BASICS OF DESIGN OF PILED FOUNDATIONS

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The Unified Method for Design of Piled Foundations

For Capacity, Drag Load, Settlement, and Downdrag

Brisbane November 28, 2008

A foundation design is carried out by a team of professionals

The Team



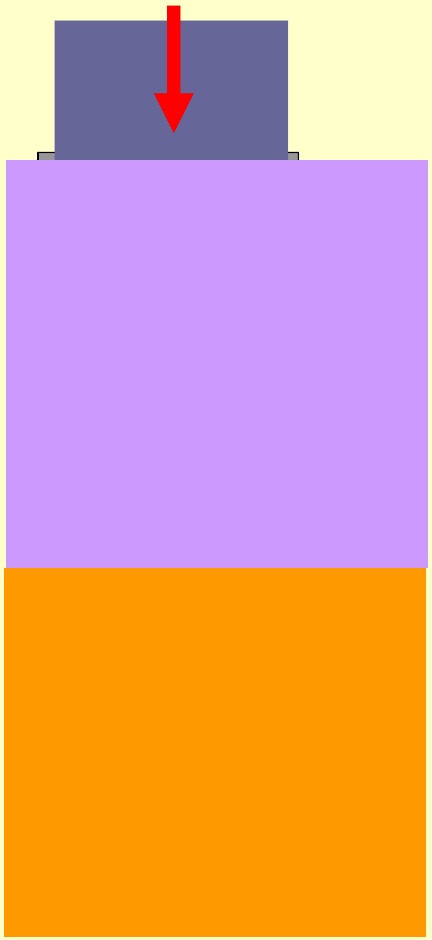
The Designer The Contractor The Inspector

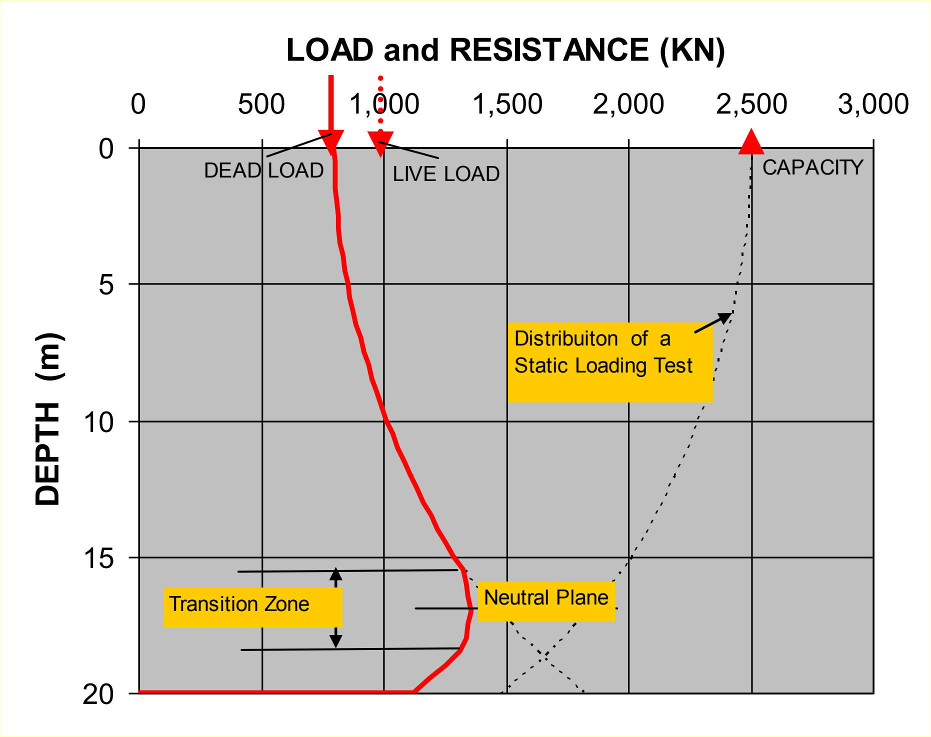
The Textbook Writer

He’s seen the action, he knows the style, but has he been there?

