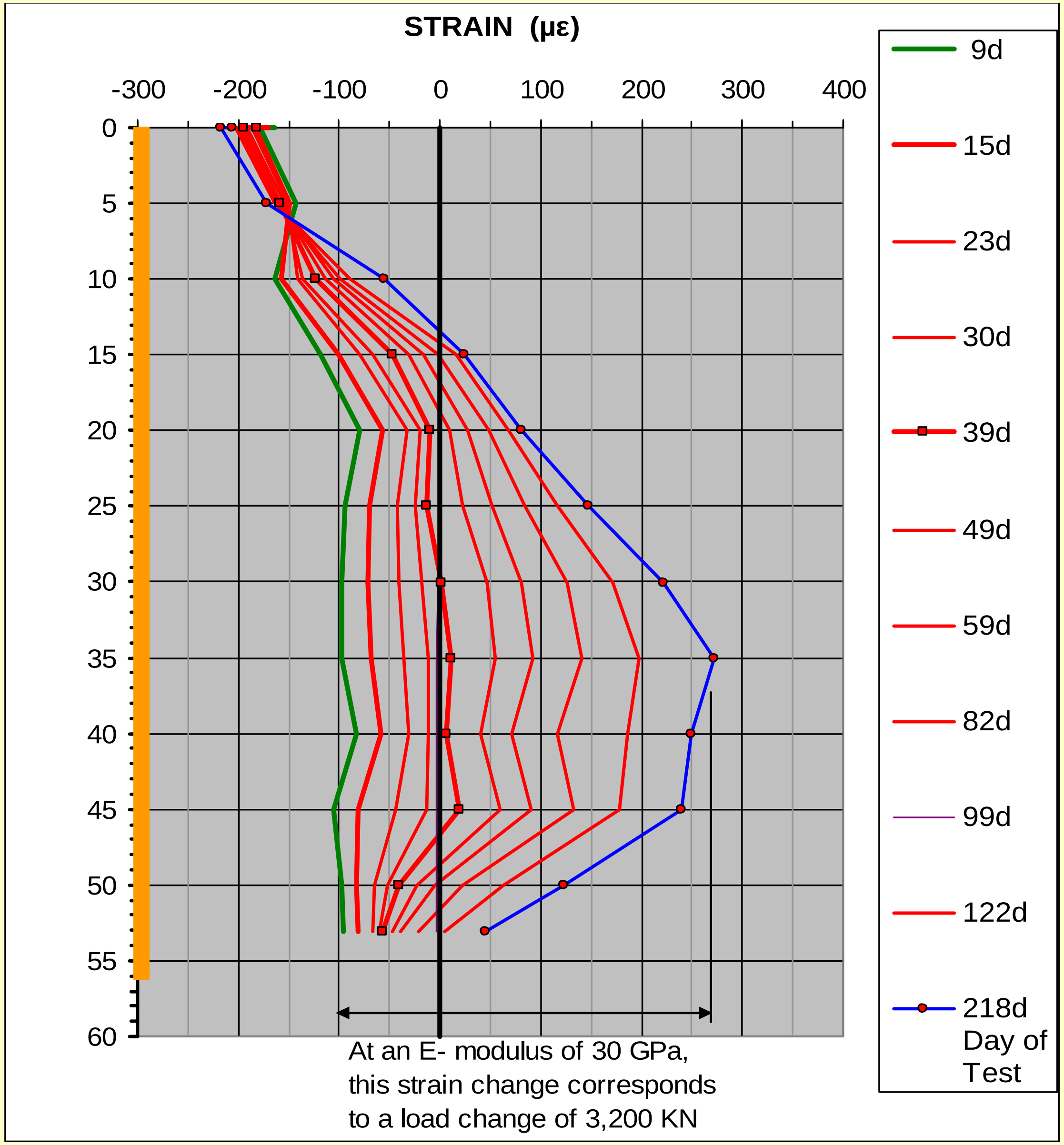
Answer to the question in the graph:

No, there's always residual force (axial) in a test pile.

Gages were read after they had been installed in the pile (= "zero" condition) and then 9 days later   (= green line) after the pile had been concreted and most of the hydration effect had developed.



Strains measured during the following additional 209-day wait-period.



The bi-directional test

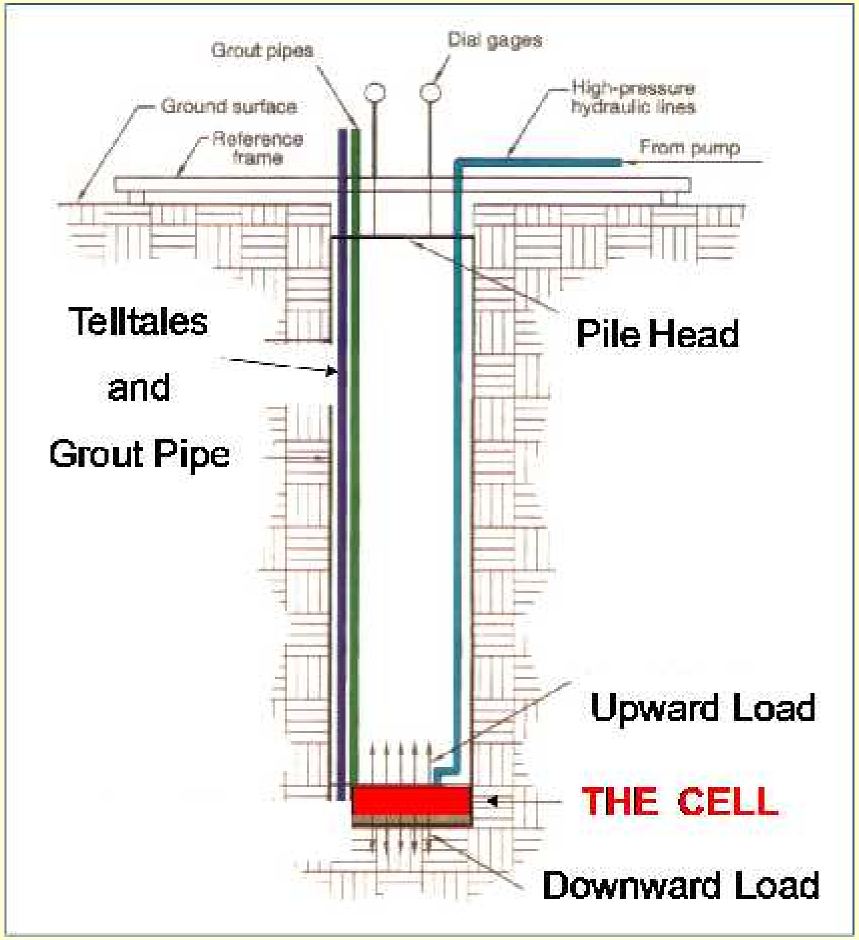
The difficulty associated with wanting to know the pile-toe load-movement response, but only knowing the pile-head load-movement response, is overcome in the bidirectional test, which incorporates one or more sacrificial hydraulic jacks placed at or near the toe (base) of the pile to be tested (be it a driven pile, augercast pile, drilled-shaft pile, precast pile, pipe pile, H-pile, or a barrette). Early bidirectional testing was performed by Gibson and Devenny (1973), Horvath et al. (1983), and Amir (1983). About the same time, an independent development took place in Brazil (Elisio 1983; 1986), which led to an industrial production offered commercially by Arcos Egenharia Ltda., Brazil, to the piling industry. In the 1980s, Dr. Jorj Osterberg also saw the need for and use of a test employing a hydraulic jack arrangement placed at or near the pile toe (Osterberg 1989) and established a US corporation called Loadtest Inc. to pursue the bi-directional technique. On Dr. Osterberg's in 1988 learning about the existence and availability of the Brazilian device, initially, the US and Brazilian companies collaborated. Somewhat unmerited, outside Brazil, the bidirectional test is now called the "Osterberg Cell test" or the "O-cell test" (Osterberg 1998). During the about 30 years of commercial application, Loadtest Inc. has developed a practice of strain-gage instrumentation in conjunction with the bidirectional test, which has vastly contributed to the knowledge and state-of-the-art of pile response to load.

The bi-directional test

Arcos Egenharia
Bidirectional test at
Rio Negro Ponte
Manaus—Iranduba, Brazil

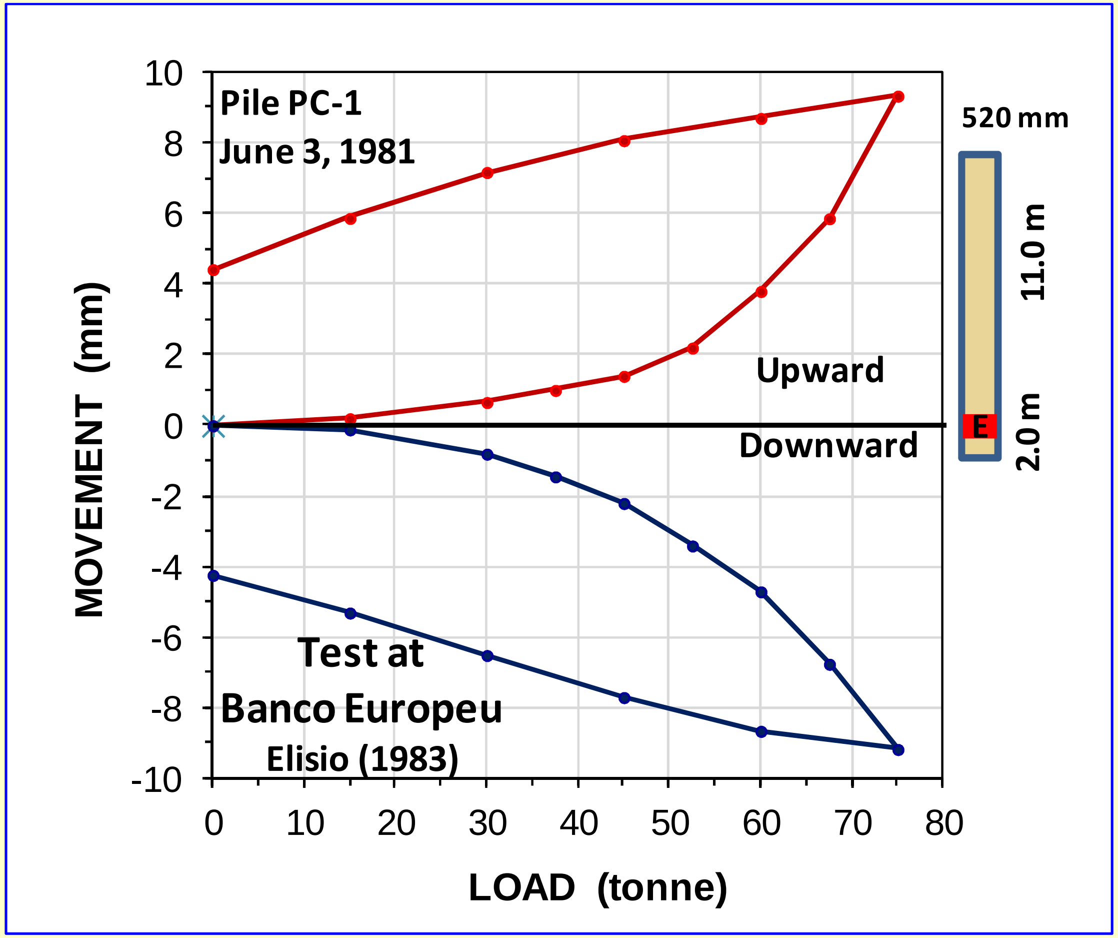
Schematics of the bidirectional test

(Meyer and Schade 1995)



Typical Test Results

(Data from Eliso 1983)

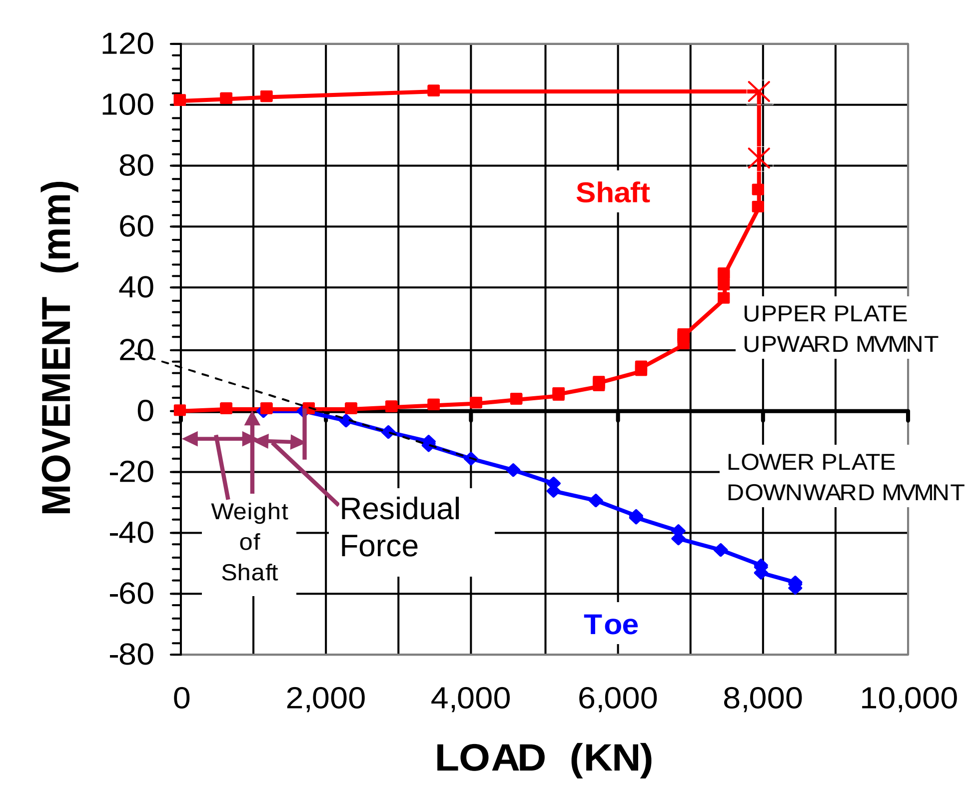


Inchon, Korea (Fugro Loadtest)

Sao Paolo, Brazil (Arcos Egenharia)

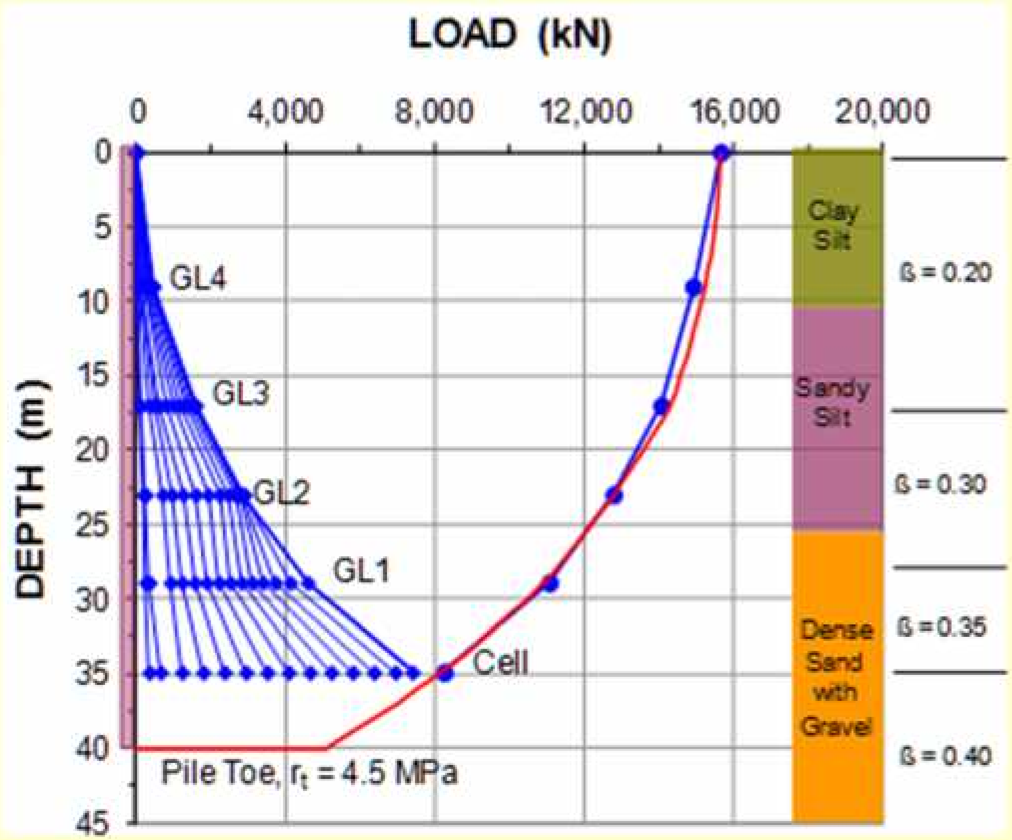
The bidirectional cell can also be installed in a driven pile. Here in a 600 mm cylinder pile (spun pile) with a 400 mm central void (installed after the driving).

Test on a 1,250 mm diameter, 40 m long, bored pile at US82 Bridge across Mississippi River installed into dense sand



Bidirectional **load-movement** curves

(The bidirectional jack is placed at the pile toe)

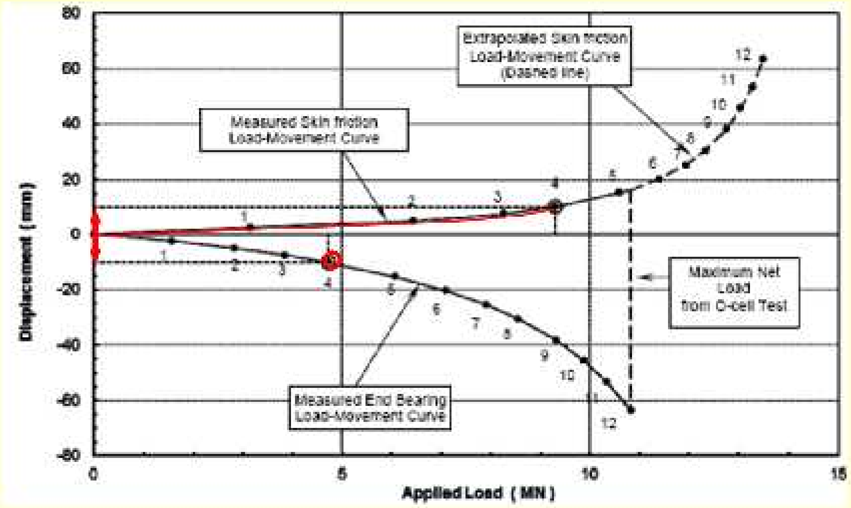


Bidirectional **load-distribution** as-measured and as-“flipped” over to show the equivalent head-down distribution. (The red curve is the curve fitted in a UniPile calculation applying effective stress analysis with the **<u>target</u>** beta-coefficients indicated to the right).

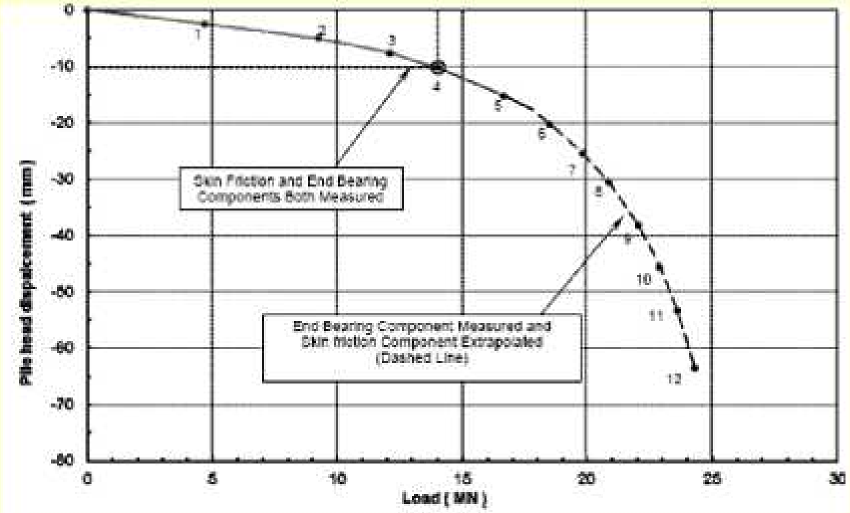
Test data from Fugro Loadtest 2002

The Equivalent Head-down Load-movement Curve

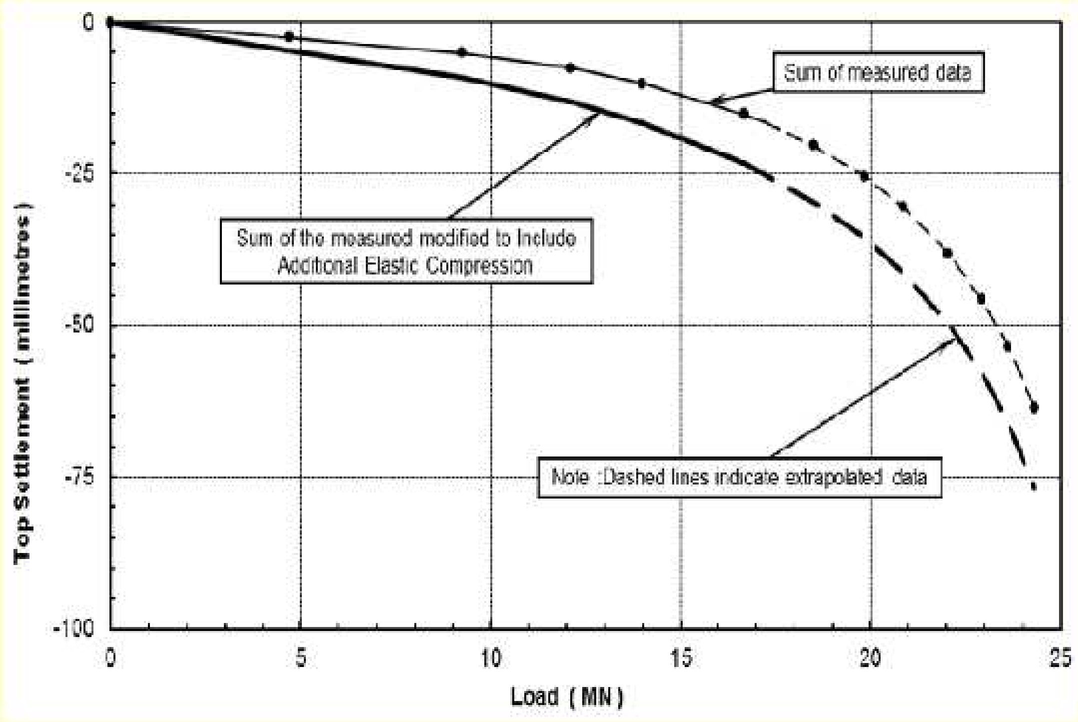
Measured upward and downward curves



Construction of the "Direct Equivalent Curve"



From the upward and downward results, one can produce the equivalent head-down load-movement curve, the curve that one would have obtained in a routine "Head-Down Test". The curve needs to be corrected for the increased pile compression in the head-down test.

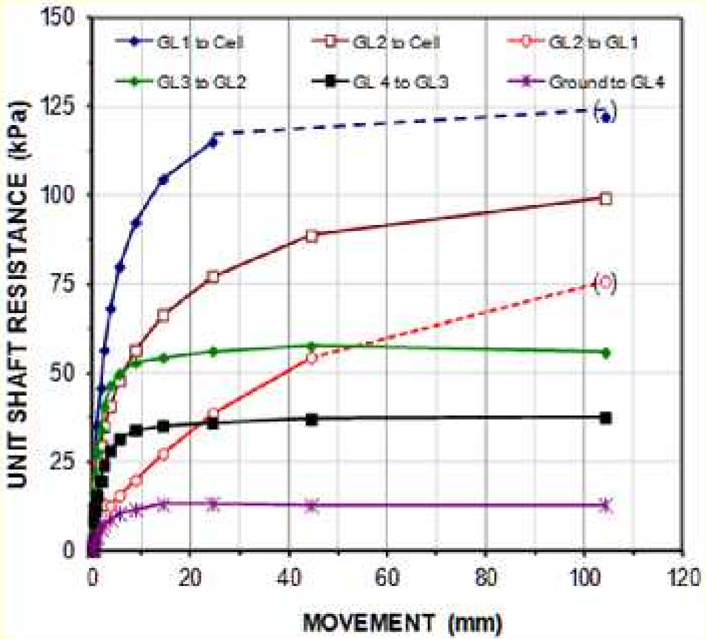


It also needs to be corrected for the fact that the bidirectional test engages the stiffer soil layers first, while the head-down test engages them last (addressed in the next two slides).

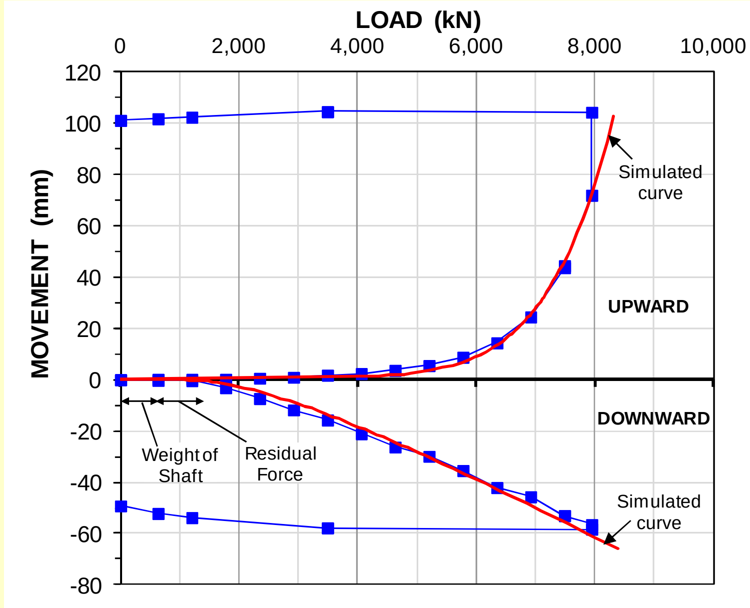
Reference: Appendix to regular reports by Fugro Loadtest Inc.

Cnt. Test on a 1,250 mm diameter, 40 m long, bored pile at US82 Bridge across Mississippi River

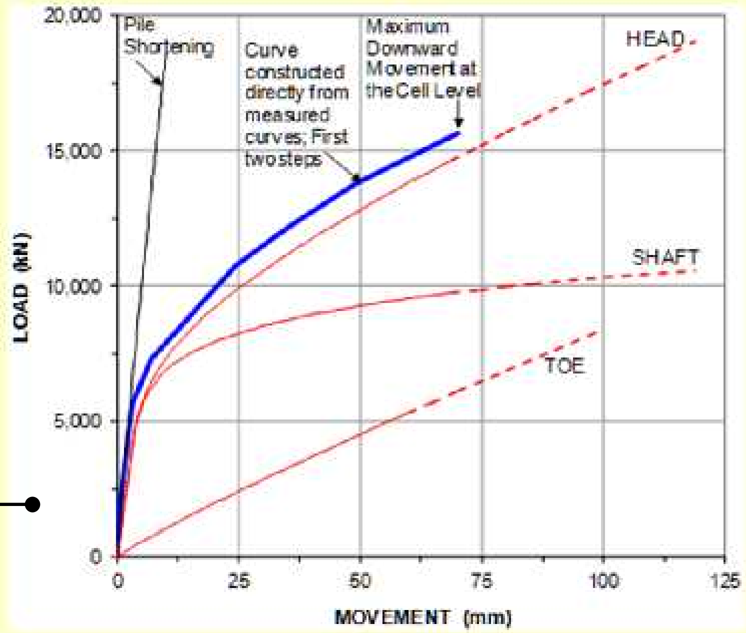
Unit shaft resistance vs. movement as evaluated from strain-gage records and telltale-measured movements.



The bidirectional load-movement curves (blue lines and square dots) simulated and fitted using UniPile (red lines).

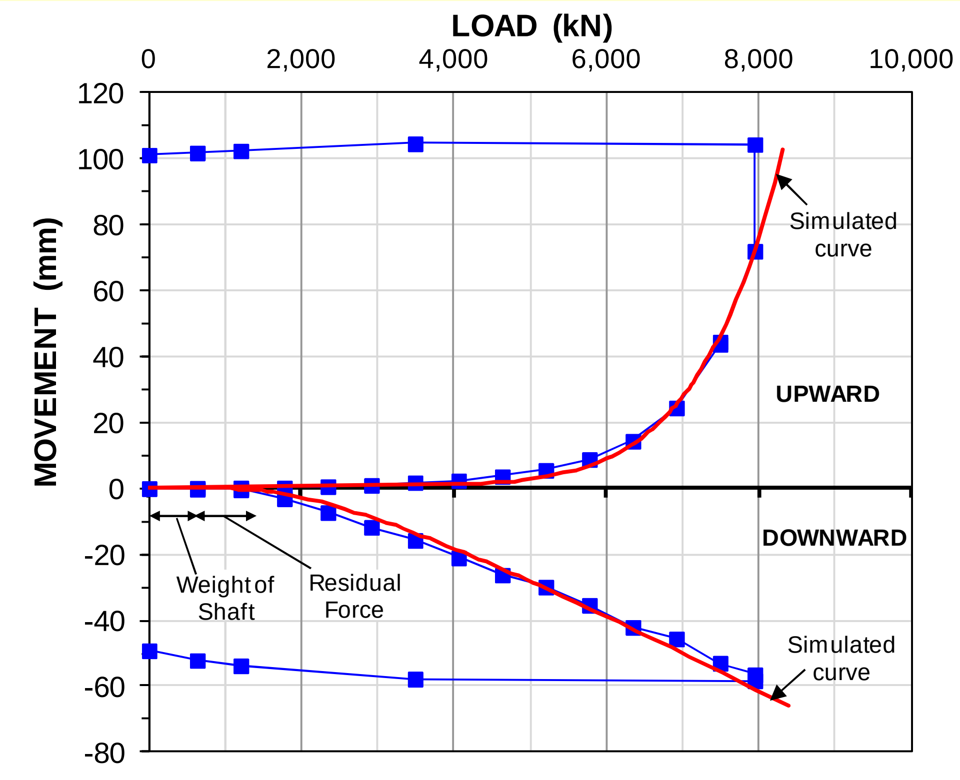


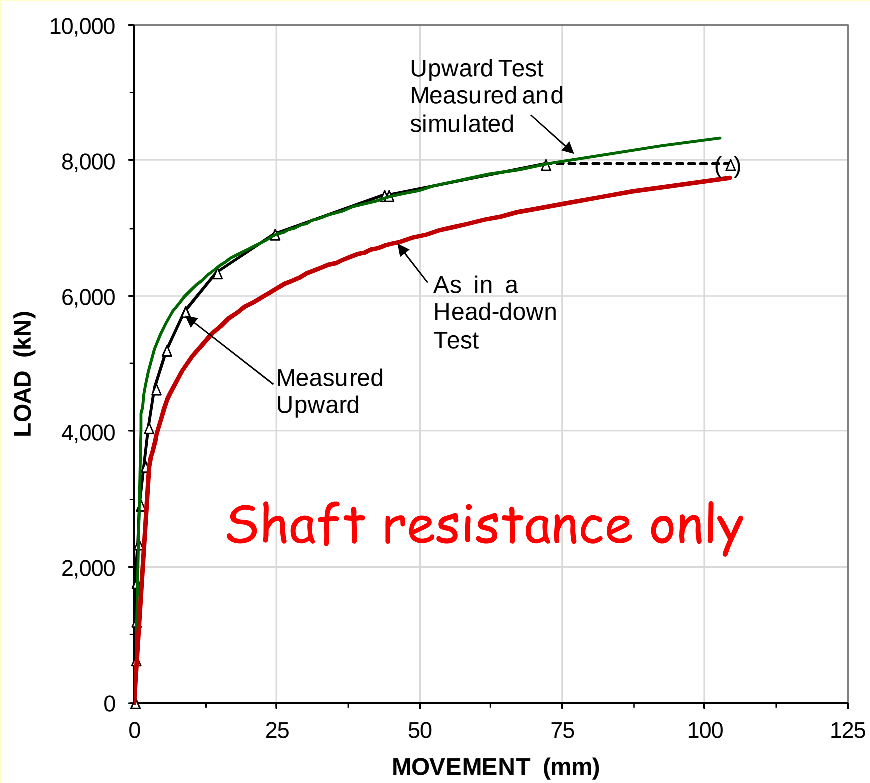
Once the simulation of the test is completed, the analysis can produce the equivalent head-down load-movement curves, applying the t-z and q-z functions resulting from the fit to the measured bidirectional curves.

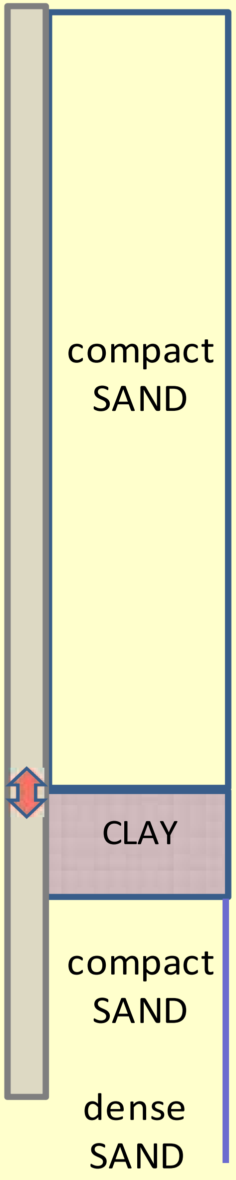


Fellenius (2015)

The difference between the measured upward cell movement and the simulated Equivalent Head-down load-movement for the pile length above the bidirectional cell is due to the fact that the upward cell engages the lower soil first, whereas the head-down test engages the upper soils first, which are less stiff than the lower soils. The left graph shows the measured and simulated <u>upward and downward</u> curves (same as on previous slide) The right graph shows the <u>upward</u> curve as measured and as simulated (in a 1st quarter plot) and compared to an Equivalent Head-down test (shaft resistance only) on the pile using the t-z curves fitted to the measured test.





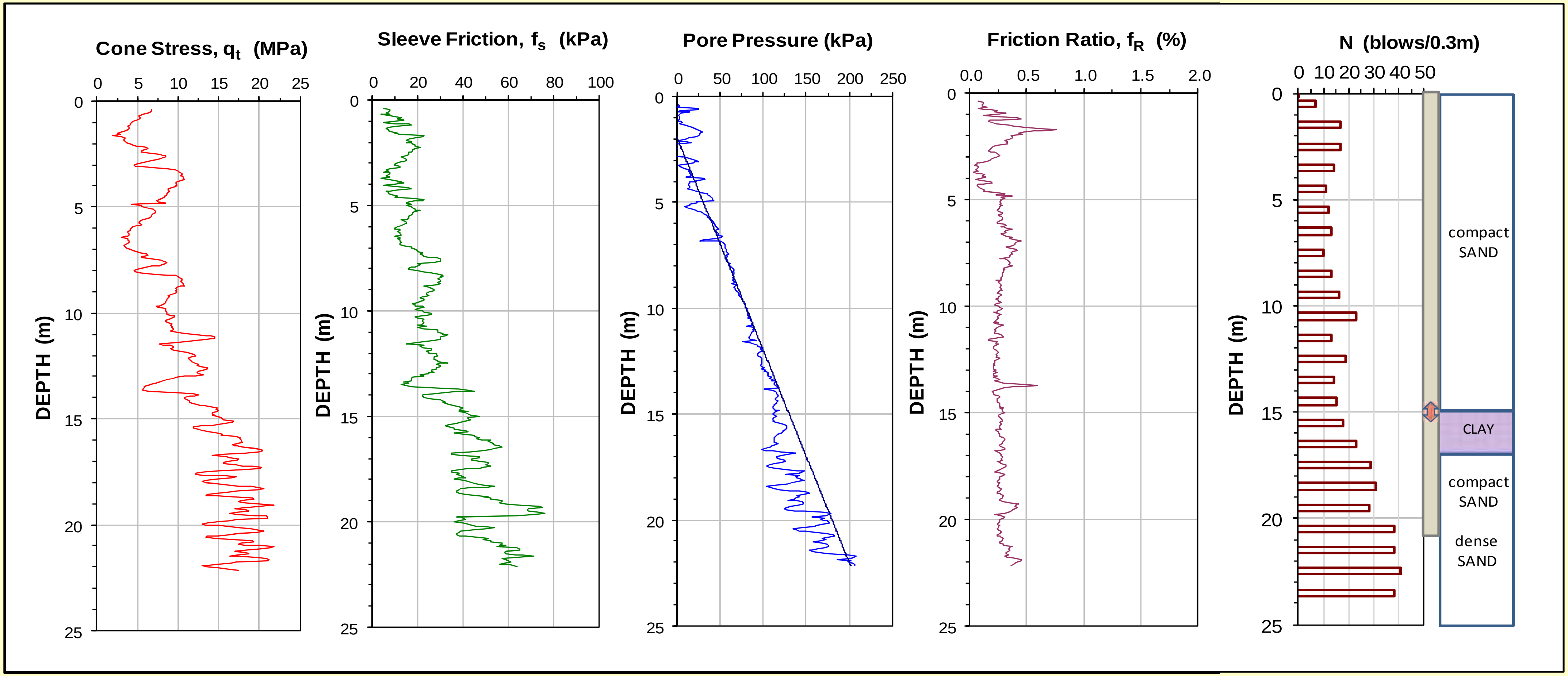


The sand becomes very dense at about 25 m depth

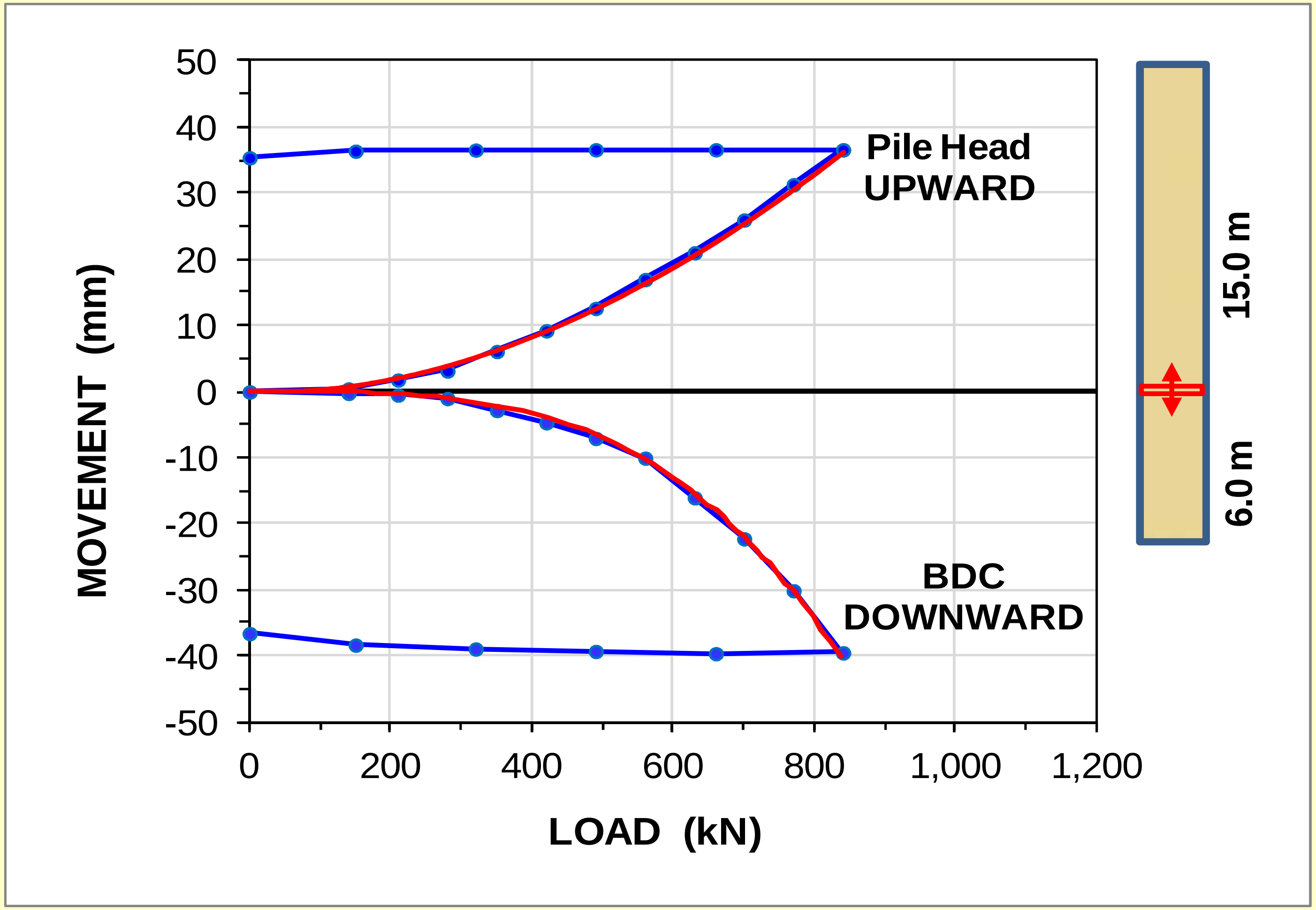
Analysis of the results of a bidirectional test on a 21m long bored pile in Sao Paolo, Brazil

A bidirectional test was performed on a 500-mm diameter, 21 m long, bored pile constructed through compact to dense sand by driving a steel-pipe to full depth, cleaning out the pipe, while keeping the pipe filled with betonite slurry, withdrawing the pipe, and, finally, tremie-replacing the slurry with concrete. The bidirectional cell (BDC) was attached to the reinforcing cage inserted into the fresh concrete. The BDC was placed at 15 m depth below the ground surface.

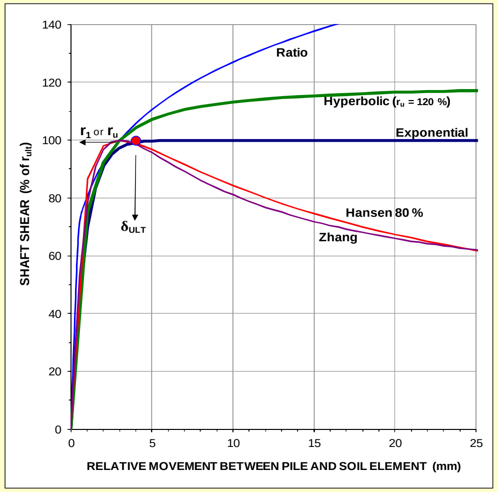
The soil profile determined by CPTU and SPT



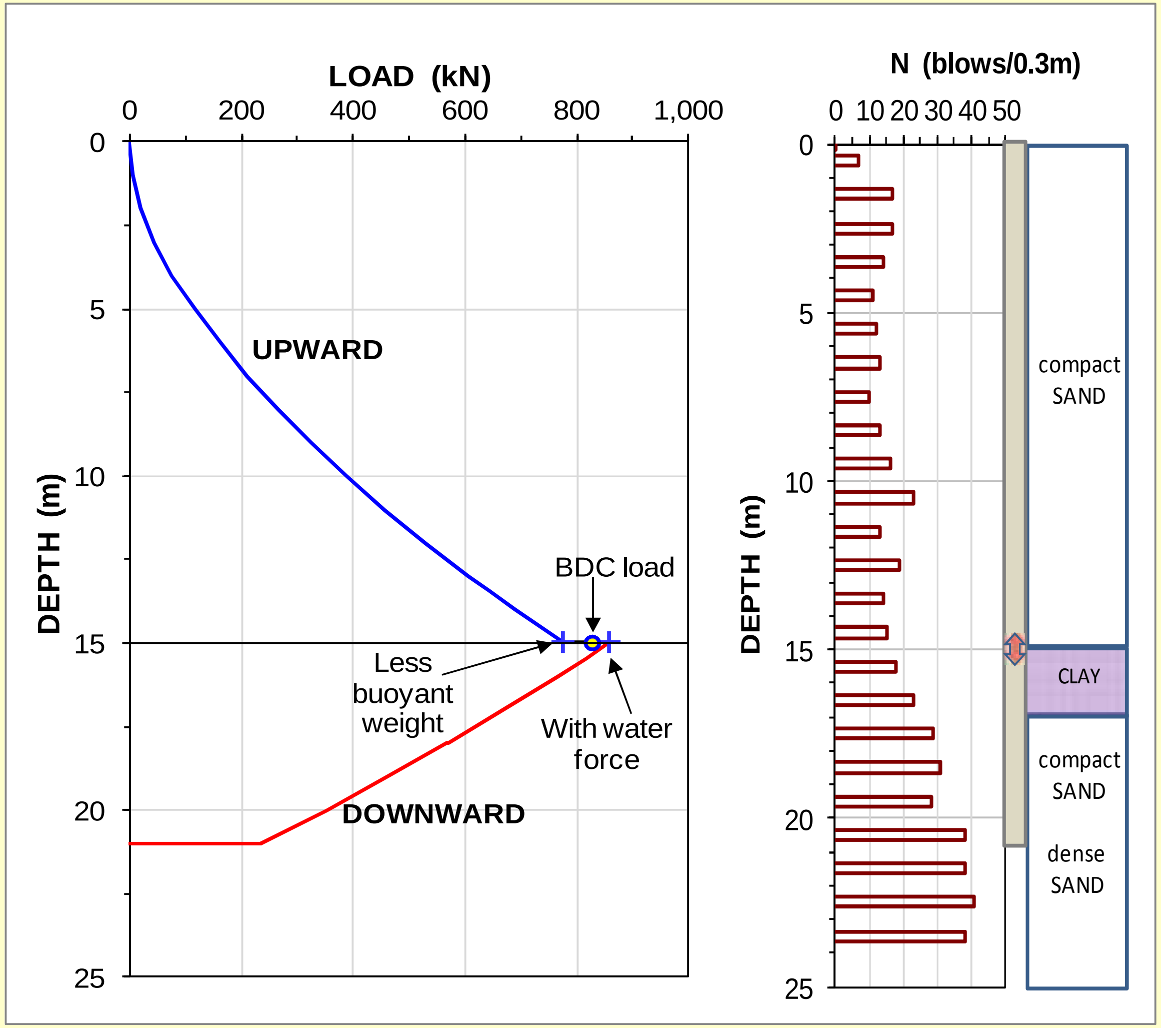
The final fit of simulated curves to the measured



Simulation performed by effective stress back-calculation
with input of t-z and q-z curves to the UniPile software
(see Slide 23)

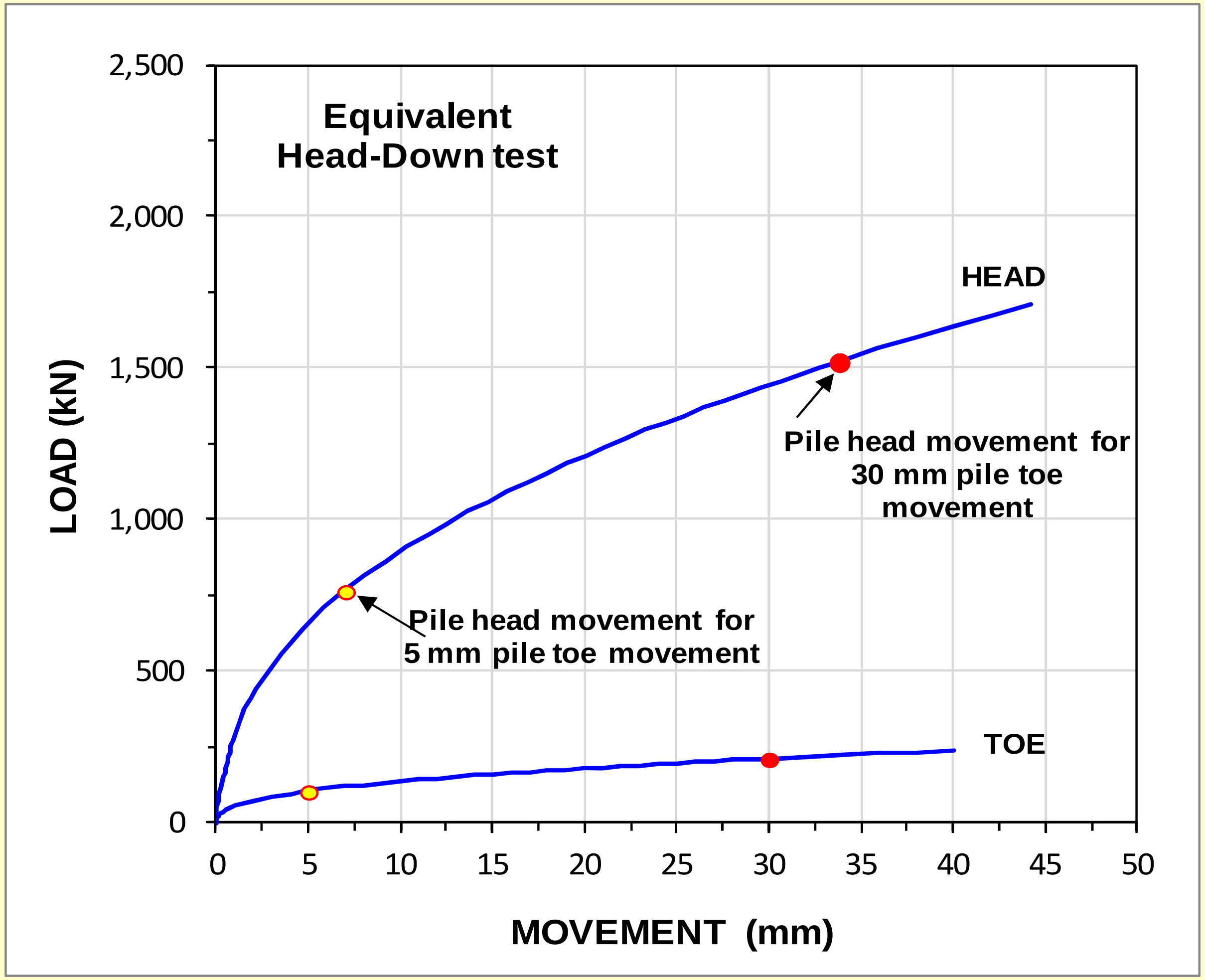


The test pile was not instrumented. Had it been, the load distribution of the bidirectional test as determined from the gage records, would have served to further detail the evaluation results. Note the below adjustment of the BDC load for the buoyant weight (upward) of the pile and the added water force (downward).