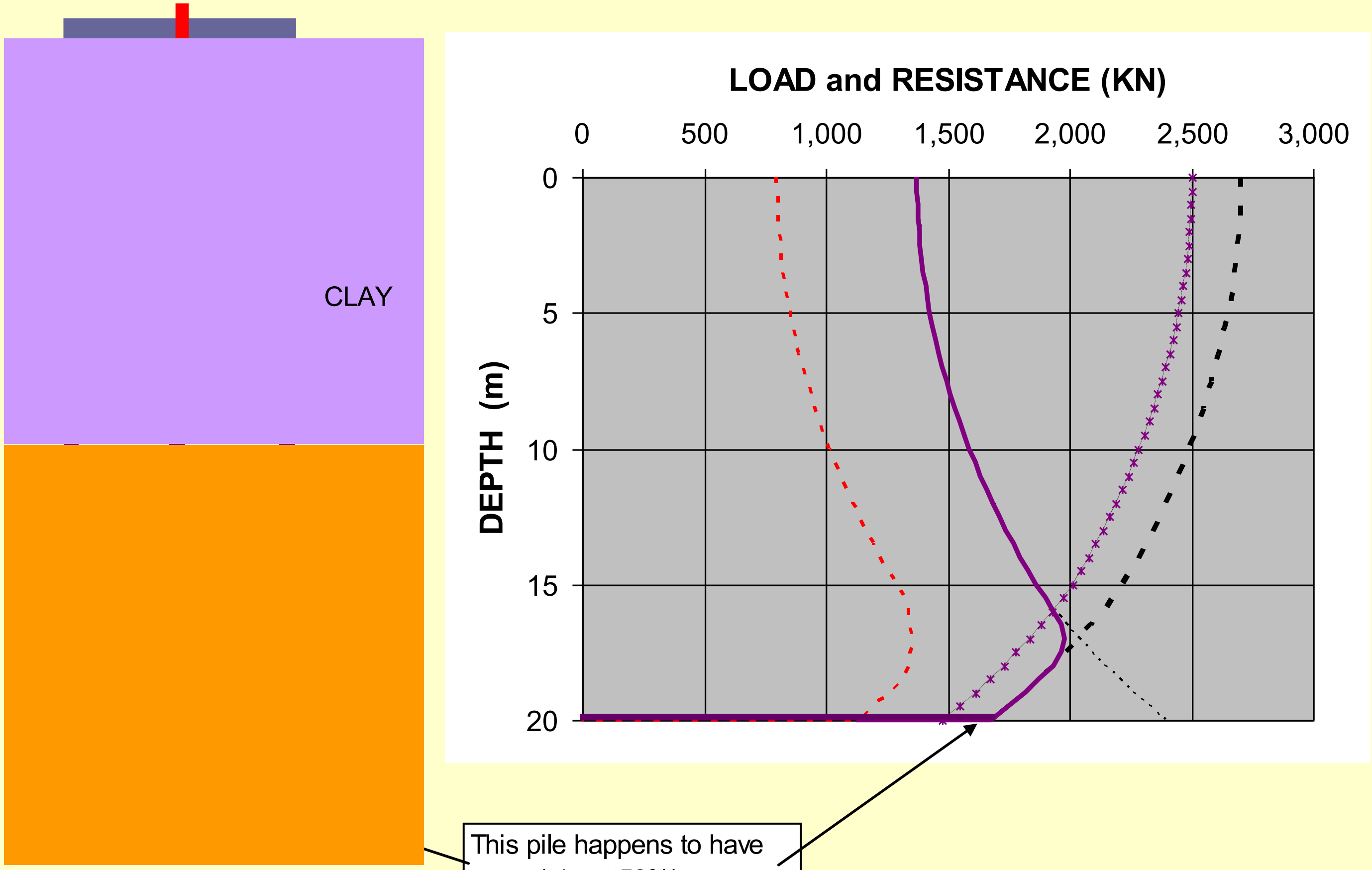
The case histories have made us realize how piles and piled foundations will respond to load and settling soil. Here are a few applications of this.