The analysis and results of the simulation appear to suggest that the pile is affected by a filter cake along the shaft. It has probably also a reduced toe resistance due to debris having collected at the pile toe between final cleaning and the placing of the concrete.

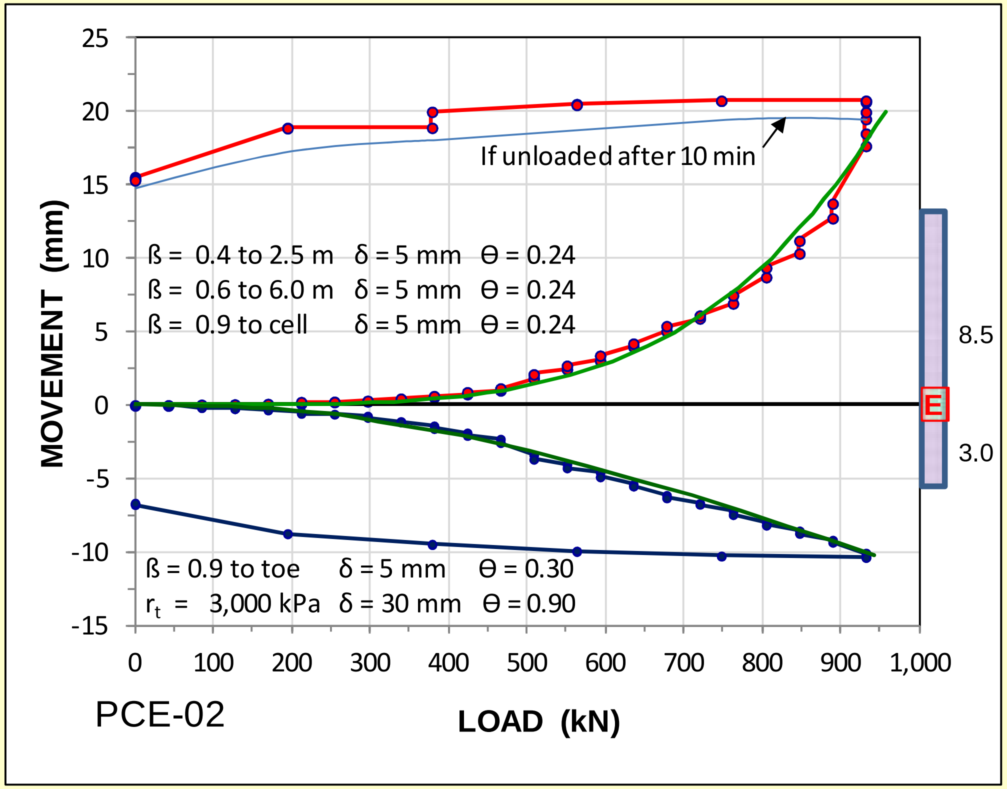
The final fit establishes the soil response and allows the equivalent head-down loading- test to be calculated

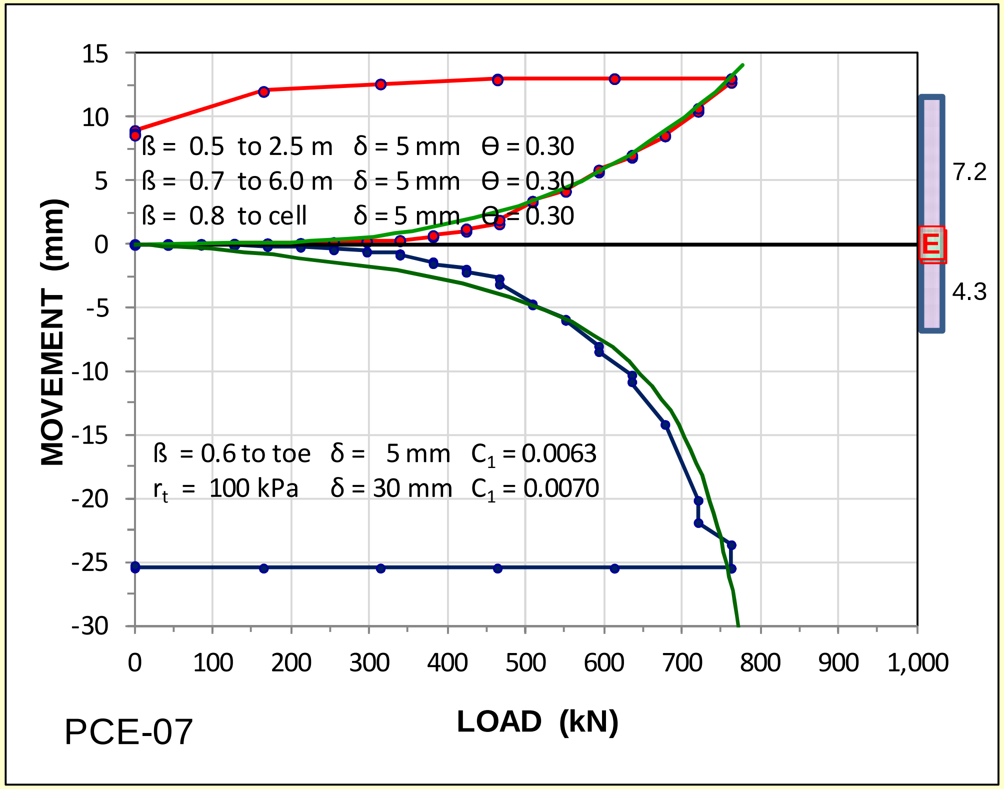


When there is no obvious point on the pile-head load-movement curve, the "capacity" of the pile has to be determined by one definition or other—there are dozens of such around. I prefer to define it as the pile-head load that resulted in a 30-mm pile toe movement. As to what safe working load to assign to a test, it often fits quite well to the pile head load that resulted in a 5-mm toe movement. In this case, that happens to be at Q = 750 kN.

The most important aspect for a safe design is not the "capacity" found from the test data, but what the settlement of the structure supported by the pile(s) might be. How to calculate the settlement of a piled foundation is not addressed here, however.

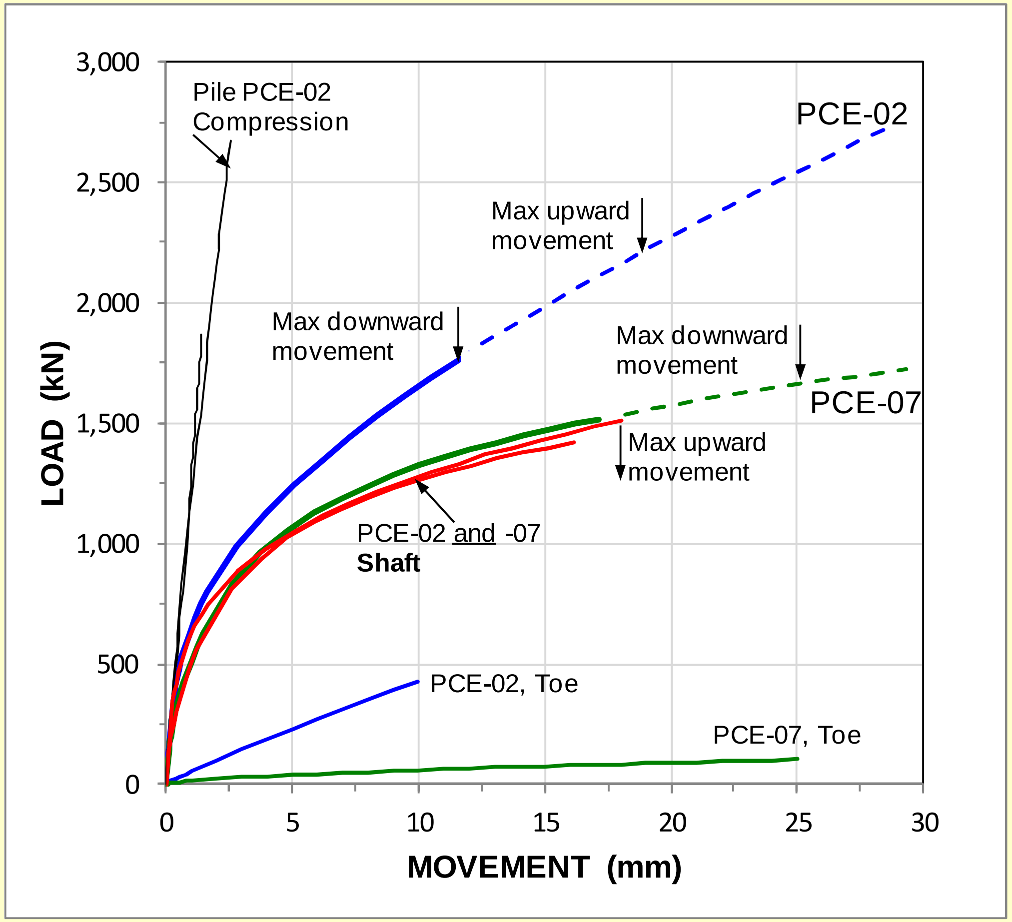
A CASE HISTORY Bidirectional tests performed at a site in Brazil on two Omega Piles (Drilled Displacement Piles, DDP, also called Full Displacement Piles, FDP) both with 700 mm diameter and embedment 11.5 m. Pile PCE-02 was provided with a bidirectional cell level at 7.3 m depth and Pile PCE-07 at 8.5 m depth.



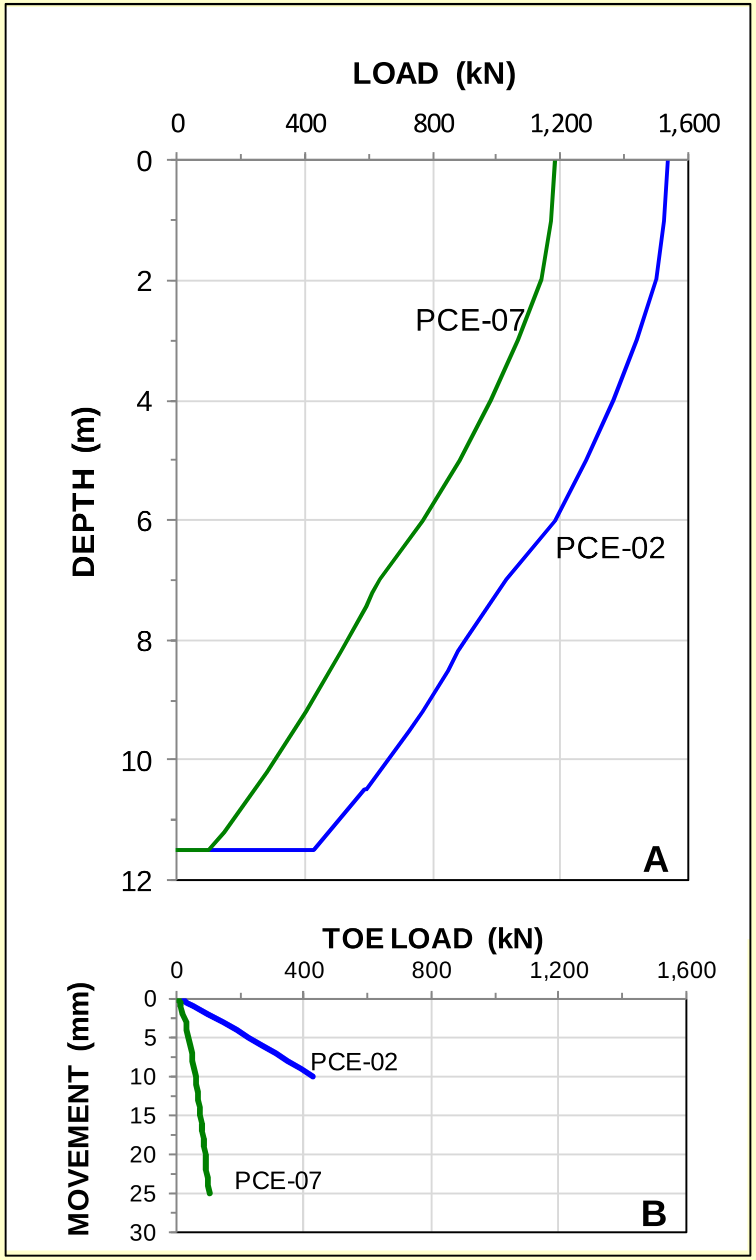


After data reduction and processing

Equivalent Head-down
Load-movements

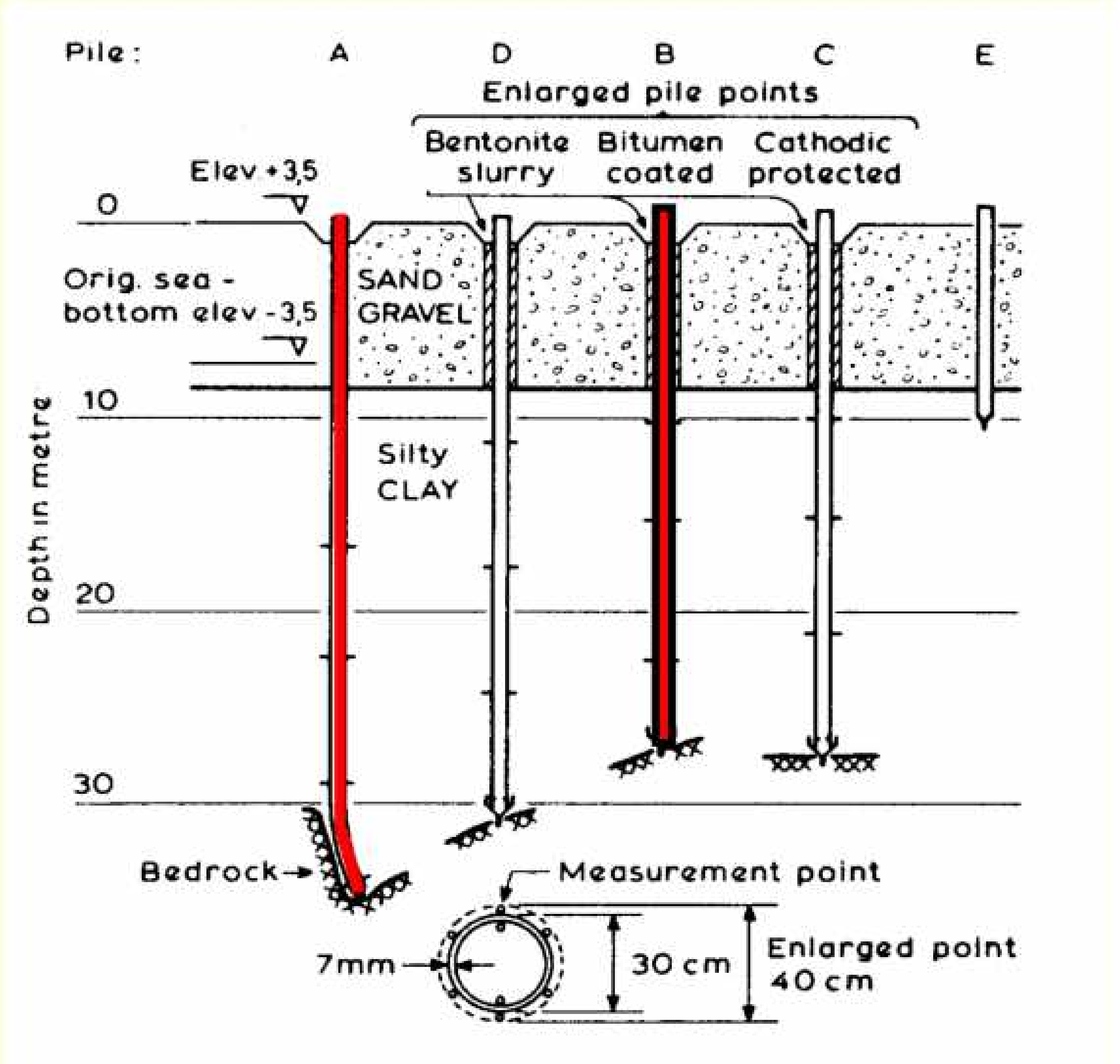


Equivalent Head-down
Load-distributions

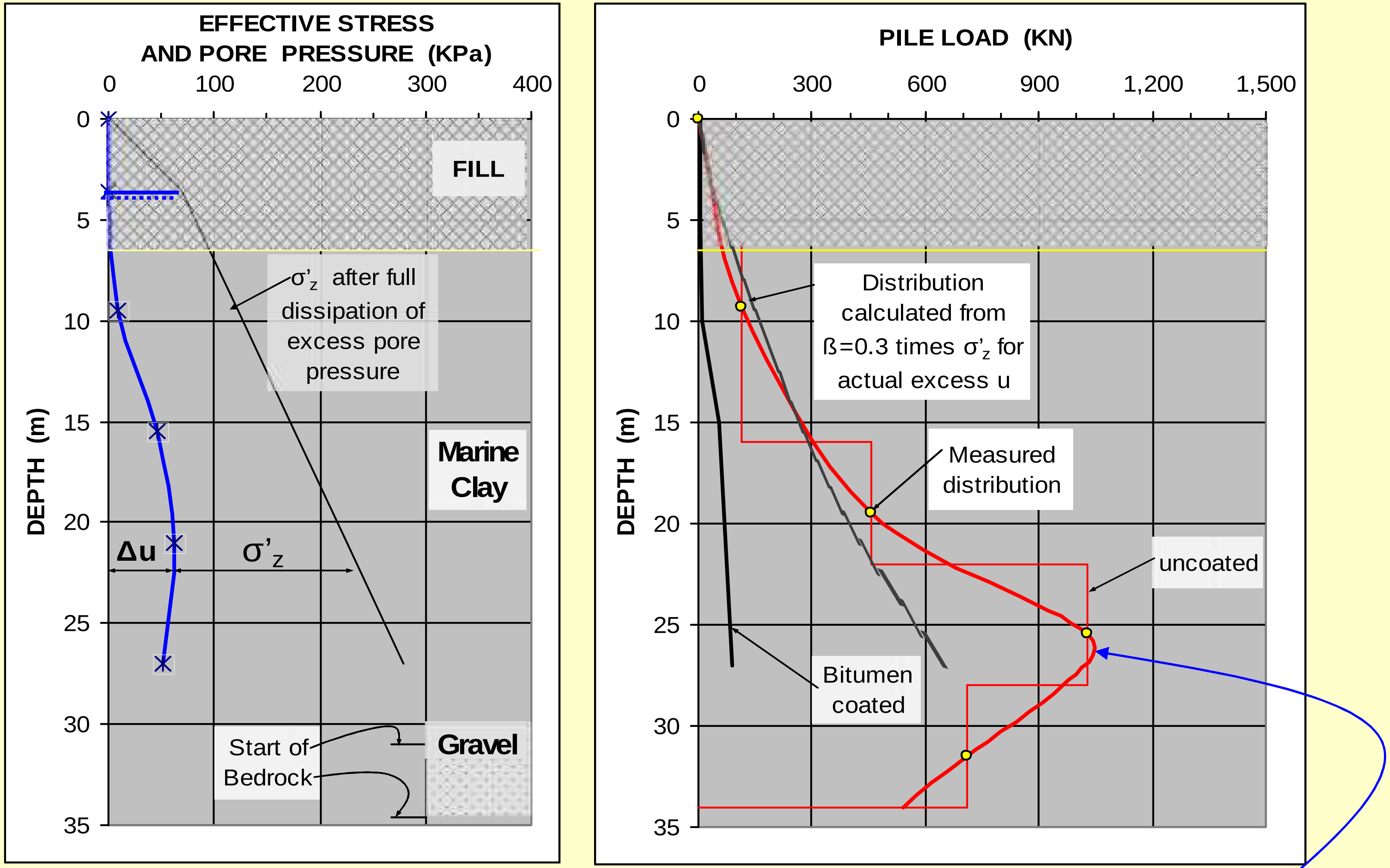


A conventional head-down test would not have provided the reason for the lower "capacity" of Pile PCE-02

The ever so scary N.S.F ghost —the drag force



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, pile shortening, and load distributions. Heröya site. (Data from Bjerrum et al., 1969).

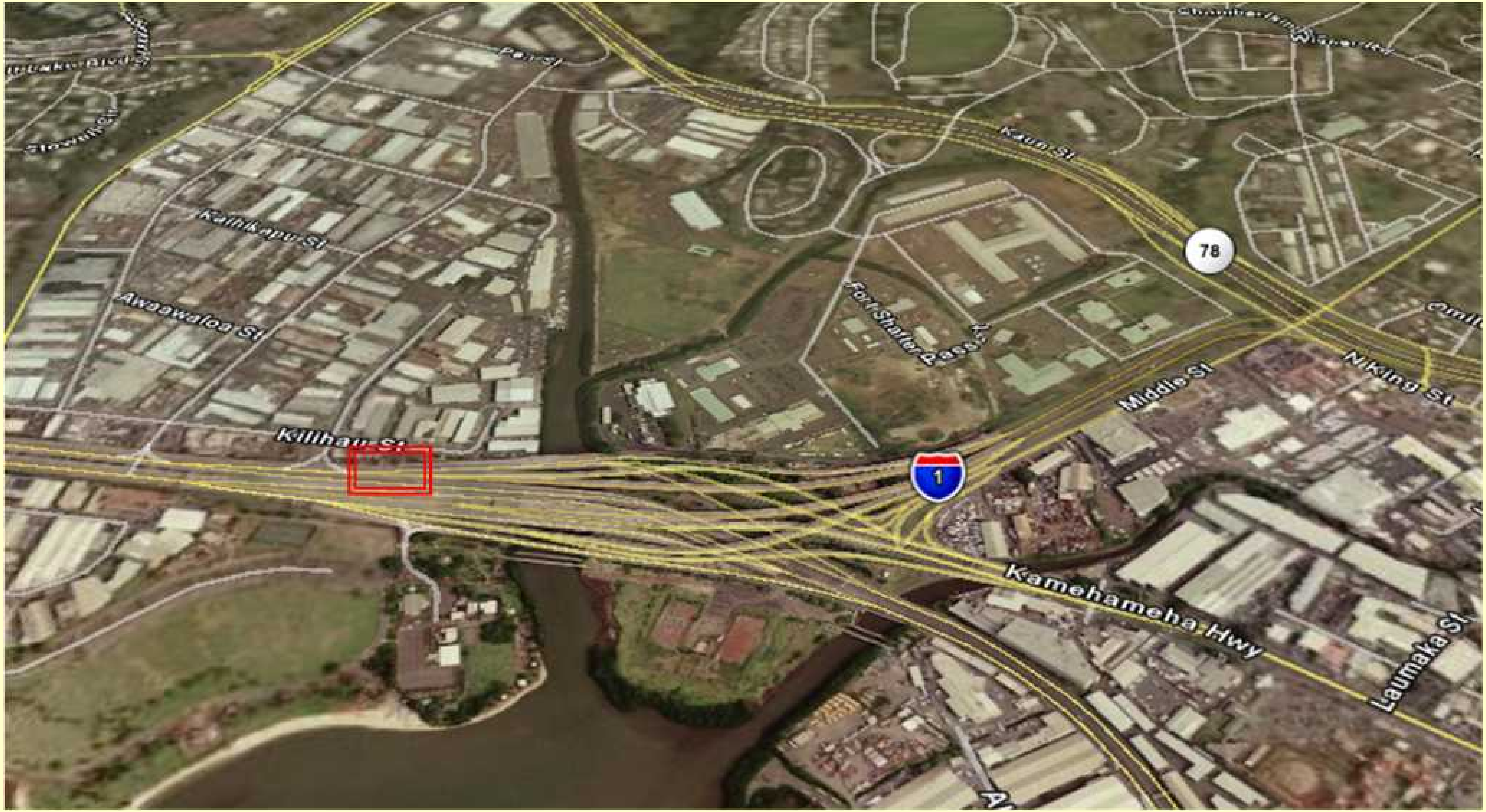
Compilation of Norwegian results

Table 1 Results of previous tests on unprotected steel piles.

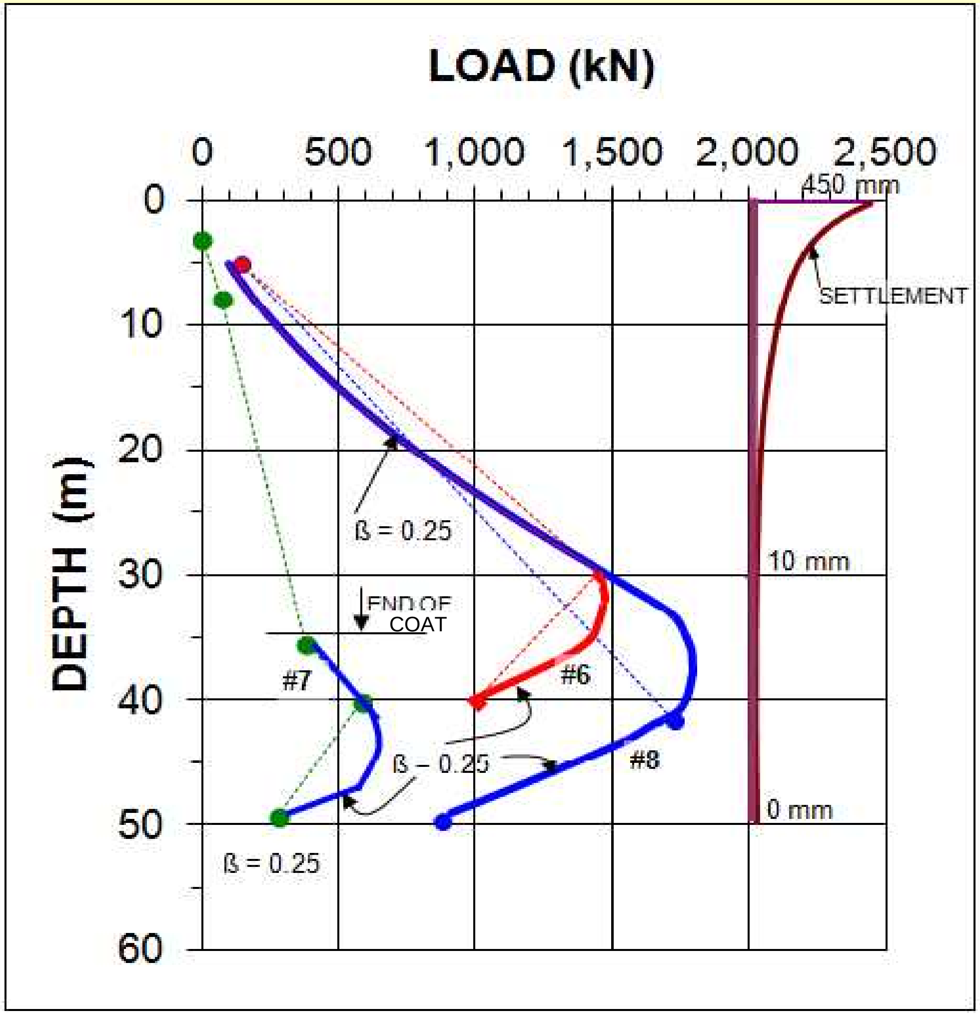
Site / Pile No.	Pile data type	Pile data length m	Time after driving years	Drag Force tons	Settlement during observation ground cm	Settlement during observation pile cm	K tan φ'
Sörenga							
B	I	53	5	≈400	≈ 200	10.0	0.20
C	II	57	2	300	≈ 27	5.3	0.18
G	II	41	2	250	≈ 7	3.2	0.23
Heröya							
85	III	32	$1\frac{1}{2}$	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.

FHWA Project, Keehi Interchange, Honolulu,Hawaii 1977



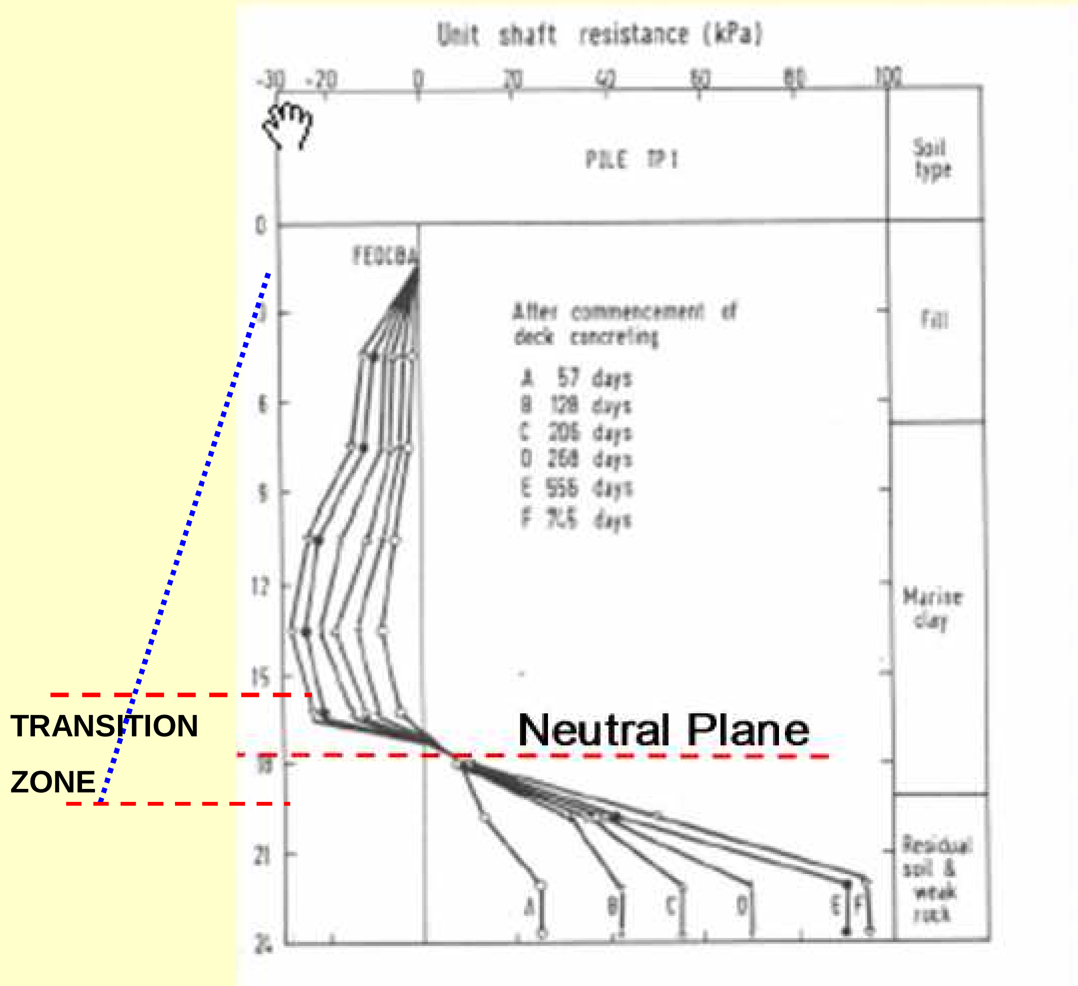
Results from 3 years of monitoring of a 30-m pile, 50-m pile, and a 30-m bitumen coated pile



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 force
3/16 inch = 4 mm

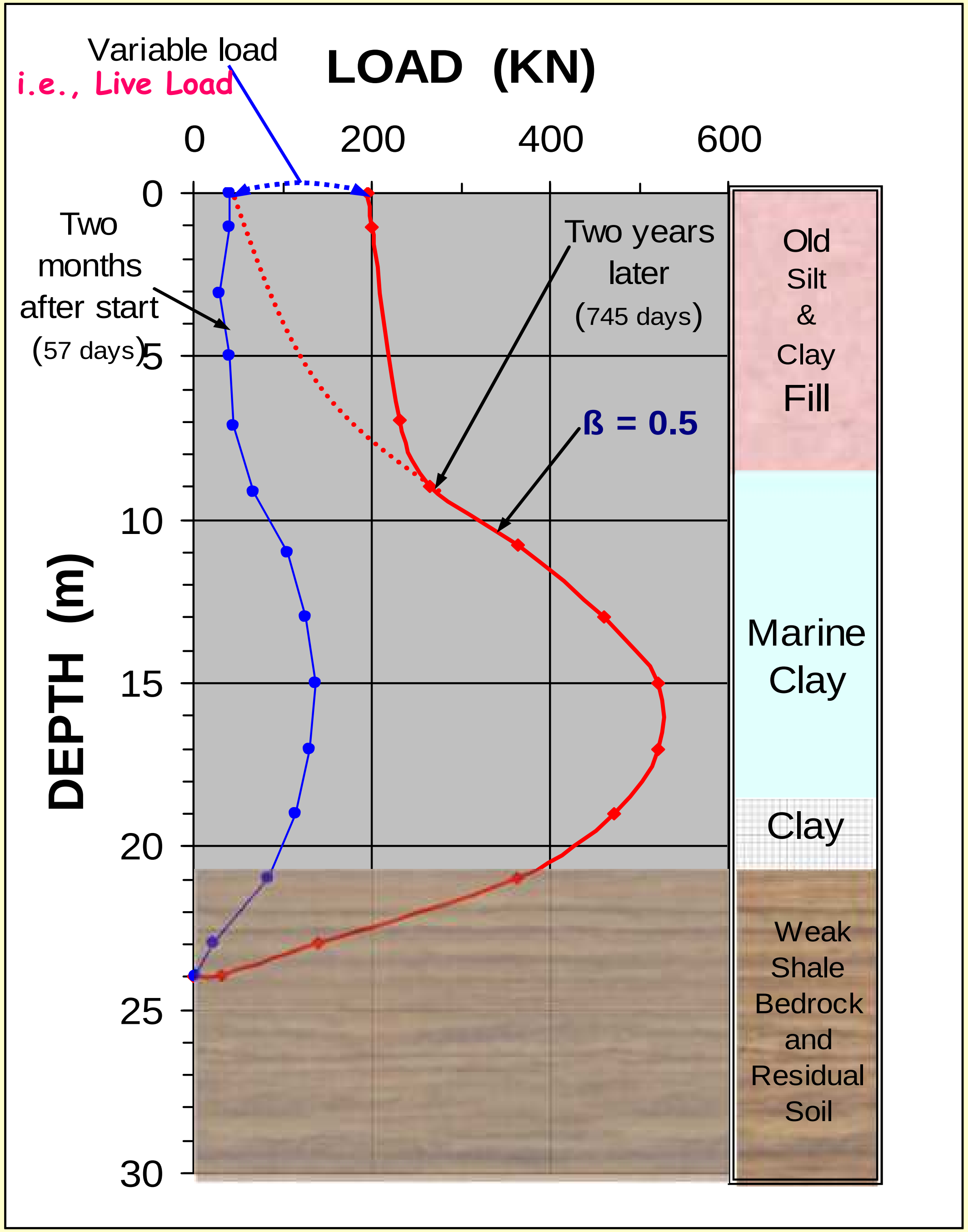
Leung, C.F, Radhakrishnan, R., and Tan Siew Ann (1991) presented a case history on instrumented 280 mm square precast concrete piles driven in **marine clay** in Singapore

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



Distribution of Unit Shaft Resistance along Pile TP1 under Service Loads

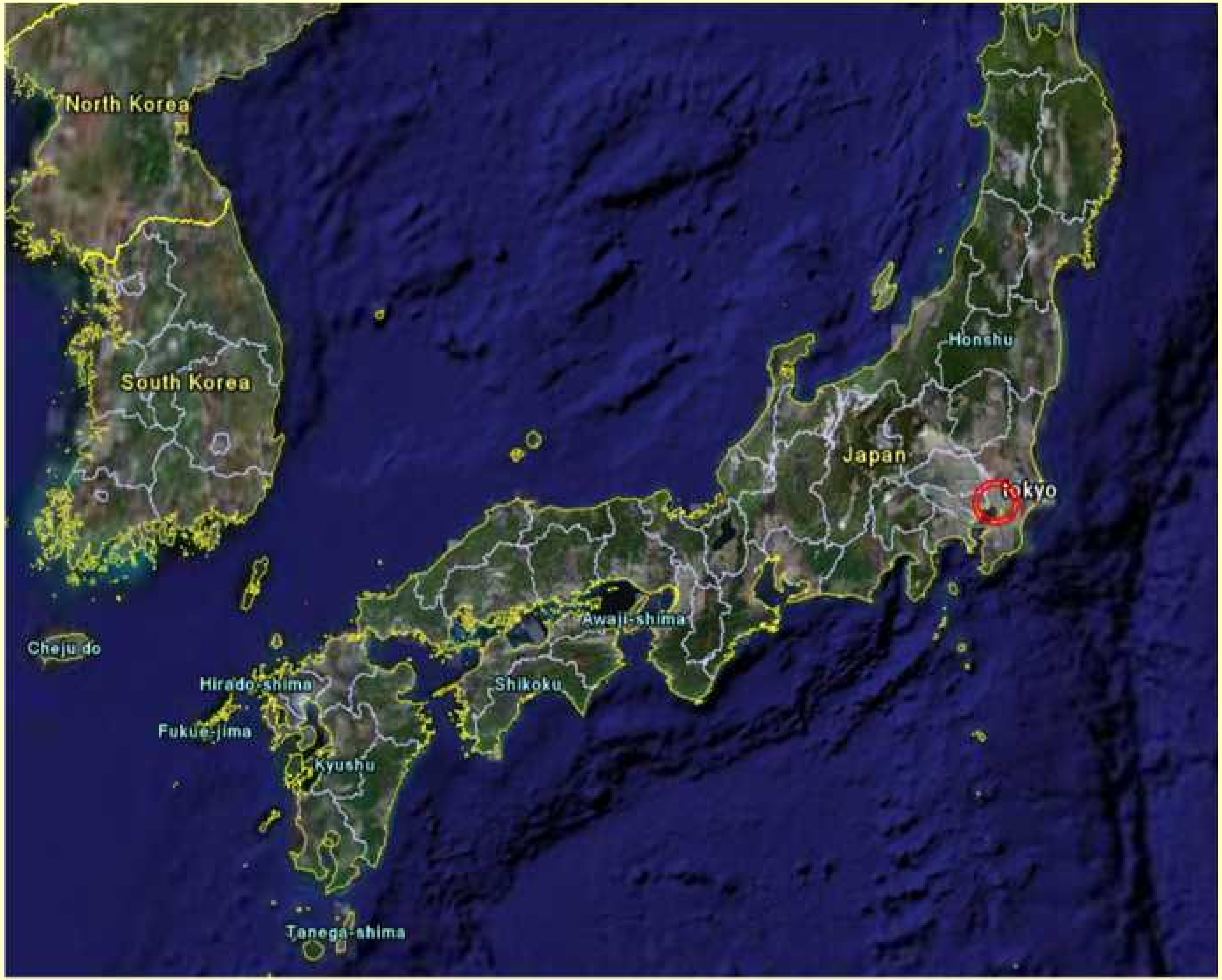
Live load and drag force cannot coexist!

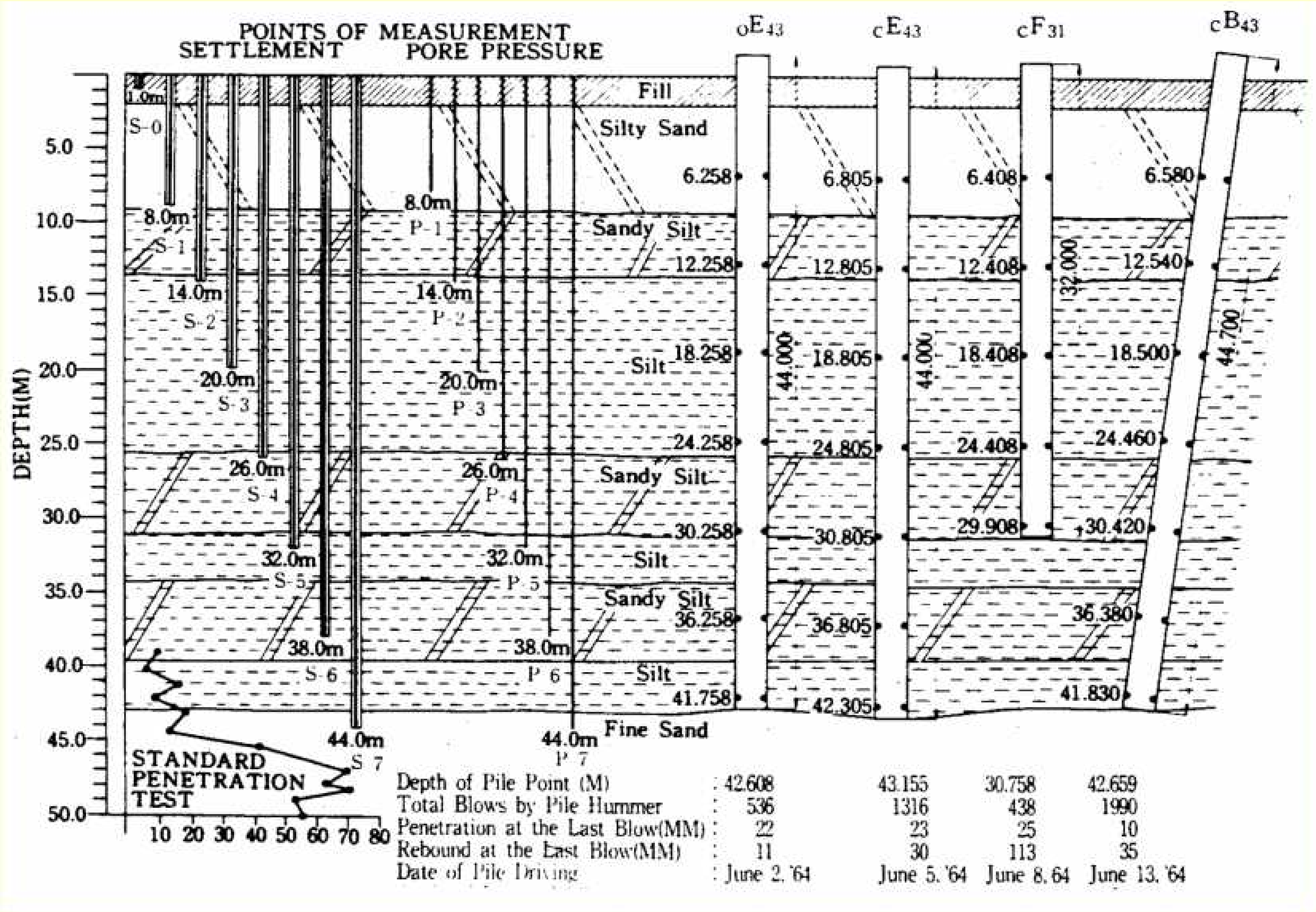


Data from Leung, Radhakrishnan, and Tan (1991)

2

Endo et al. 1969, presented a very ambitious drag-force study in Japan 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.



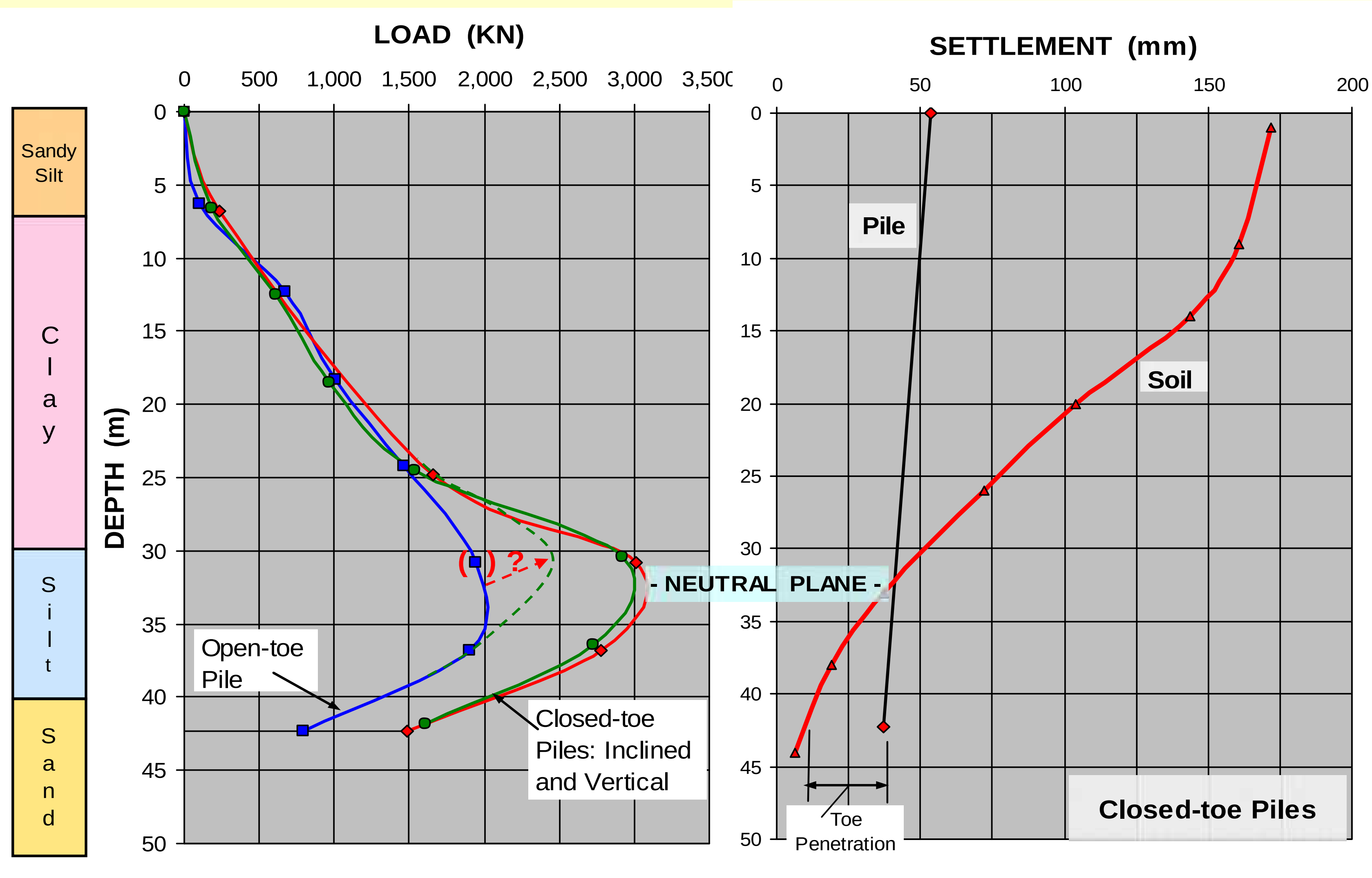


Profile of test site and piles

Closed-toe, Open-toe, Inclined, and Short Pile

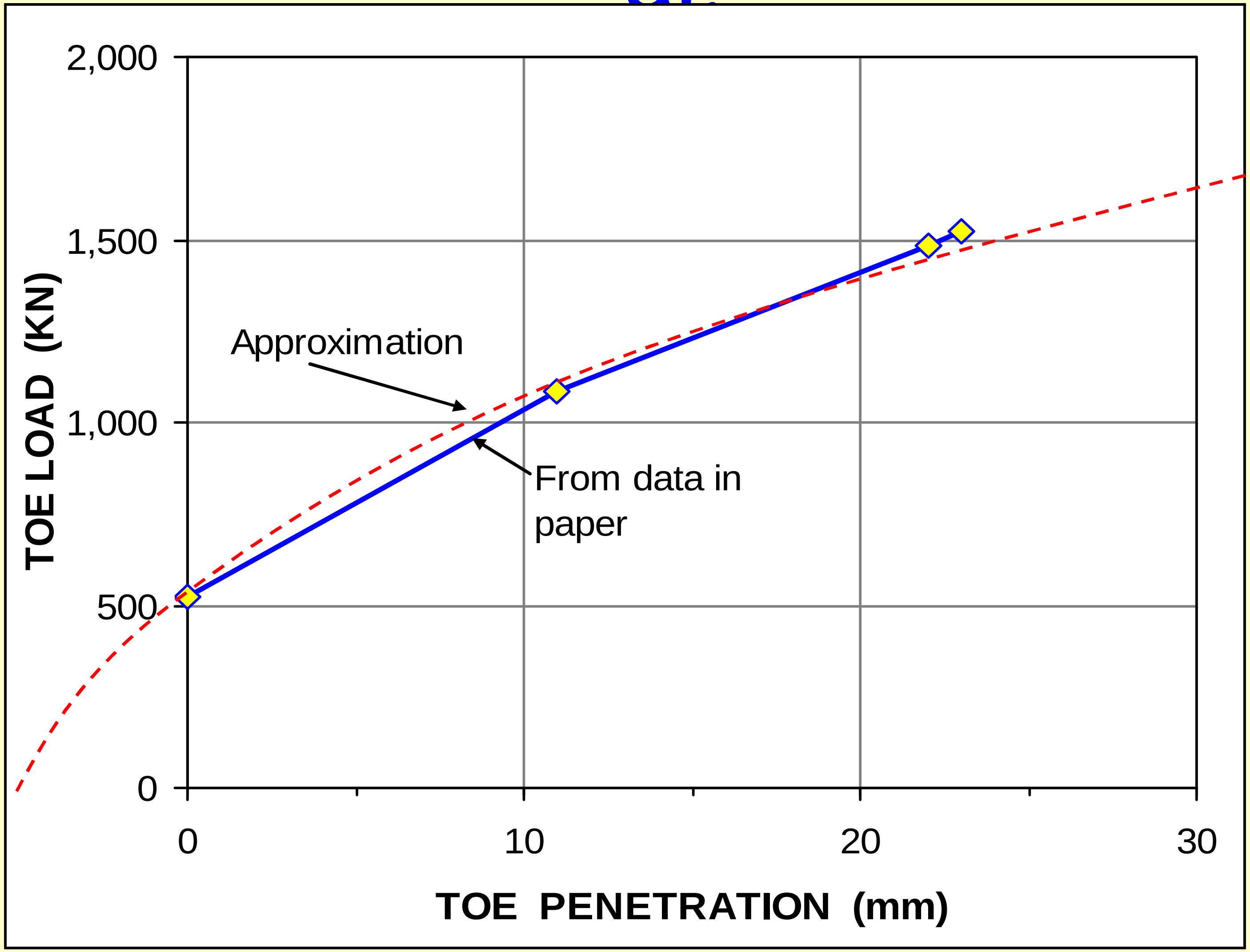
(Endo et al., 1969)