Distribution of load at the pile cap

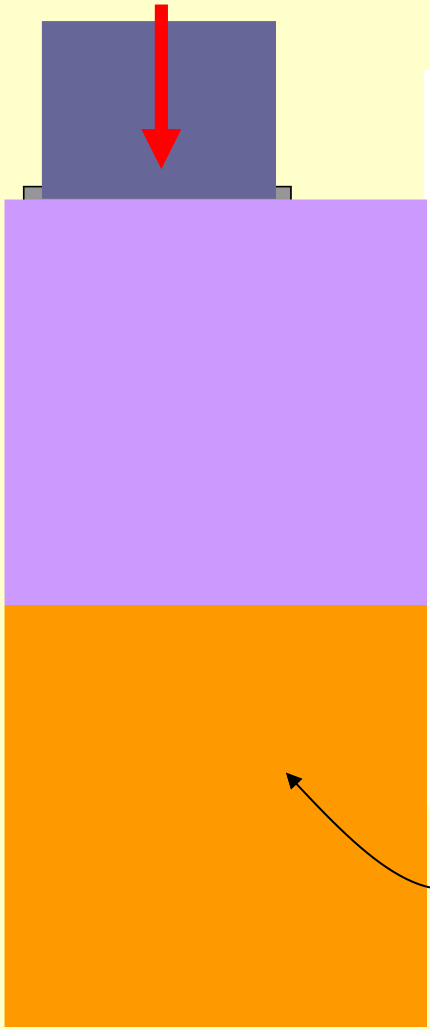




The effect of different pile length and/or different toe resistance response

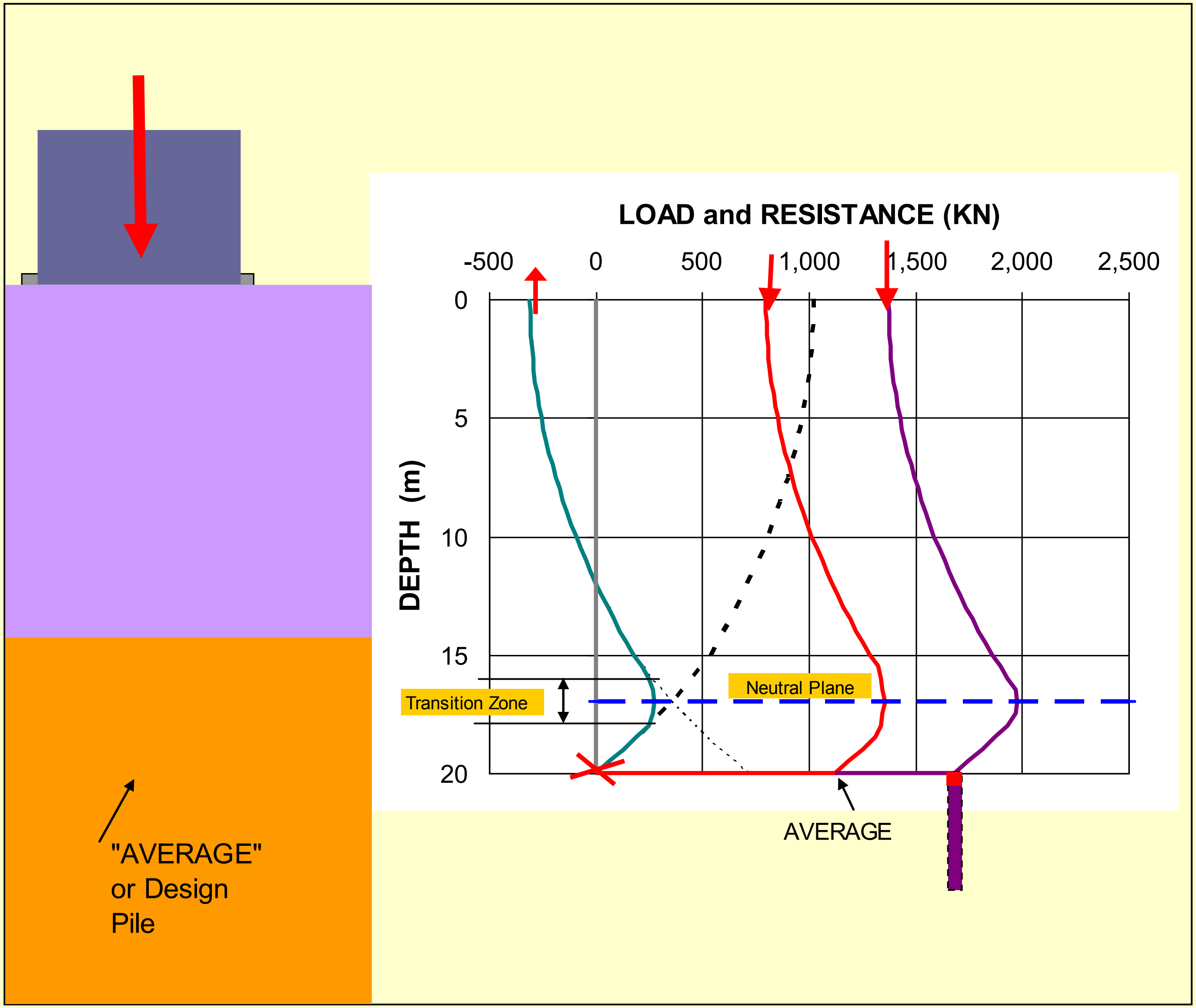


This pile happens to have more (about 50%) toe resistance or is longer.



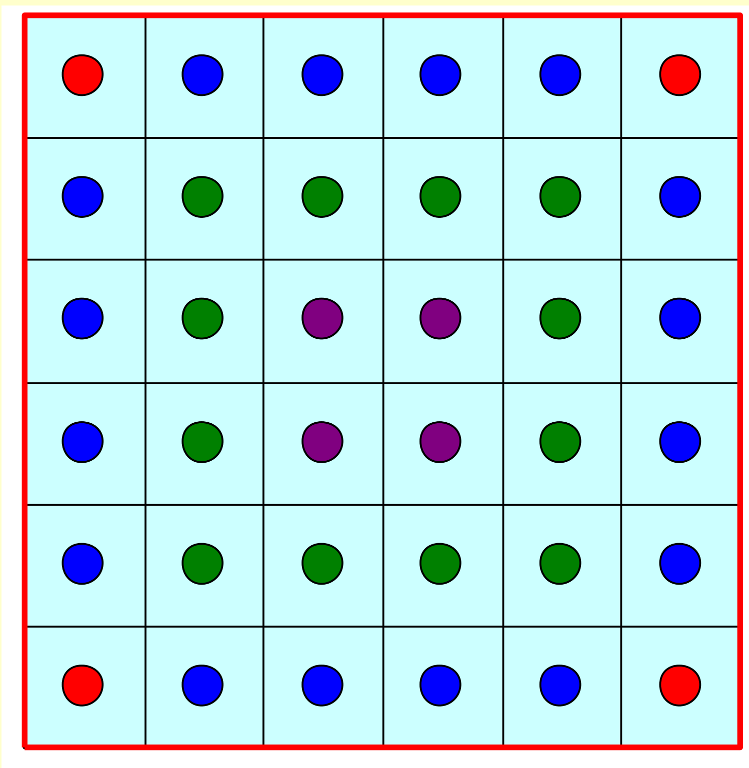
Now, let's assume that this pile is damaged at the pile toe, or that debris collected at the toe, eliminating the toe resistance.

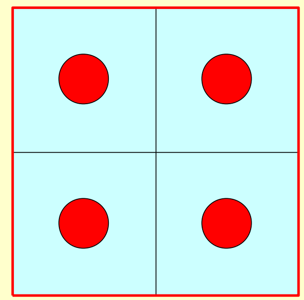
So, what is the effect of this?

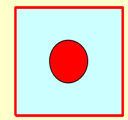

LOAD and RESISTANCE (KN)
-500
0
500
1,000
1,500
2,000
2,500
0
5
10
15
20
DEPTH (m)
Transition Zone
Neutral Plane
AVERAGE
"AVERAGE"
or Design
Pile

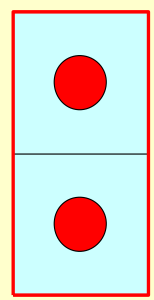
Piles inside a pile group as opposed to at the perimeter of the group

The analysis of the capacity and resistance distribution detailed in the foregoing deals with analysis of a single pile or small groups of piles, where interaction between the piles does not occur or is negligible. However, for larger groups, the group effect is substantial.