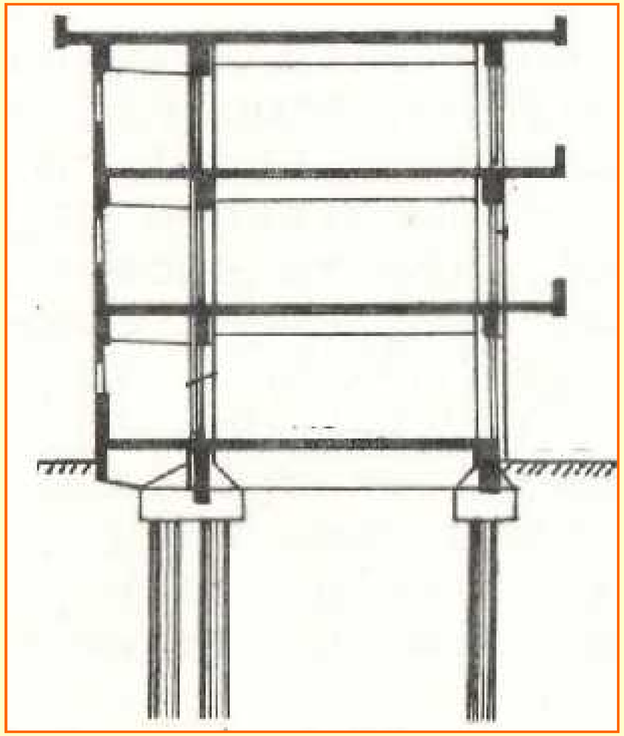
Neutral plane = Force Equilibrium = Settlement Equilibrium



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

Toe forces and toe penetrations extracted from the graphs of Endo et al.

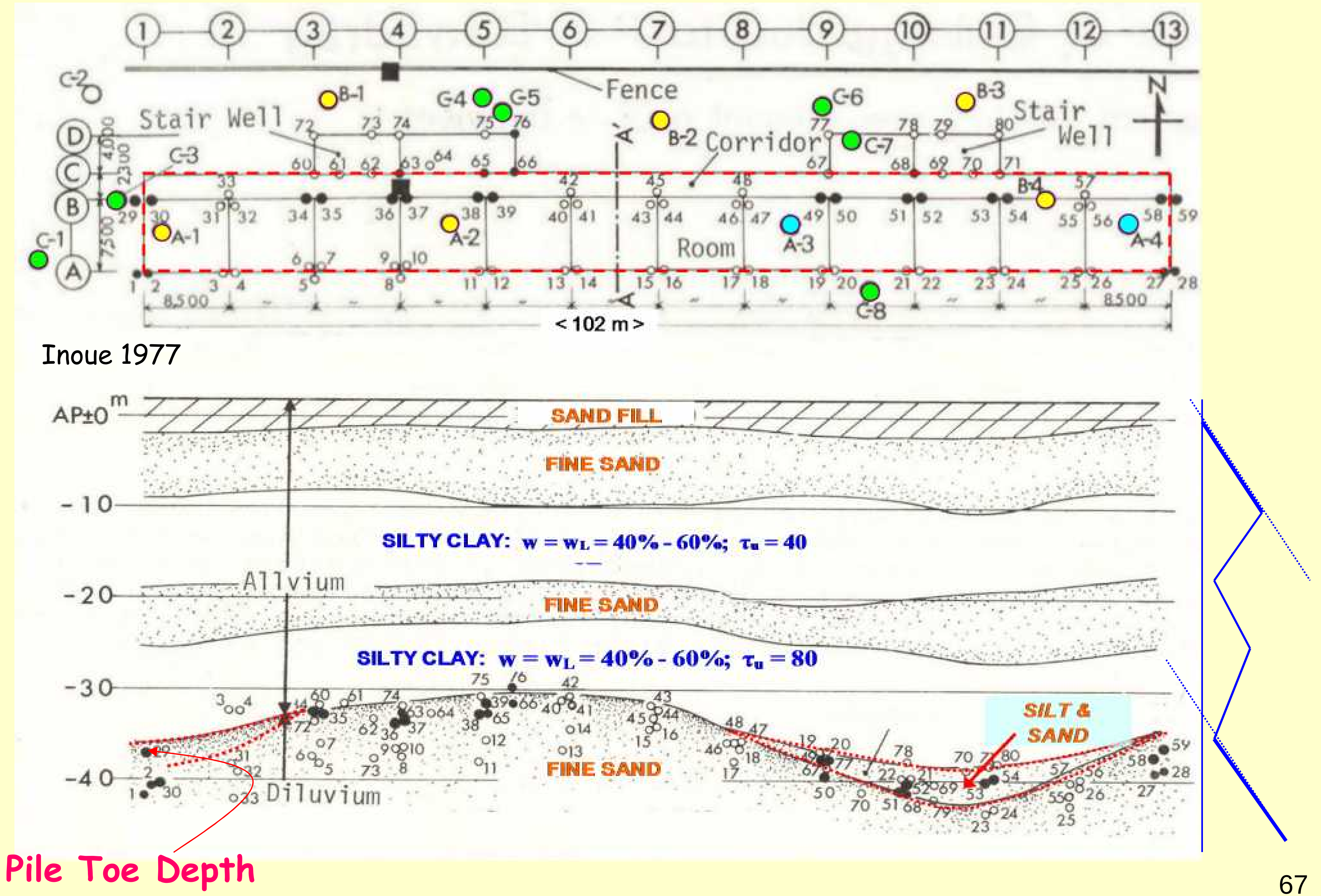




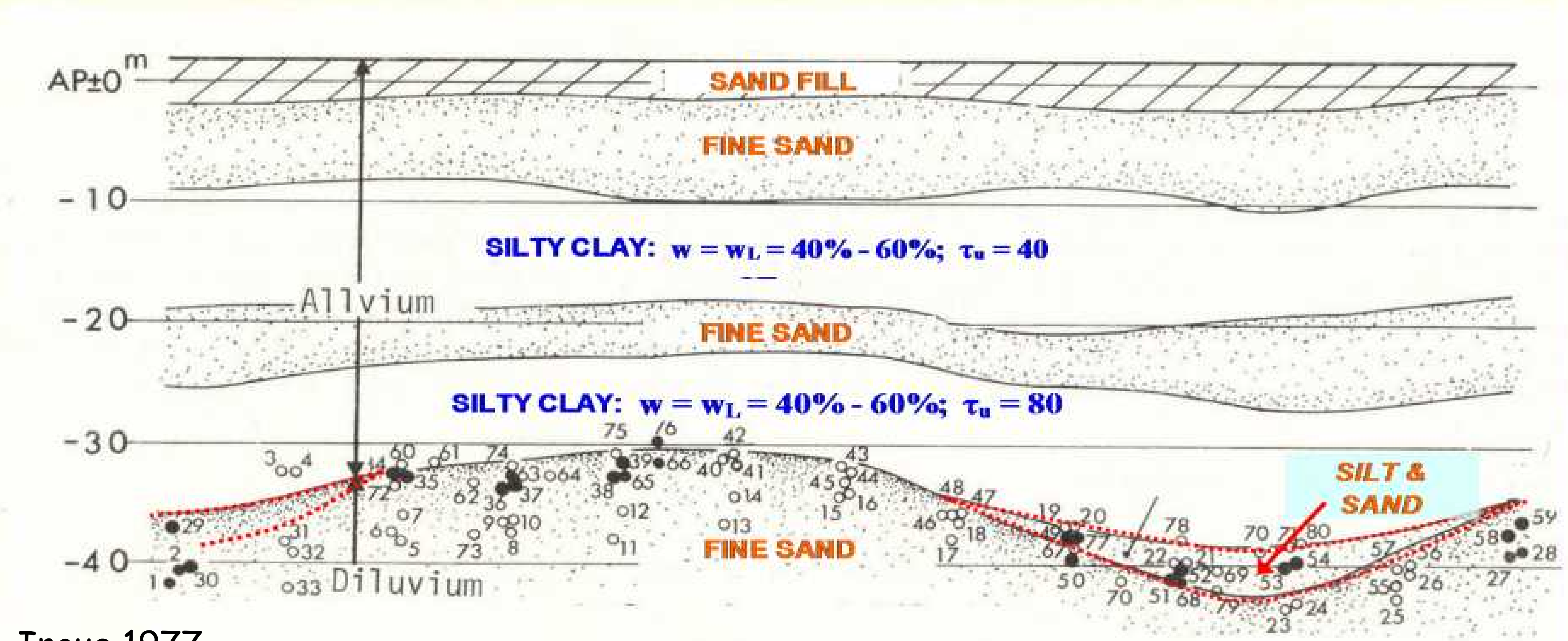
Inoue, Y., Tamaoki, K., and Ogai, T., 1977. Settlement of building due to pile downdrag. Proc. 9th ICSMFE, Tokyo, July 10-15, Vol. 1, pp. 561– 564.

A Downdrag Case

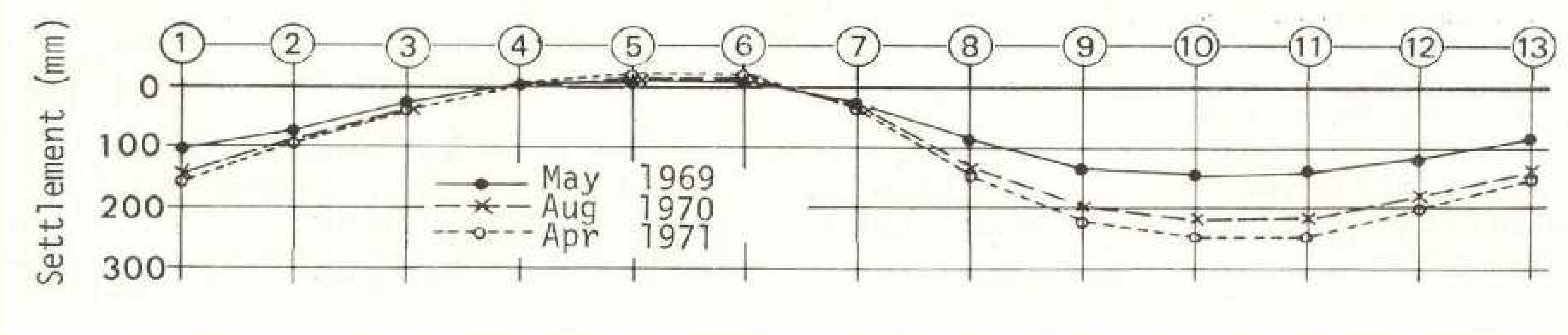
A three-storey building with a foot print of 15 m by 100 m founded on 500 mm diameter open-toe pipe piles driven through sand and silty clay to bearing in a sand layer at about 35 m depth. The piles had more than adequate capacity to carry the building. Two years after construction, the building was 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
Stair Well
Room
< 102 m >
Inoue 1977
SAND FILL
FINE SAND
SILTY CLAY: w = wL = 40% - 60%; τu = 40
Allvium
FINE SAND
SILTY CLAY: w = wL = 40% - 60%; τu = 80
FINE SAND
SILT & SAND
Diluvium
Pile Toe Depth



Inoue 1977



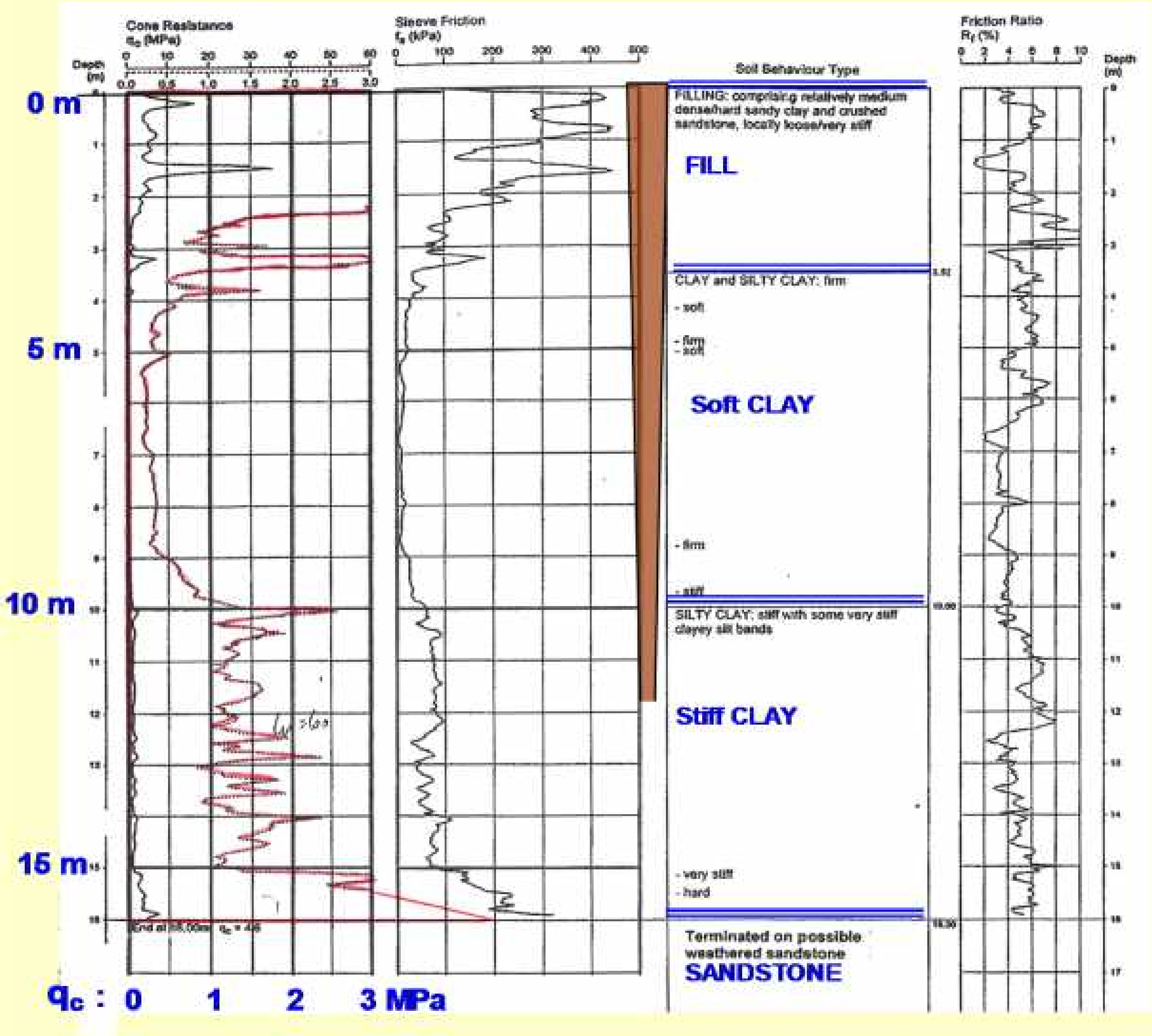
Settlement between piles in Row 6 and Row 10
from Sep. 1967 through May 1969 = 150 mm.

Slope ≅ 1 : 100
(Sep 67 Apr 71)

Looks bad, but, note, the building foundation is OK. No downdrag resulted because the neutral plane is located in the competent not settling soil. Of course, the pile s have drag forces, but that environmental effect is there be the settlement small or large. Presence of drag force is of no consequence for foundations—it is always present, regardless.

Singapore Apartments

Office Building placed on long toe-bearing piles in Brisbane, Australia. A later built, light extension was placed on short wood piles.



Data Courtesy (2007) of Wagstaff Piling Pty. Ltd., Queensland, Australia.

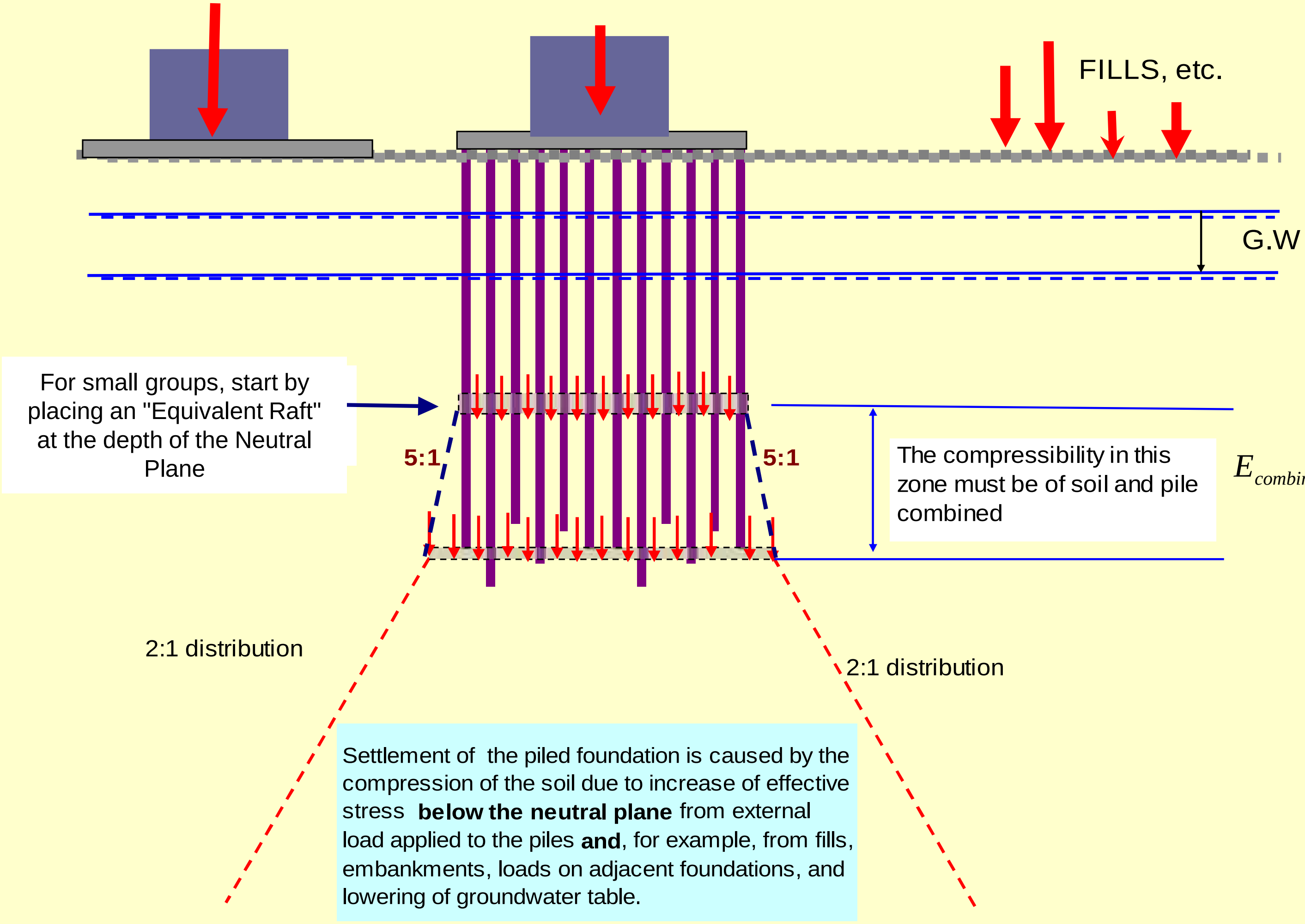
View toward the roof

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).