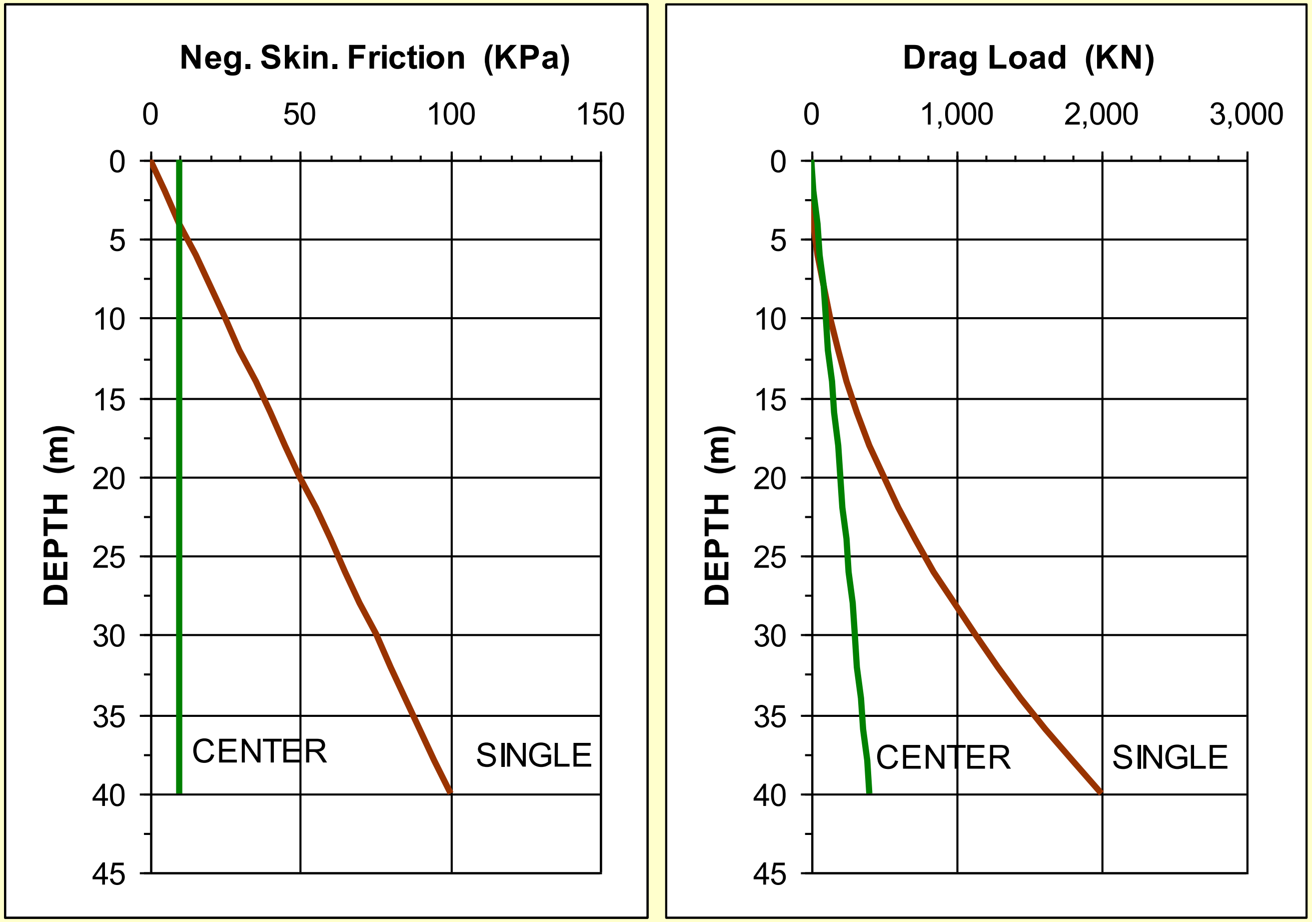




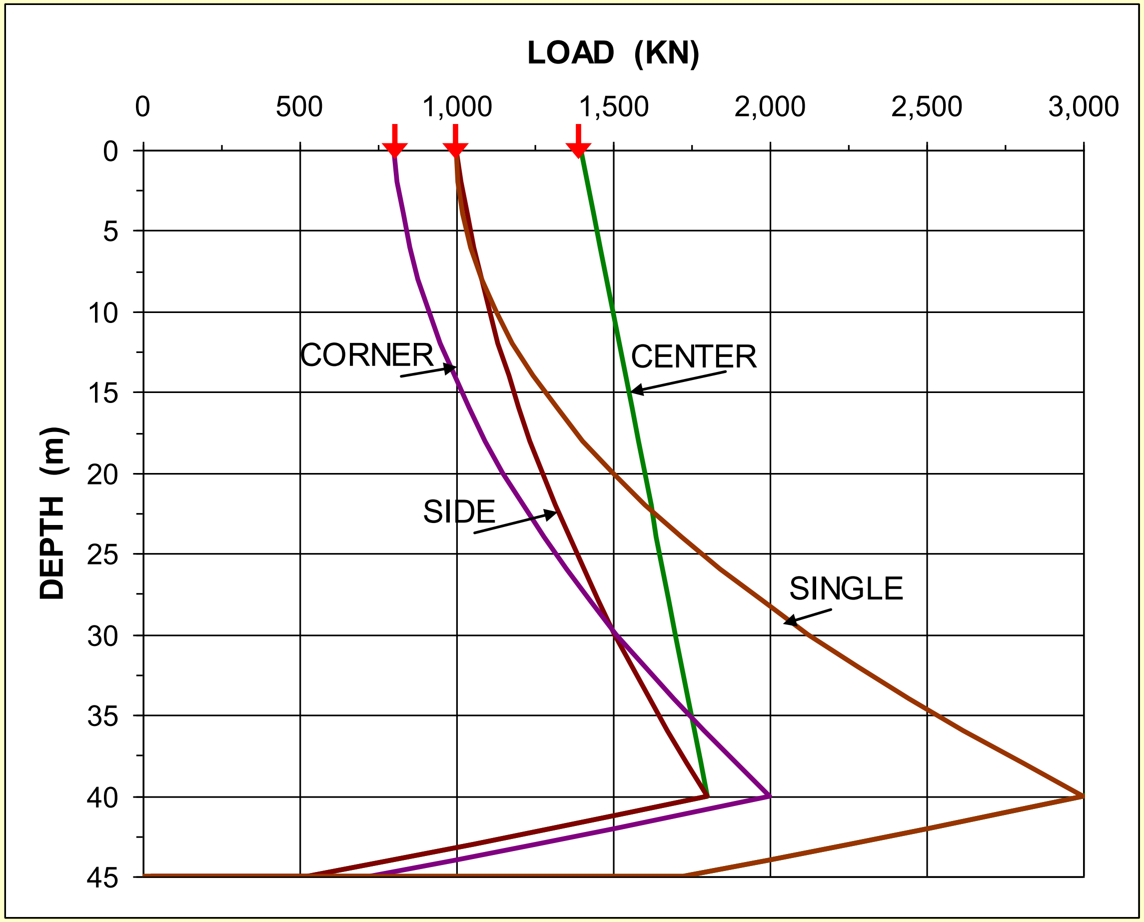




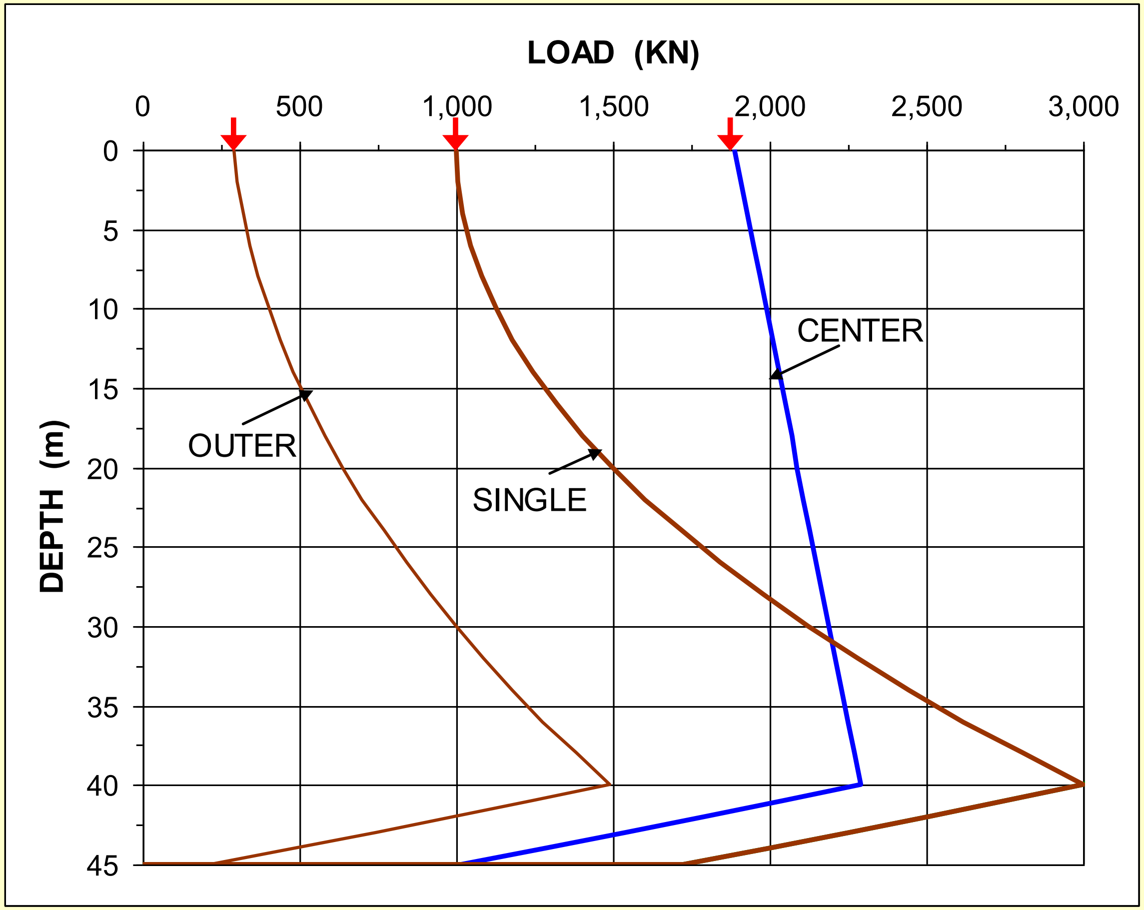
A large pile group (36 piles), two small groups (4 and 2 piles), and a single pile.



Distributions of unit negative skin friction and of accumulated drag load for a single pile and for a fully shielded pile in the center of the group.



Load distribution in the pile group piles ("Center", "Side", and "Corner") and single pile ("Single), assuming the Center piles fully shielded, and the Center and Corner piles partially shielded from the negative skin friction effect.



Load distribution in the pile group piles ("Outer" and "Corner") and single pile ("Single), assuming the Center piles fully shielded, and the Outer piles not shielded from the negative skin friction effect.

We can use our understanding to critically review design recommendations in current text books and standards. For example:

A quote from a textbook *) assigned to 4th Year students at several North American Universities

"Piles located in settling soil layers are subjected to negative skin friction called downdrag. The settlement of the soil layer causes the friction forces to act in the same direction as the loading on the pile. Rather than providing resistance, the negative skin friction imposes additional loads on the pile. The net effect is that the pile load capacity is reduced and pile settlement increases. The allowable load capacity is given as:"