$ $ $ $

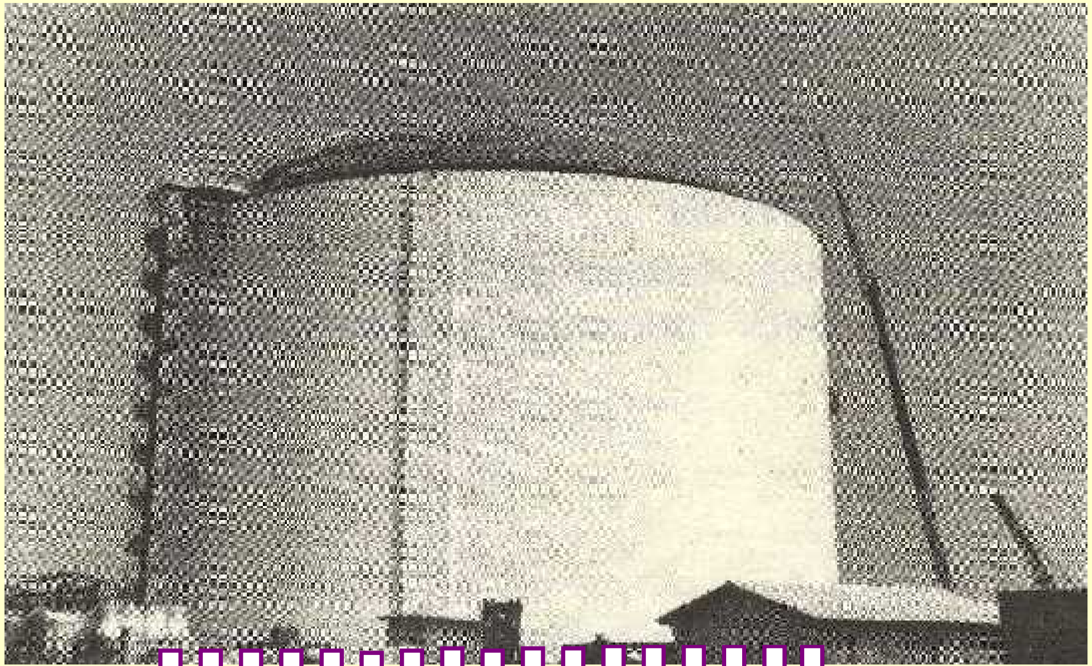
Settlement Analysis of Large Pile Groups by the Equivalent Raft Method

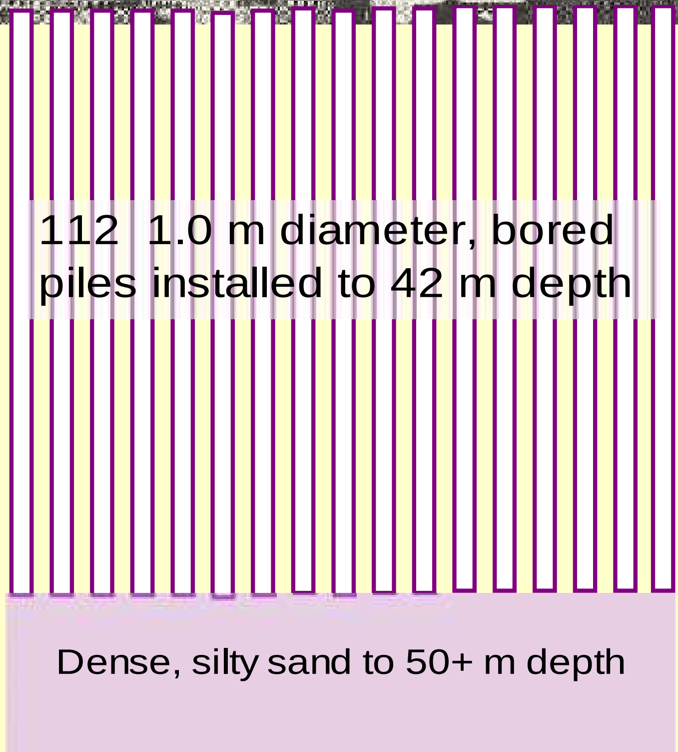


$$E_{combined} = \frac{A_{pile}\,E_{pile} + A_{soil}\,E_{soil}}{A_{pile} + A_{soil}}$$

N.B., the above approach goes far beyond the 1948 Terzaghi and Peck suggestion of placing The Equivalent Raft at the lower third depth

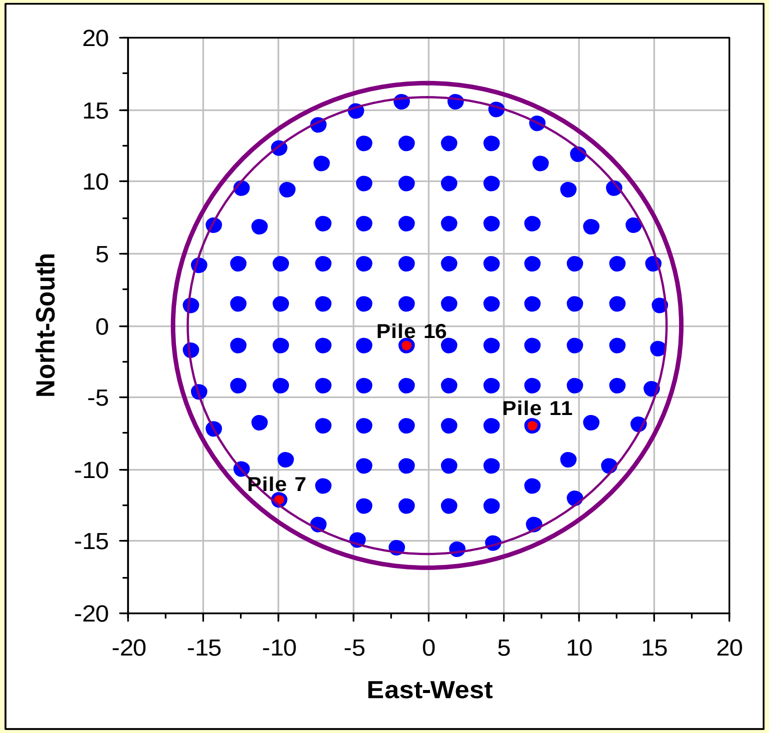
Liquid storage tank in Tessaloniki, Greece (Savvaidis 2009)



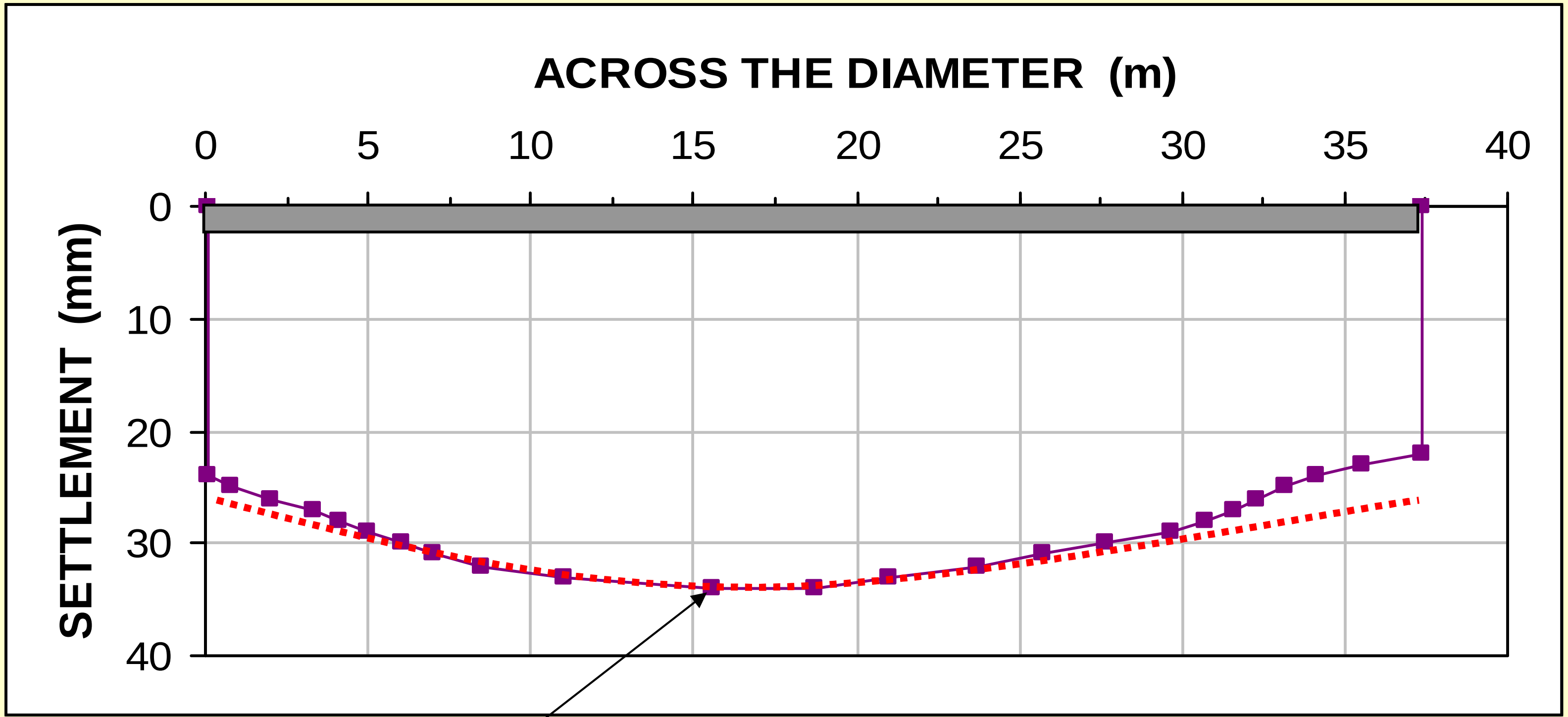


No Settlement

Settlement



Settlement measured across a tank diameter during a Hydro Test



Curve calculated for a flexible footing located at the pile toe level with parameters fitted to the settlement measured at the tank mid-point (calculations were performed with UniSettle).

Fellenius and Ochoa 2013; 2016

A quote from a textbook *)

The net effect negative skin friction is that the pile load capacity [sic!] is reduced and pile settlement increases. The allowable load capacity [sic!] is given as:

$$Q_{allow} = \frac{Q_{ult} - Q_{neg}}{F_S} - Q_{neg}$$

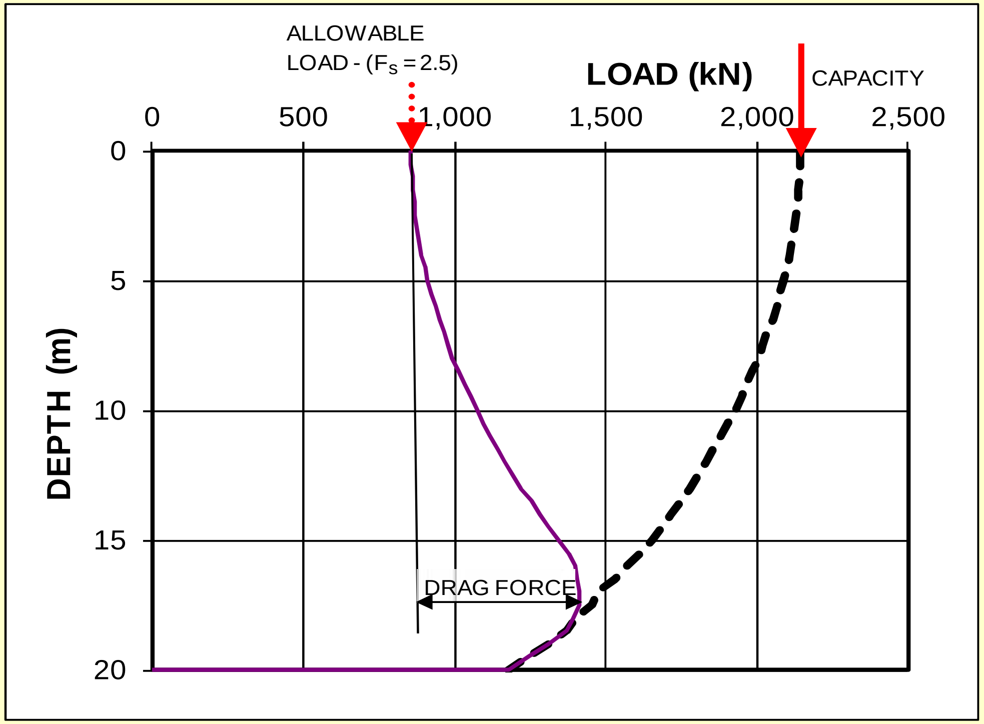
It could have been worse. Logically, the drag force (Q_{neg}) should have been increased by a factor of safety.

But so what! There is so much lack of logic in the approach, anyway.

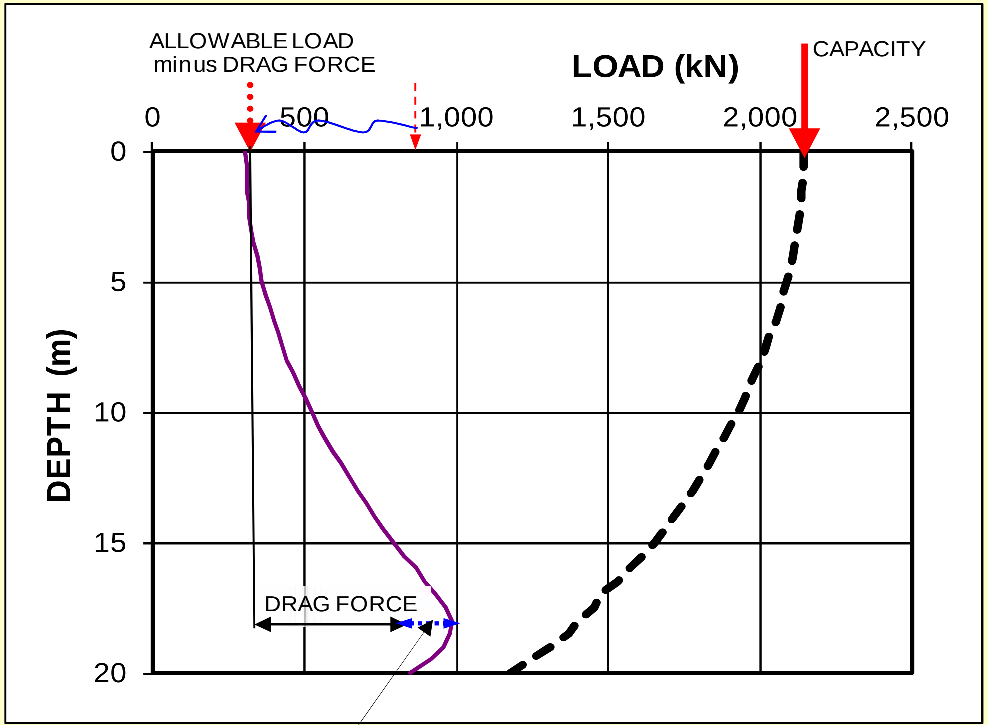
***) Compassion—perhaps misdirected—compels me not to identify the author**

Do not include the drag force when determining the allowable load!

Drag force must neither subtracted from the pile capacity nor from the allowable load



Effect of subtracting the drag force



HAS INCREASED!

If the pile capacity had been reduced with the amount of the drag force before subtracting the drag force from the "net capacity", so determined, there would have been no room left for working load!

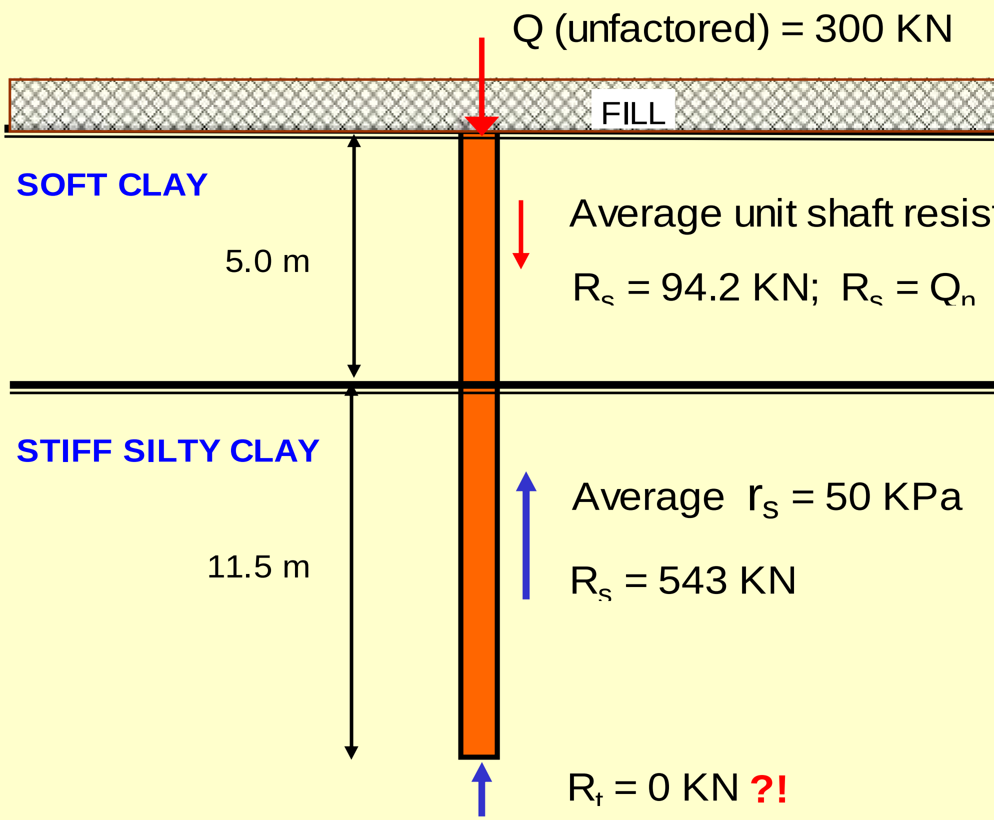
The Euro Code

The European Community has recently completed EuroCode 7, which is supposed to be adopted by all member states. The EuroCode treats the drag force as a load (an "action") similar to the load from the structure, and requires it to be added to that load and also that it is subtracted from the pile capacity)! Moreover, the shaft resistance in the soil layer that contributes to the drag force is disregarded when determining the pile resistance. That is, when a capacity has been determined to, say, 1,000 kN and the drag force is expected to be, say, 200 kN (an unrealistically low value), the "usable capacity", i.e., the usable unfactored resistance, is a mere 800 kN. This value is then factored. If resistance factor is 0.5, the factored resistance is 400 kN. When, as required, the factored drag force is subtracted (applying a drag-force load-factor of, say, 1.5, the amount left to support the factored load from the structure is 100 kN!

What "salvages" the economy of some designs must be that the EuroCode clauses advocate that the designer maintains the faithful approach that "***the drag force cannot really be that large, can it, please?***" in determining the magnitude of the drag force. Incredibly, the EuroCode says little on how to calculate settlement of piled foundations and nothing is stated about downdrag!

Unfortunately and regrettably, the recently issued US AASHTO LRFD Specs have adopted the EuroCode approach! A few US State DOTs, e.g., Utah, have wisely rejected the AASHTO Specs and apply the Unified Method.

Eurocode Guide , Example 7.4 (Bored 0.3 m diameter pile)



<u>CALCULATION</u>

$$f_q*300 + f_n*94 \leq 543/f_r$$

$$1.35*300 + 1.35*94 \leq 543/1.0$$

$$532 \leq 543$$

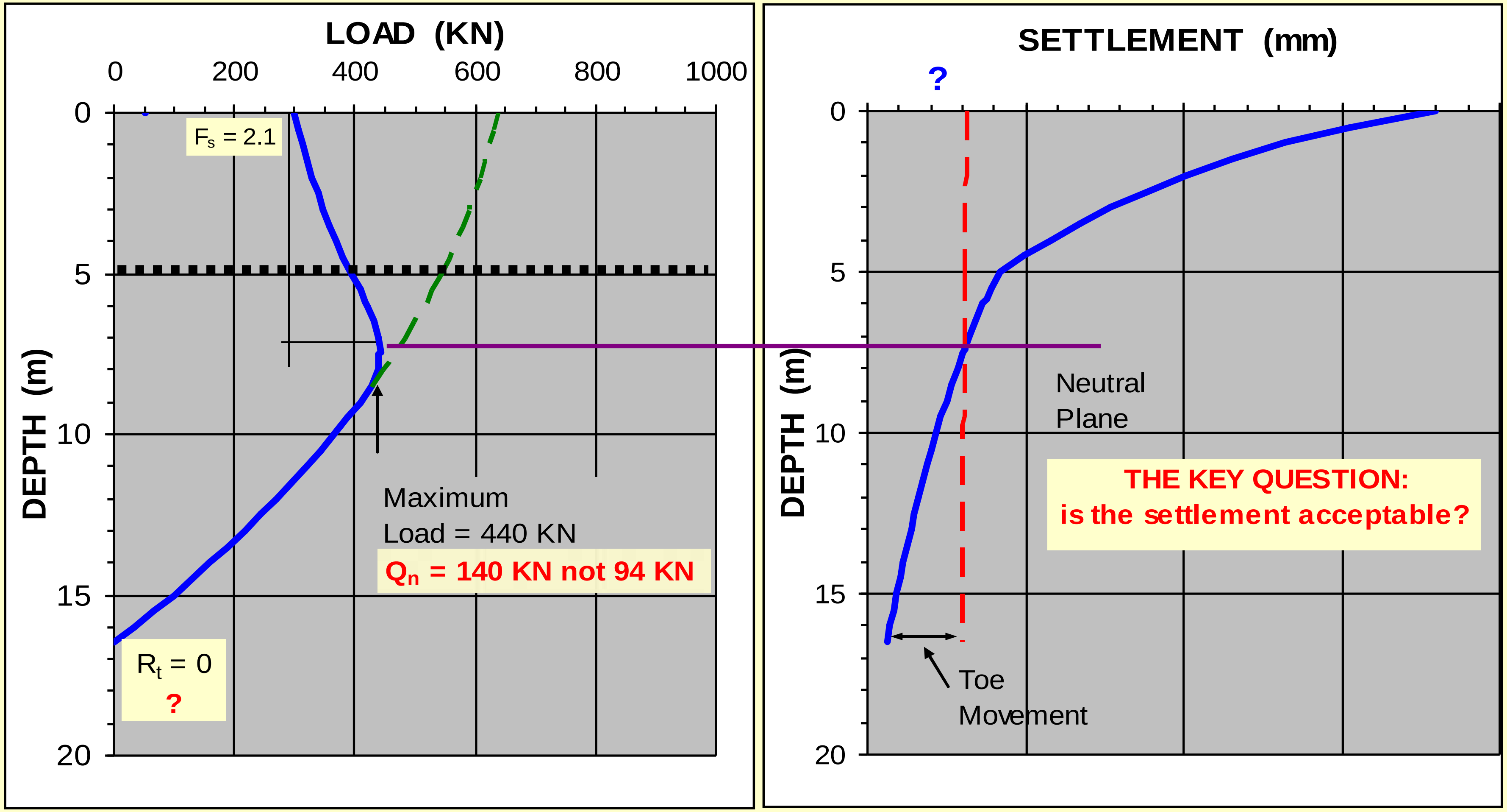
(Alternative: If f_r = 1.1, the length in the silty clay becomes 12.4 m)

"*The settlement due to the fill is sufficient to develop maximum negative skin friction in the soft clay*".

The Guide states that the two r_s-values are from effective stress calculation. The values correlate to soil unit weights of 18 KN/m³ and 19.6 KN/m³, ß-coefficients of 0.4 **<u>in both layers</u>** with groundwater table at ground surface, and a stress of 30 KPa from the fill. Note that the example presupposes that the analysis is carried out for the long-term conditions.

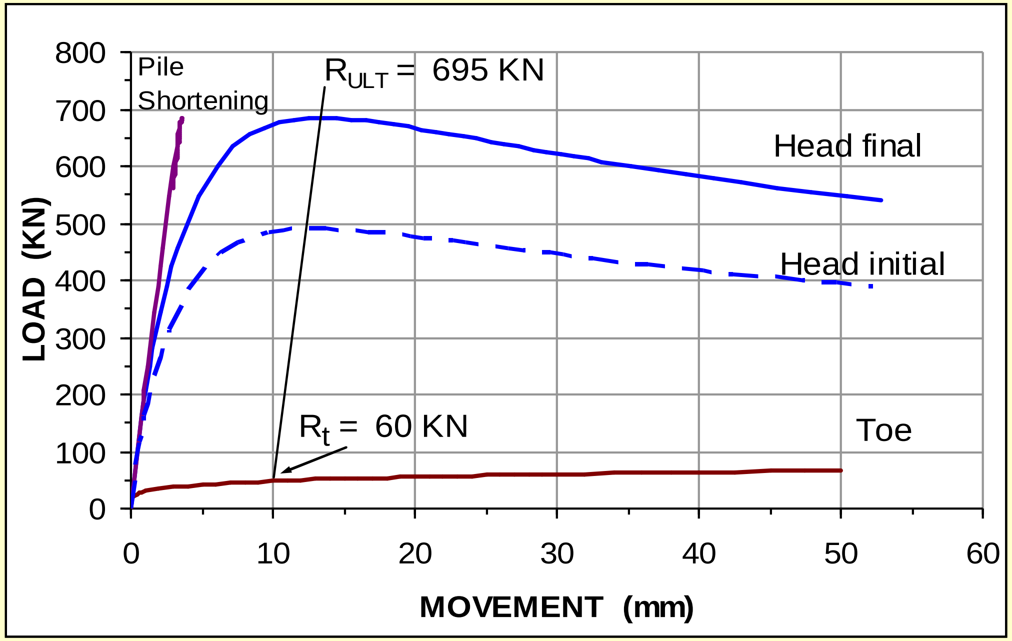
The Guide states that the neutral plane lies at the interface of the two clay layers. Based on the information given in the example, this <u>cannot be correct</u>. But there is a good deal more wrong with this "design" example.

Results of analysis using the given numerical values (ß = 0.4 for both layers and zero toe resistance)



If the settlement is acceptable, there may be room for shortening the pile or increasing the load. That would raise the location of the neutral plane. Would then the pile settlement still be acceptable? And, would not some toe resistance develop?

Let's assume that a static loading test has been carried out on the example pile and taken well beyond the the usual maximum movement. (N.B., to assume no toe resistance is too conservative and a small value has therefore been added to the example). Toe resistance is a function of toe movement. Let's also assume a toe response per a q-z function representative for a stiff clay, and a strain softening shaft resistance. Then, the loading test results in the following pile-head load-movement curves. The curve marked "**initial**" is from a test assumed carried out during the design phase. The curve marked "**final**" is assumed to have been carried after the consolidation (or most of it) caused by the fill has developed — we could have marked it "forensic", i.e., a test undertaken by the lawyers after the piled foundation showed to settle excessively.

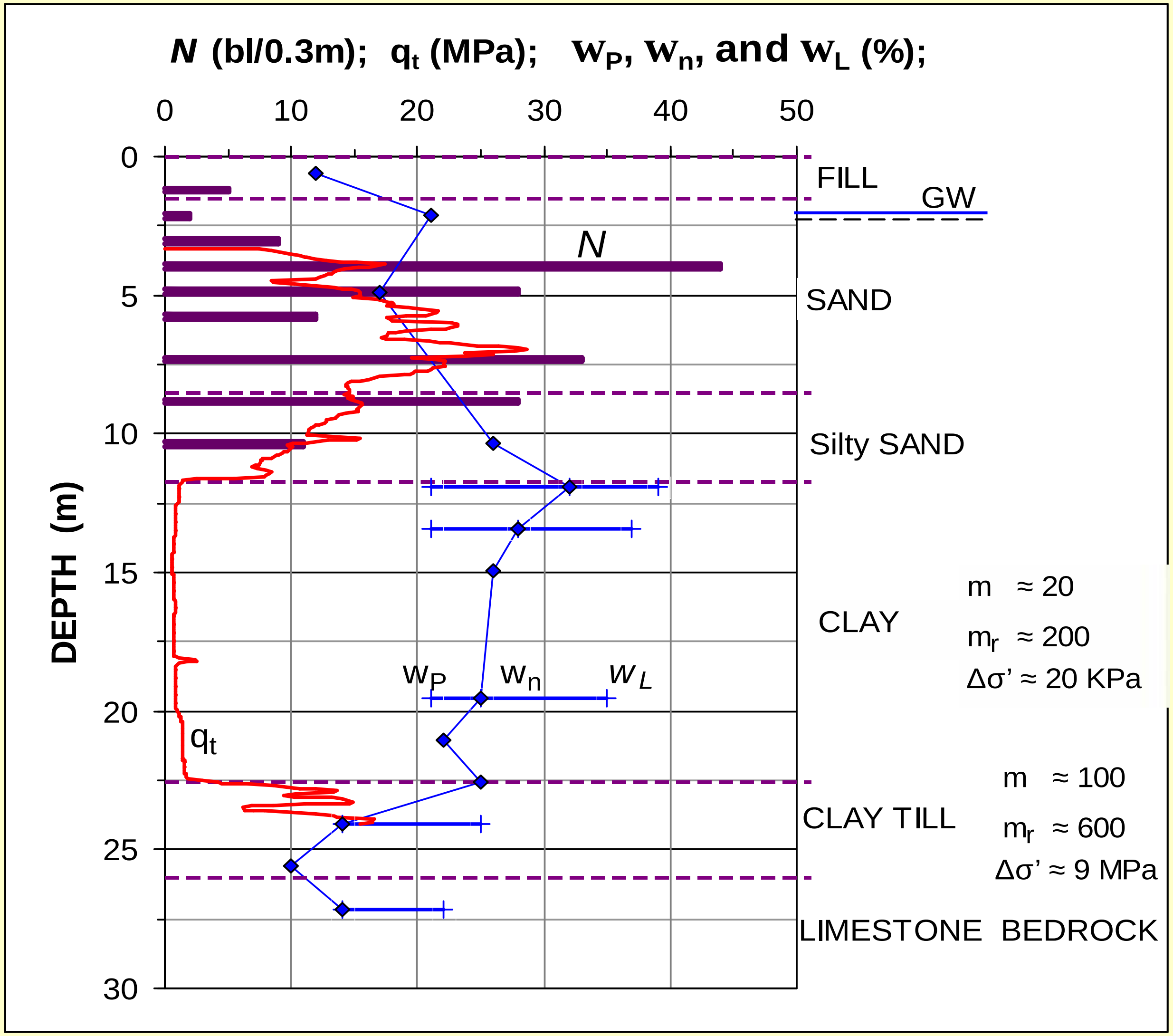


Designers **do not usually design for the final conditions — it is commendable that they did for this example, but the design must also consider the initial conditions. The structure is probably built at the same time as the fill is placed and before the consolidation process had strengthened the soil. I do not think everyone would be comfortable placing a sustained load of 300 KN on piles that plunge at 500 KN (test at initial condition).**

Example of the Unified Design Approach as Applied to a Refinery Structure

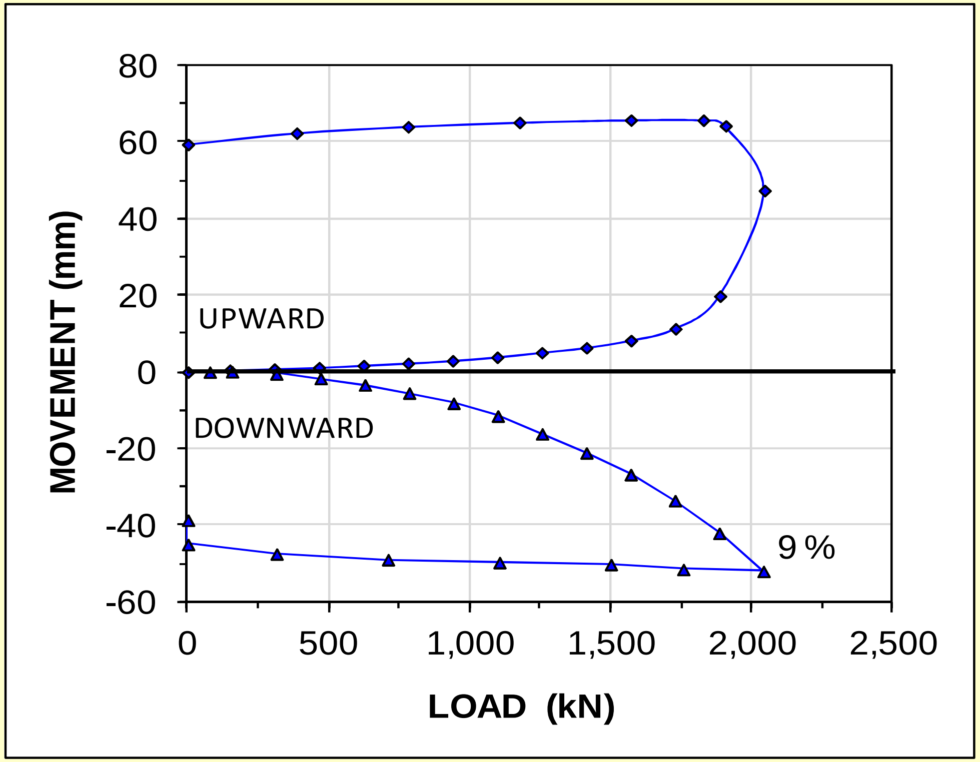
Soil Profile

Design for a large refinery expansion was undertaken at a site reclaimed from a lake in the 1960s. The natural soils consist of sand deposited on normally consolidated, compressible post glacial lacustrine clay followed by silty clay till on limestone bedrock found at about 25 m to 30 m depth below existing grade. The site will be raised an additional 1.5 m, which will cause long-term settlement. Piled foundations are needed for all structures.

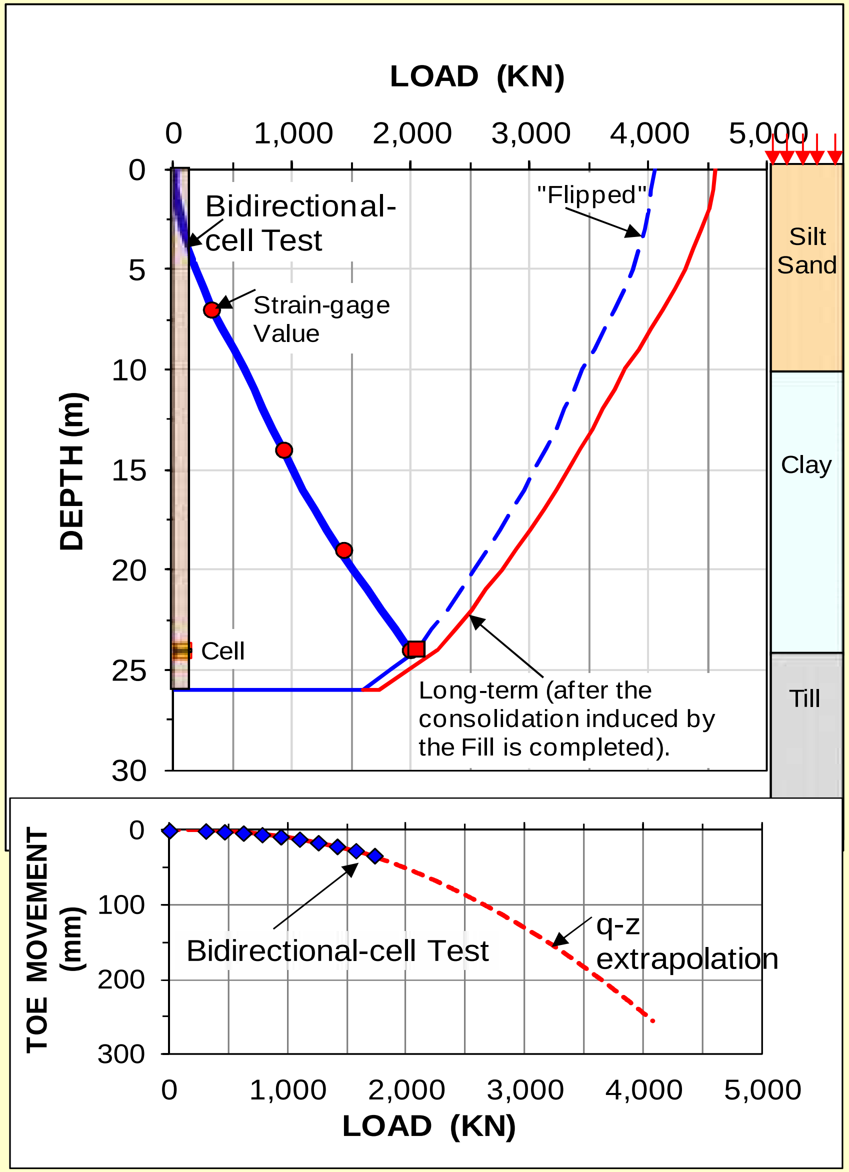


Fellenius and Ochoa (2009)

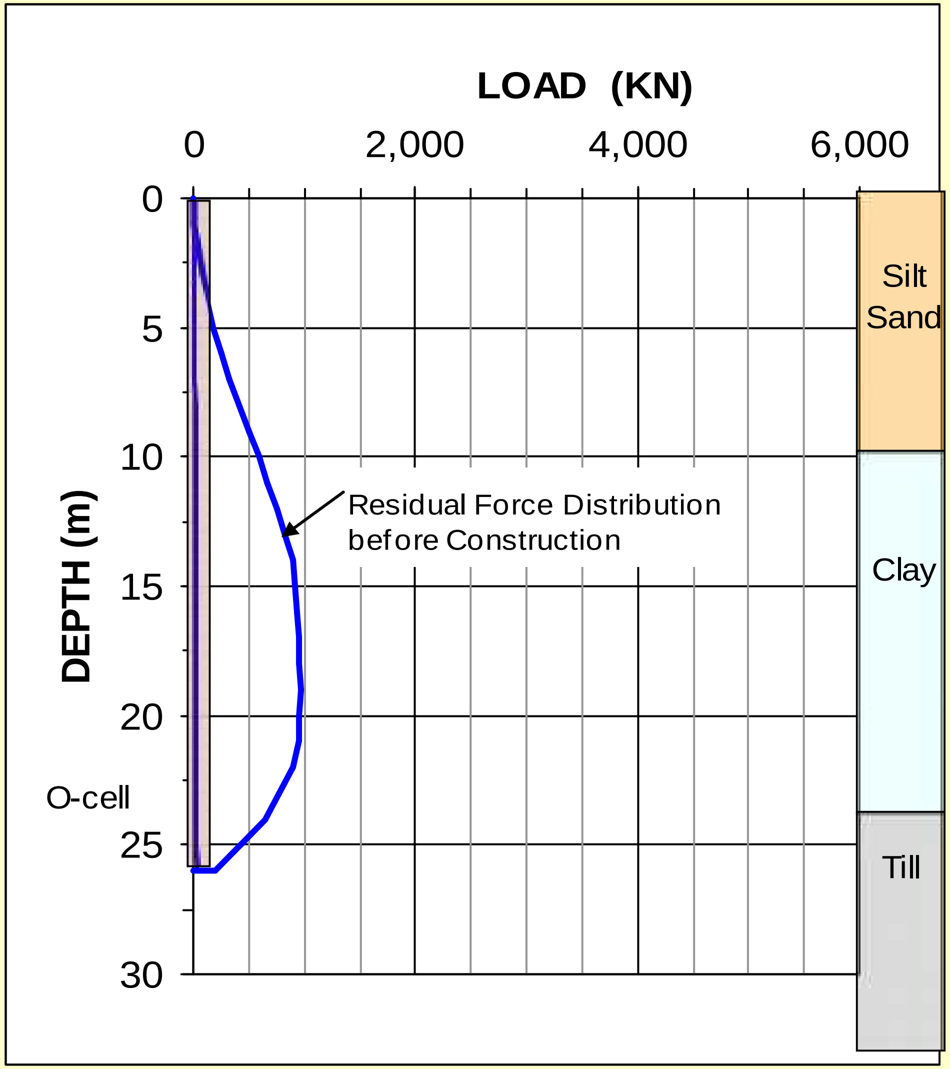
Results of a bidirectional-cell test on a 575-mm diameter test pile, a 26 m deep bored cylindrical pile. A 1.5 m thick fill will be placed over the site after construction. Piles are single or in small groups.



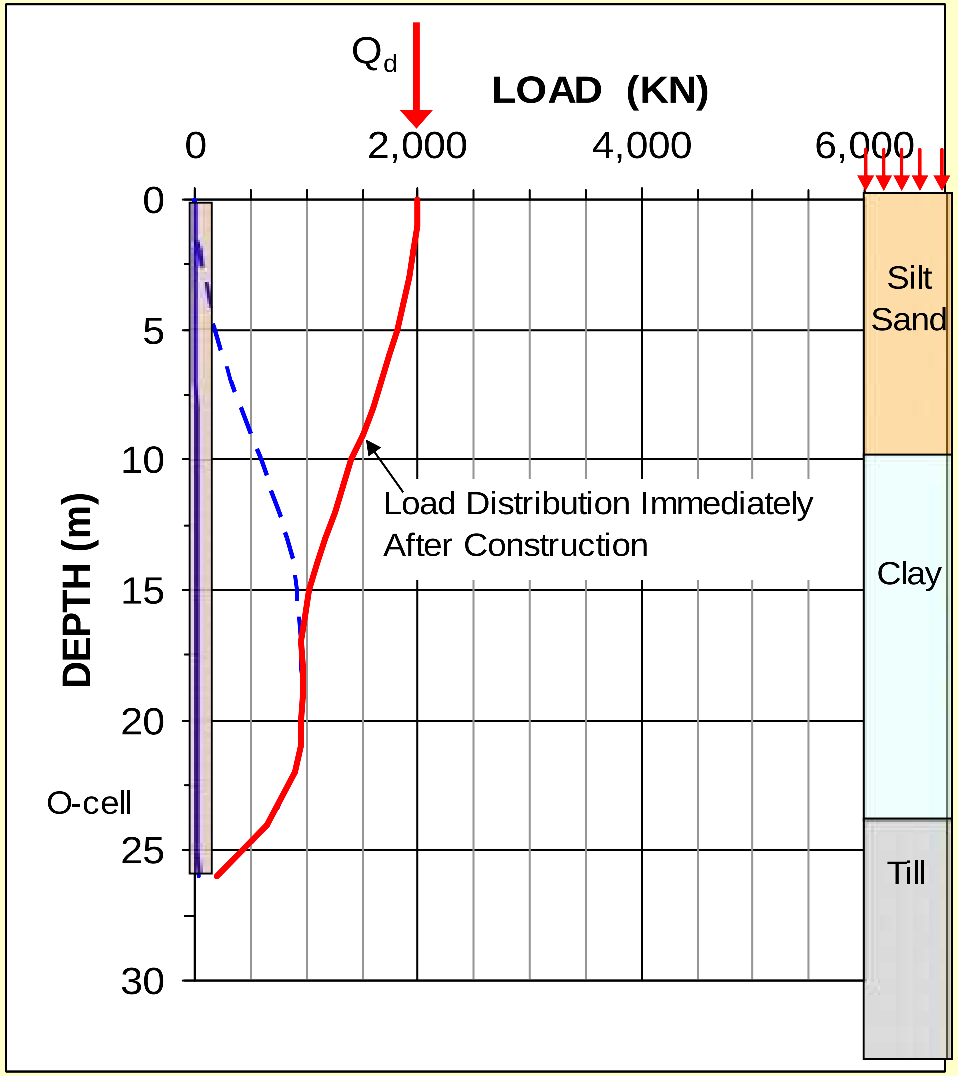
Results of analysis of test data: Load Distributions



Distribution of residual force in the pile after installation, but before load is applied to the pile.

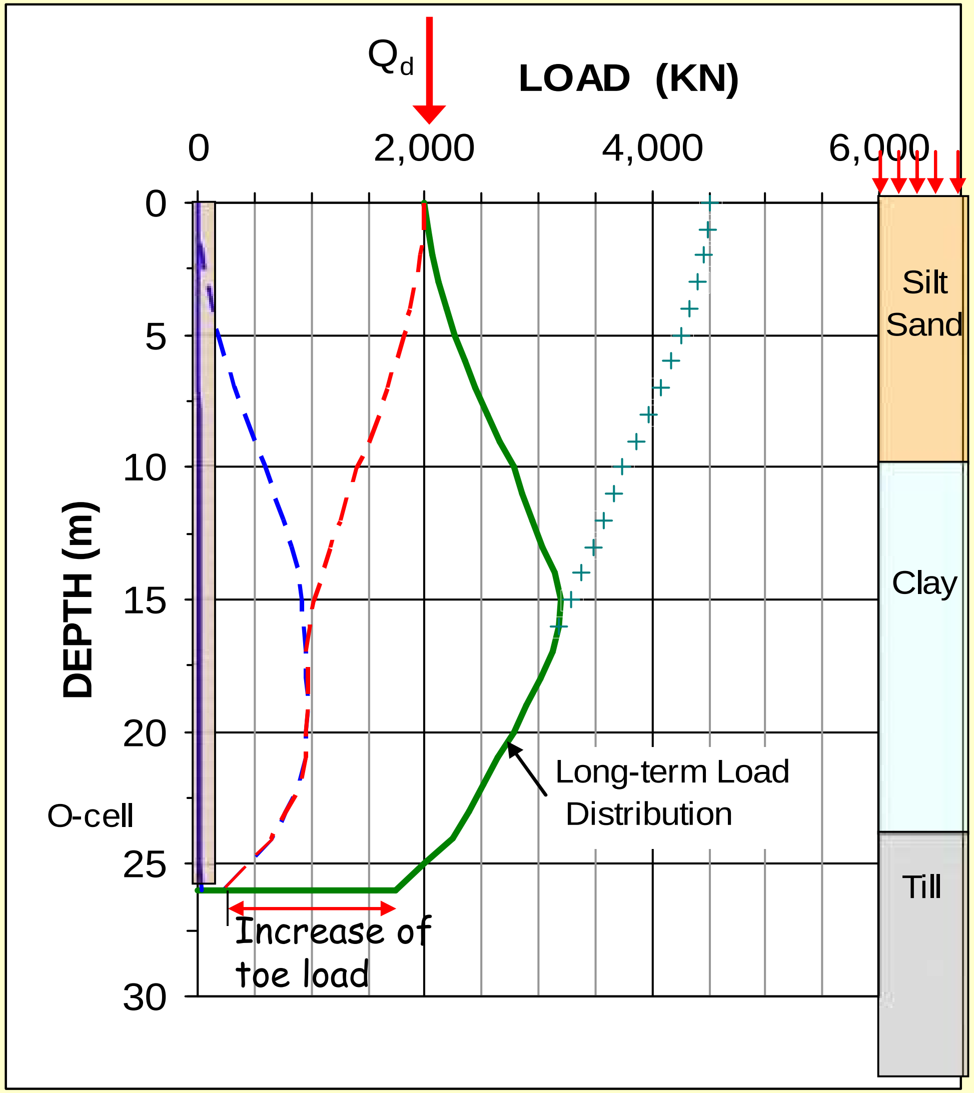


Distribution of load in the pile immediately after the pile starts to sustain the load from the structure.



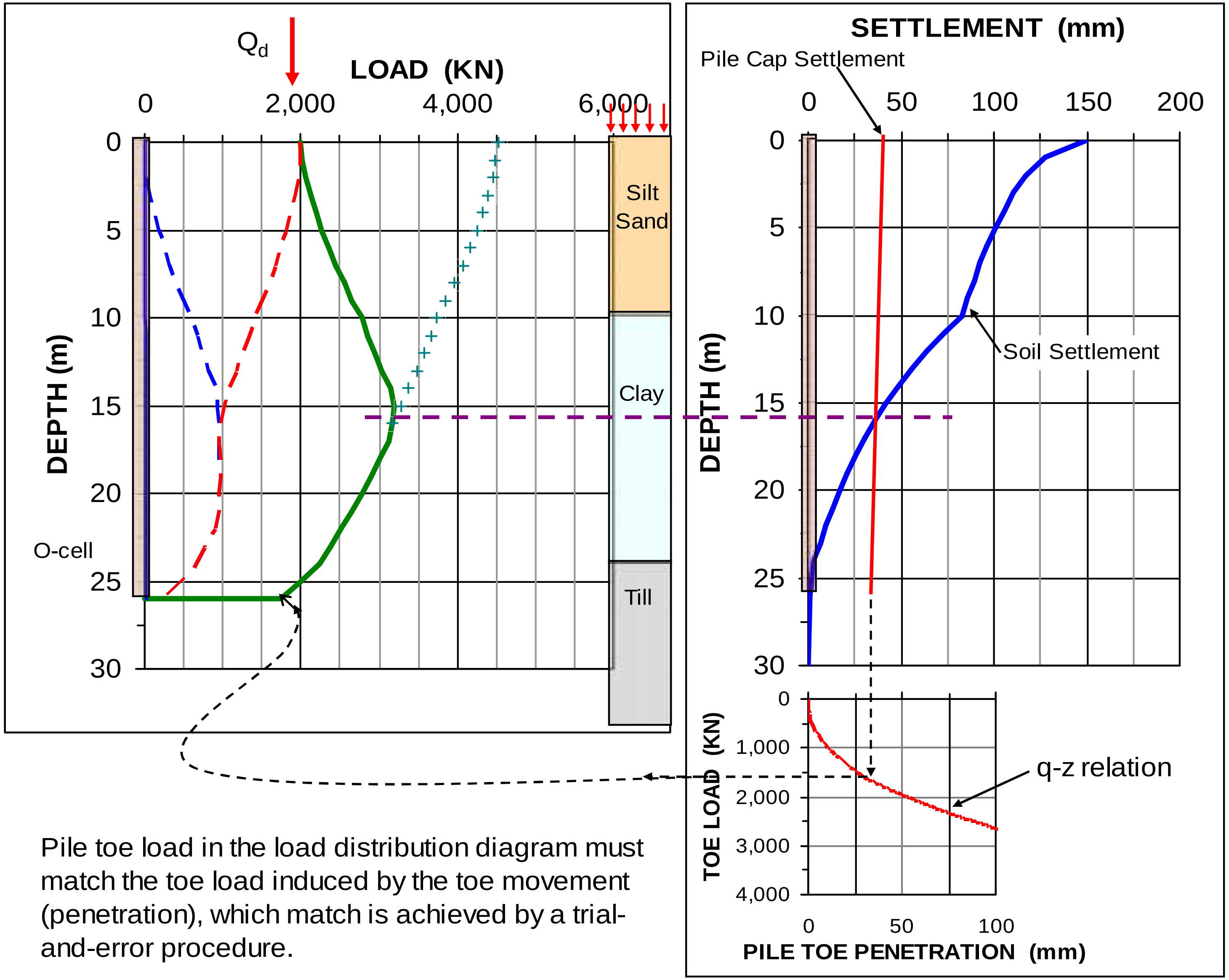
Other than at the pile toe, the amount of residual force and its distribution are not that interesting