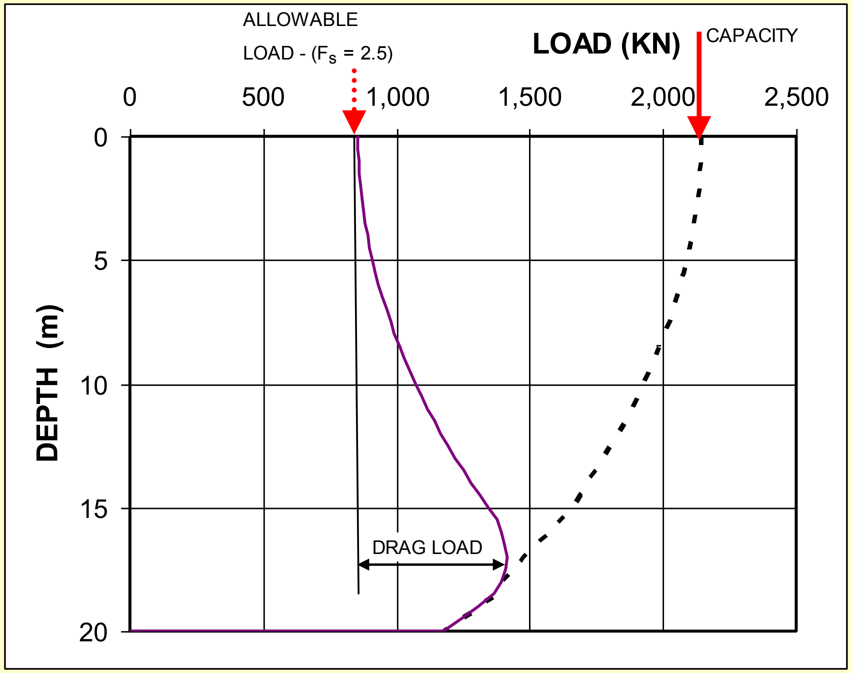
$$Q_{allow} = \frac{Q_{ult}}{F_S} - Q_{neg}$$

If you think this ghastly recommendation is correct, you have not been paying attention!

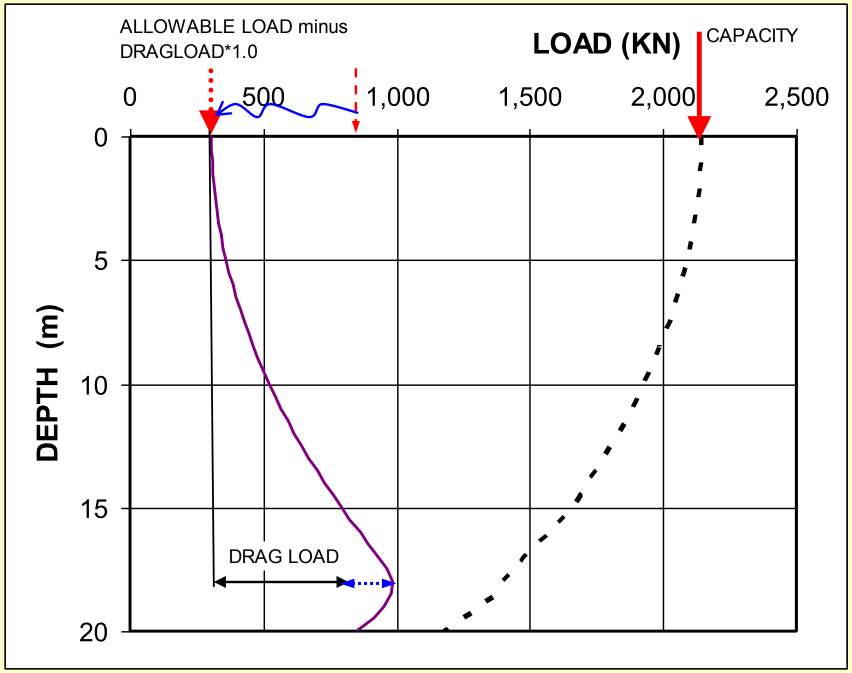
*) Compassion—perhaps misdirected—compels me not to identify the author

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

Drag load **not** subtracted from the allowable load



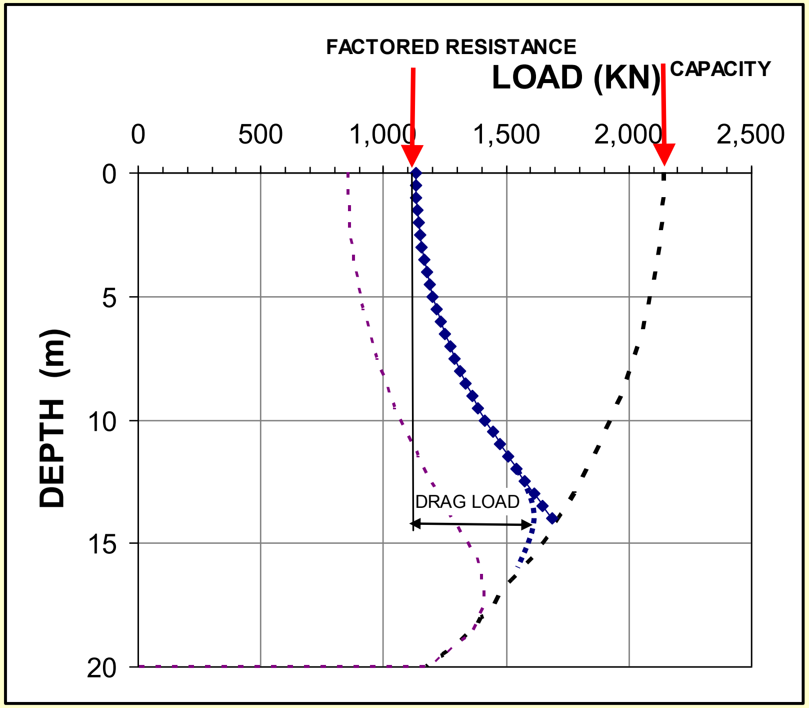
Drag load subtracted!