Long-term load distribution



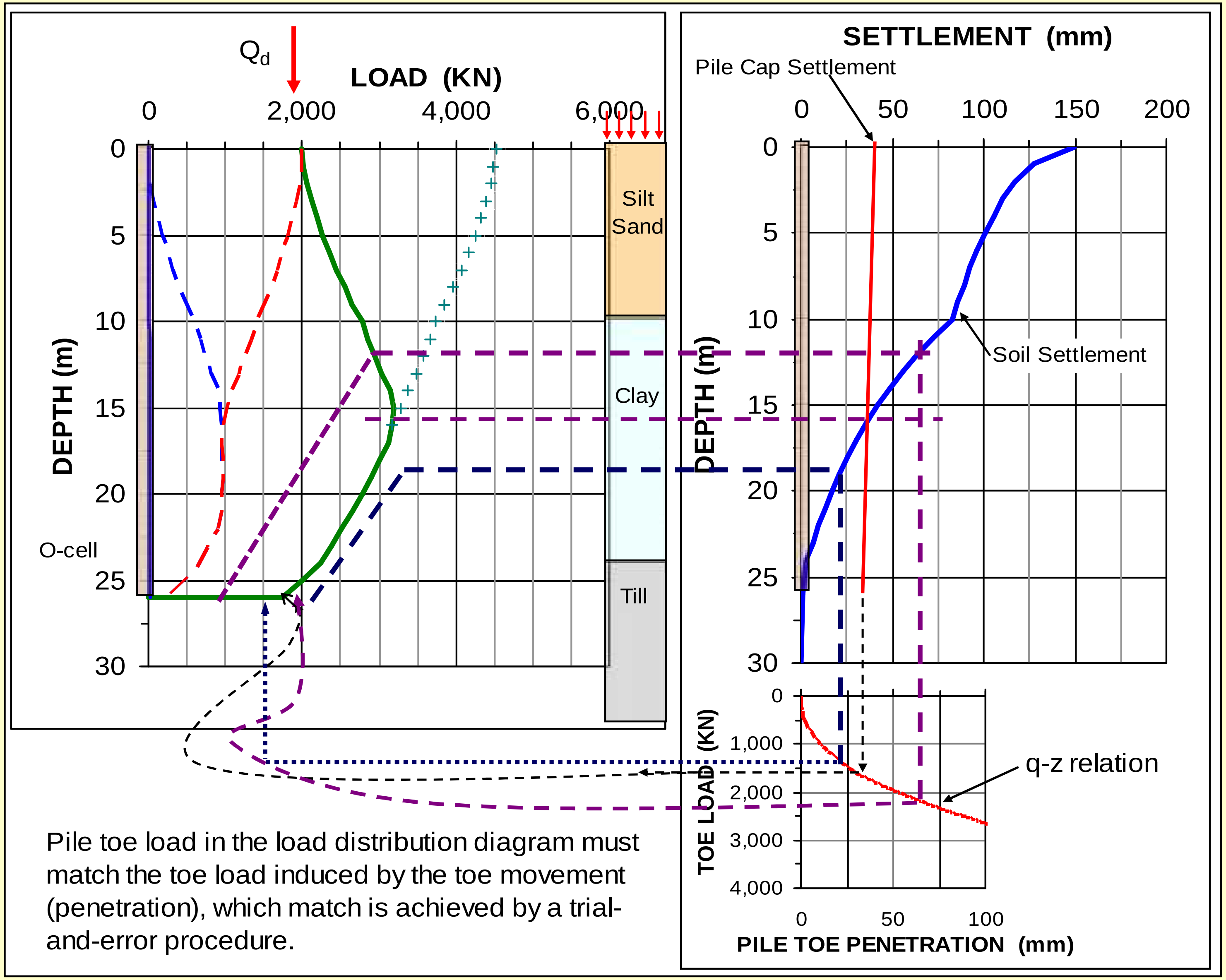
The shaft shear is assumed to be fully mobilized. However, the toe resistance value to use is a function of the toe penetration due to downdrag and can only be determined from assessing the soil settlement distribution.

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

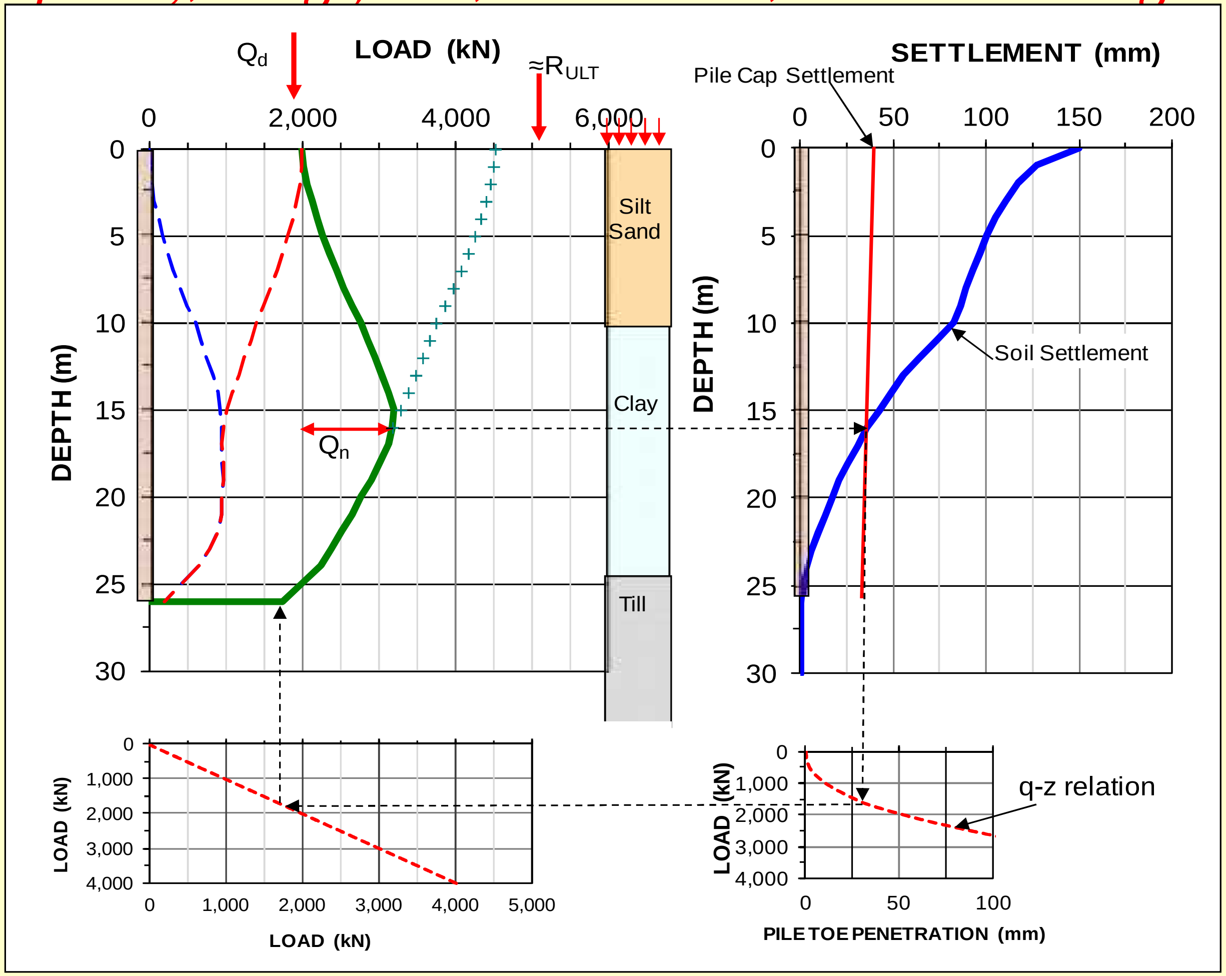
Force and settlement (downdrag) interactive design.
The unified pile design for capacity, drag force, settlement, and downdrag



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.

The Unified Piled Foundation Design

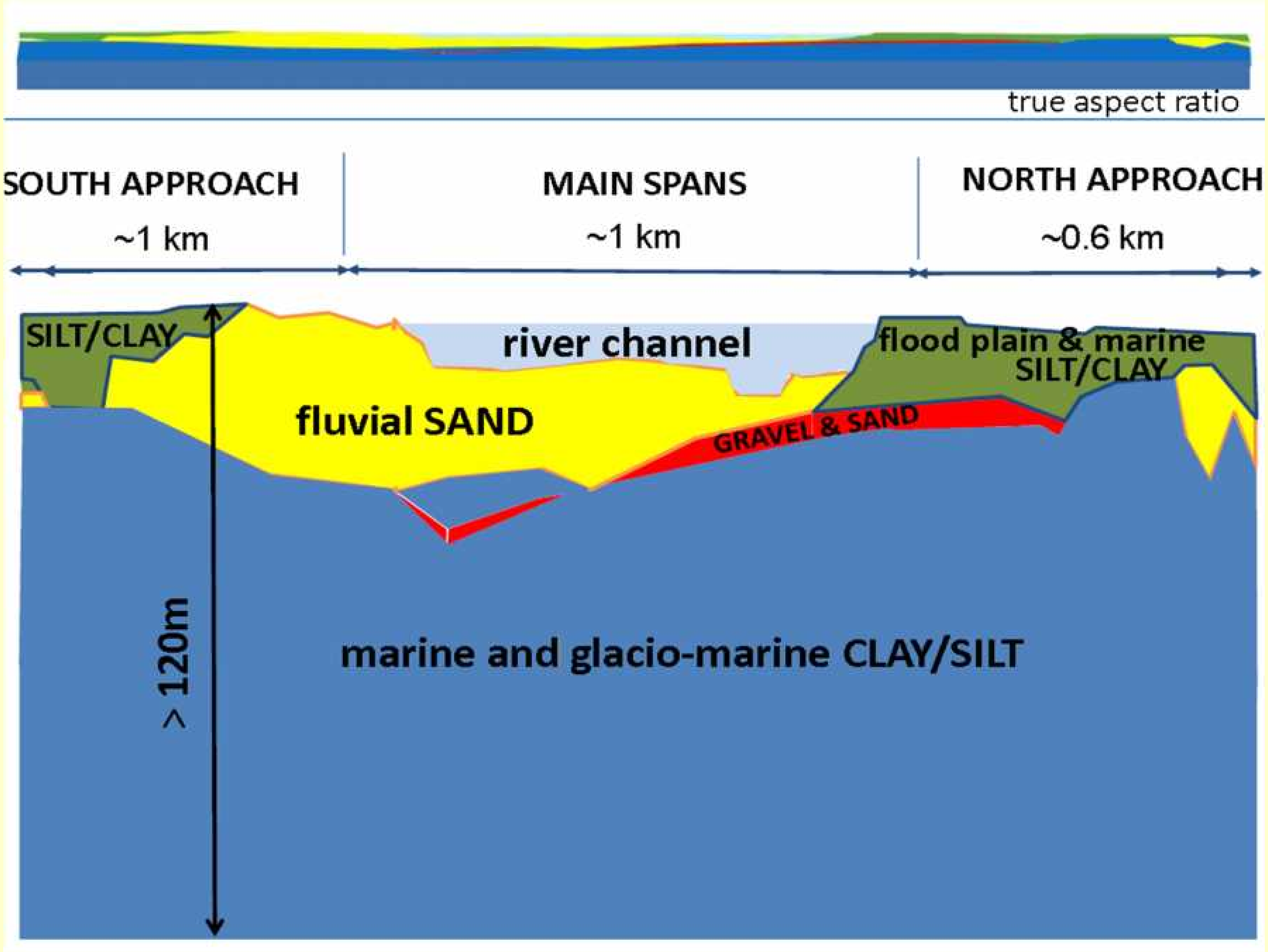
Force and settlement interactive design for capacity, drag force, settlement, and downdrag



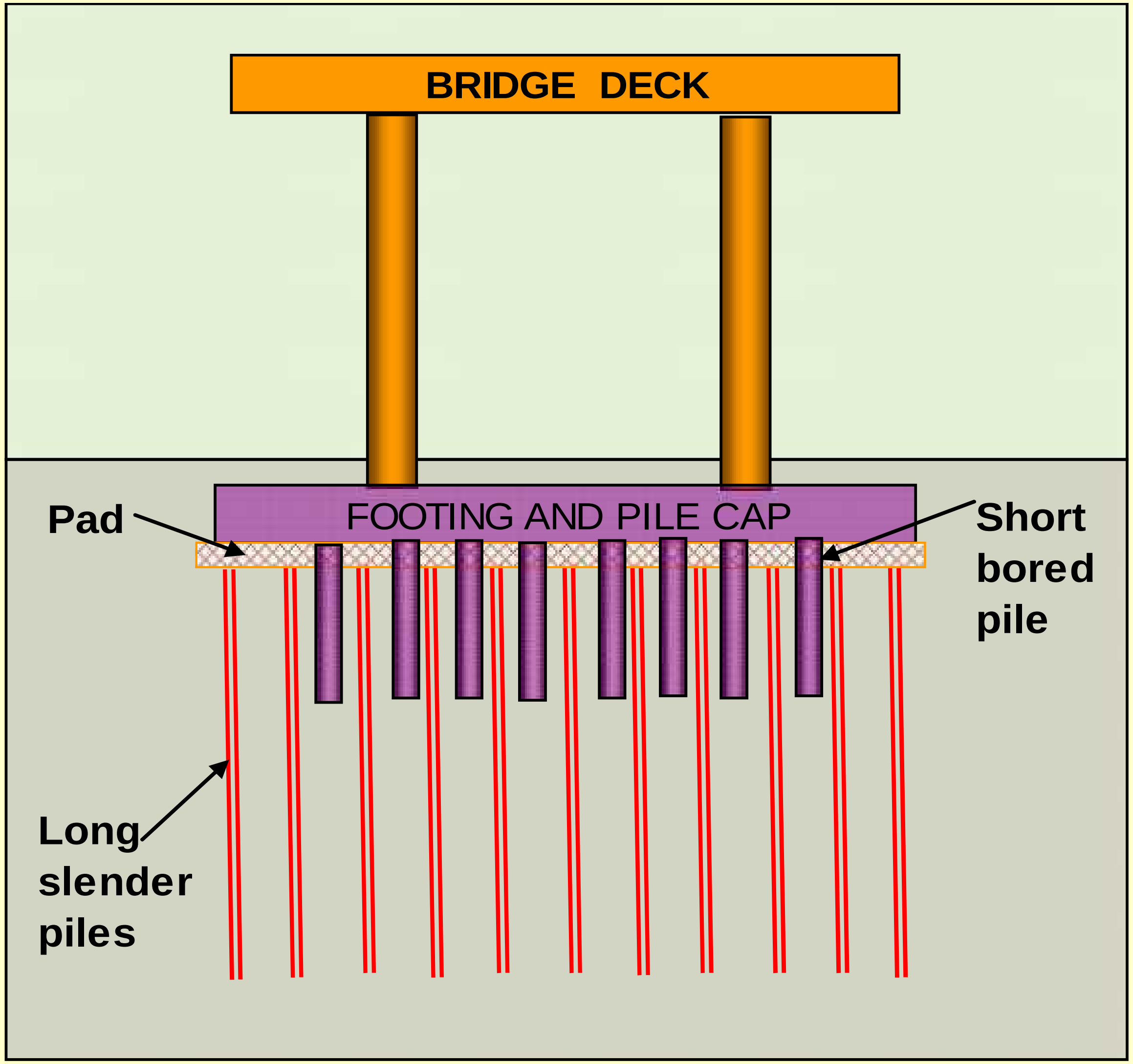
The design is based on three "*knowns*": The shaft resistance distribution, the toe load-movement response, and the overall settlement distribution.

A recent modern application of a piled pad foundation is the foundations for the Rion-Antirion bridge piers (Pecker 2004). Another is the foundations of the piers supporting the Golden Ears Bridge in Vancouver, BC (Sampaco et al., Naesgaard et al. 2012), pictured below.

Golden Ears Bridge in Vancouver, BC.



Piled pad foundation piers supporting the Golden Ears Bridge in Vancouver, BC.



Bored piles:
(900 mm; 8 m) to provide lateral resistance

and

Driven piles:
(300 mm; 30 m) piled-pad piles over an about 100 m thick deposit of soft compressible clay

16

ShinHo and MyeongJi Housing Project,
in the estuary of the Nakdong River, Pusan, Korea

Project Managers: Drs. Song Gyo Chung and Sung Ryul Kim, Dong-A University, Busan


Tangshan
North Korea
Dalian
Pyongyang
Incheon
Seoul
Qingdao
Daejeon
South Korea
Daegu
Kwangju
Busan
(Pusan)
Fukuoka
Cheju do
Hirado-shima
Jiangsu
Yangzhou
Kumamoto
Fukue-jima
Kyushu
Nanjing
Wuxi
Suzhou
Shanghai
Kagoshima


Miryang
Ulsan
Yangsan
Masan
Busan
(Pusan)
Chinhae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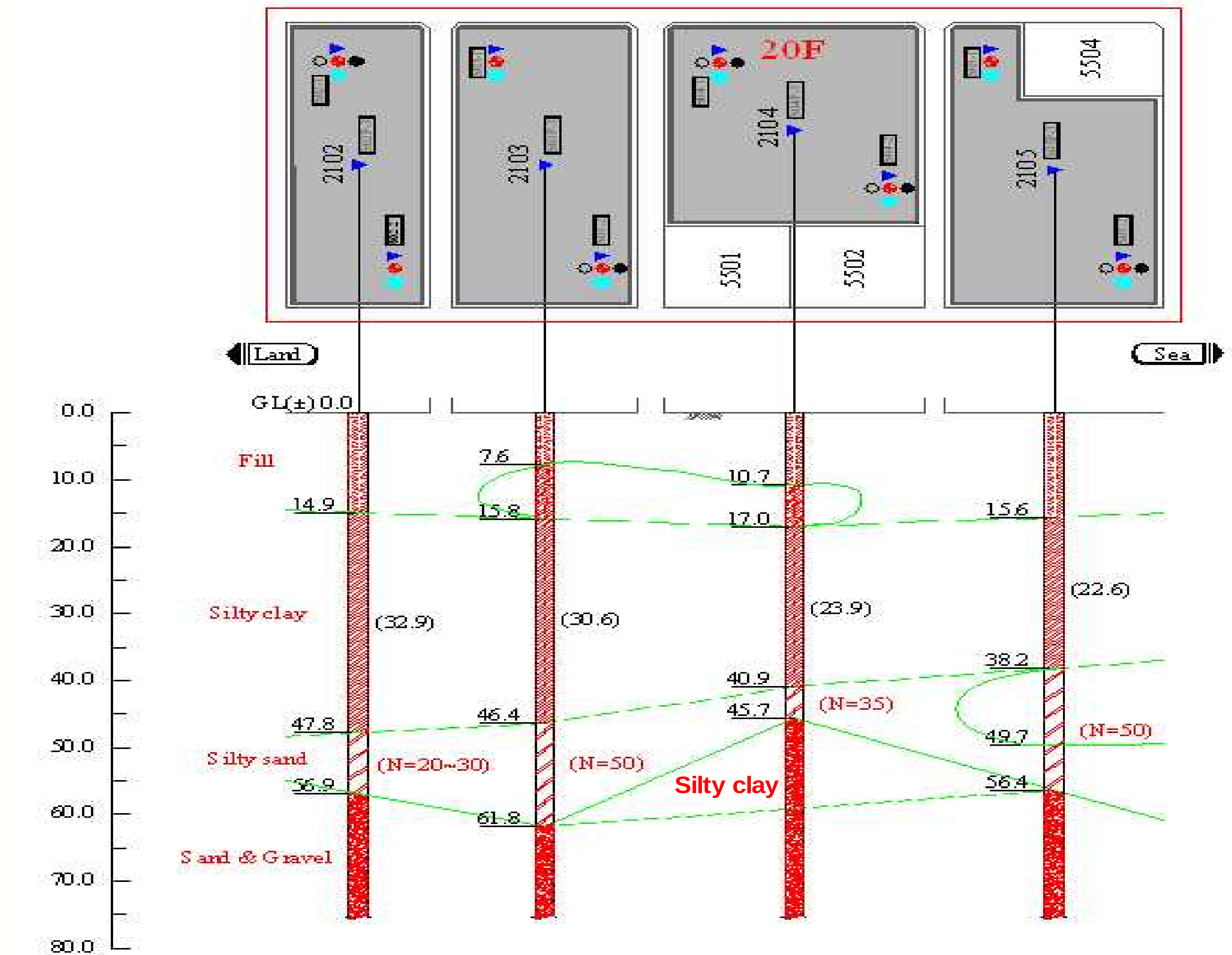
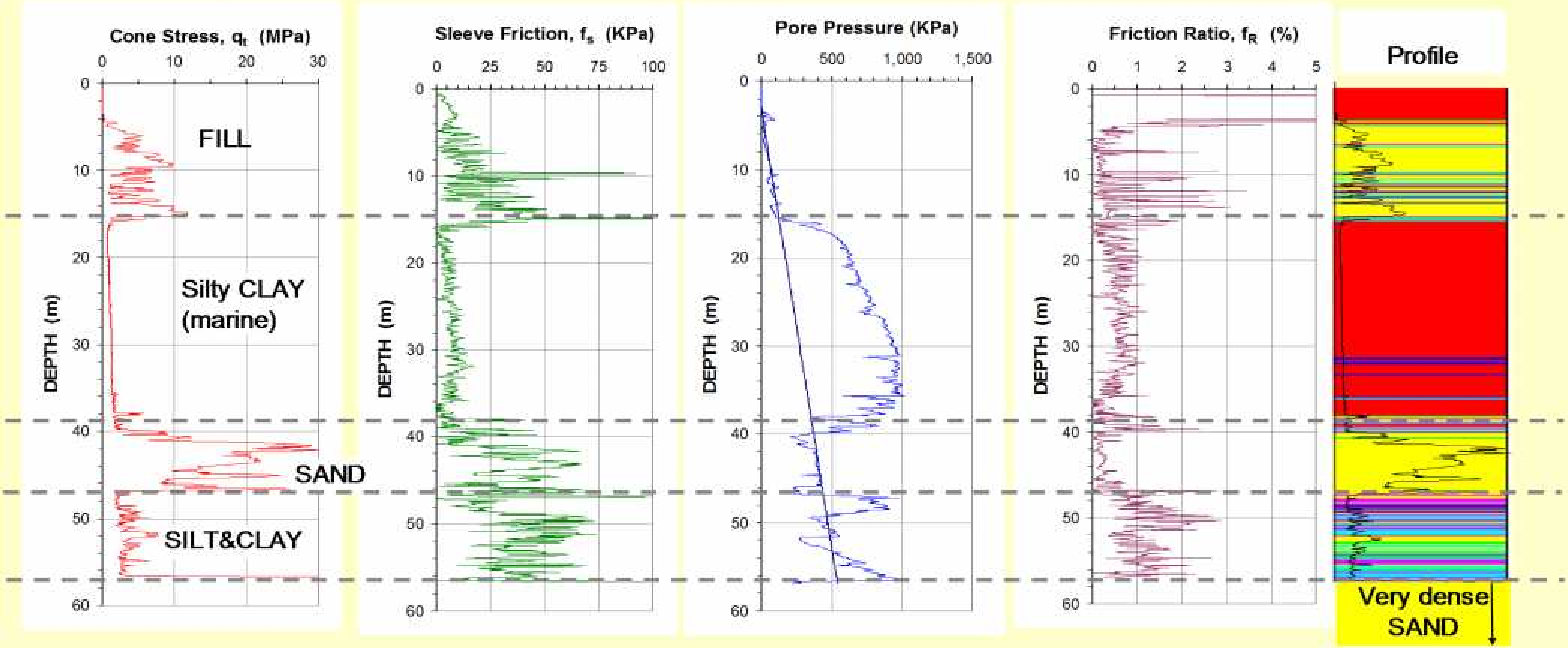
AIR VIEW
(Shinho Site)

SITE PLAN (SH Site)





CPTU sounding at the location of the Shin-Ho test pile

The pile alternative investigated was a 600 mm diameter cylinder pile with a 100 mm wall driven closed-toe

The questions to resolve in the design were

1. What is the capacity in the different layers?

2. What is the depth to the force equilibrium/settlement equilibrium, i.e., the neutral plane

3. What will be the maximum load in the pile? Is the structural strength adequate?

4. What is the settlement of the pile as a function of the location of the neutral plane?

The field tests were designed to answer the questions. The foundation design was per the Unified Design Method. Total savings were about us$300 million over the alternative of steel pipe piles (which included the drag force as an active load).


Thank you for your attention