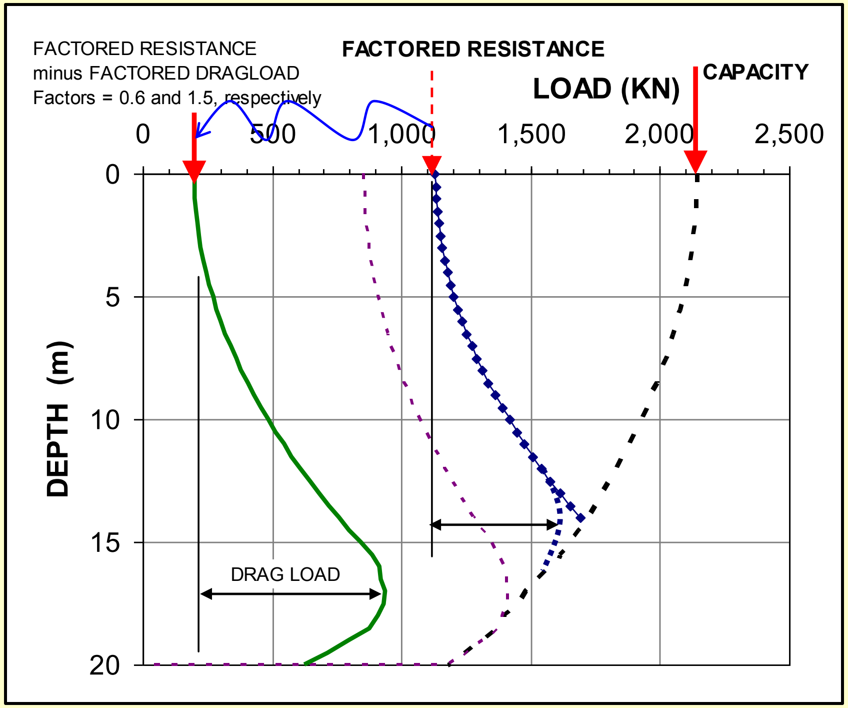
Similarly for the **LRFD**:

Do not include the drag load when determining the factored resistance!

Drag load **not** subtracted from the factored resistance



Drag load factored and subtracted!



Imagine a shaft-bearing pile (no toe resistance) with a certain capacity and an allowable load for a factor of safety of 2.0.

If a factor of safety of 2.0 is applied also to the drag load and the drag load is subtracted from the allowable load . . . , then ?

The allowable load becomes zero!

Imagine that same pile designed for uplift: Logically, if one subtracts the drag load for the push case, should one not add it for the pull case ??!!??

Do you think that there is a difference in bearing capacity between an ordinary precast and a prestressed pile? — The stress in the pile has nothing to do with the bearing capacity.

SETTLEMENT

A. **Load placed on a pile** causes downward movements of the pile head due to:

1. 'Elastic' compression of the pile.
2. Load transfer due the movement response of the soil **<u>at the pile toe</u>**. Along the shaft, movement occurs in the soil, of course, but that is irrelevant. The only soil movement affecting the movement at the pile head is that occurring at the pile toe.
3. Settlement below the pile toe due to the increase of stress in the soil. This is only of importance for large pile groups, and where the soil layers below the piles are compressible.

B. The settlement due to Points A1 and A2 is always small. However, **additional settlement can be caused by downdrag**, that is, the settlement in the soil due to factors such as fills, lowering of the groundwater table, loads placed on adjacent footings and unsupported floors, etc.

A drag load will only directly cause movement due to Point A1, the 'elastic' compression. While it could be argued that Point A2 also is at play, because the stiffness of the soil at the pile toe is an important factor here, it is the downdrag that affects (a) the pile toe movement, (b) the pile toe load, and (c) the location of the neutral plane in an interactive — "unified" — process.

The drag load cannot cause settlement due to Point A3, because there has been no stress change in the soil below the pile toe. (Note, the unloading of the soil due to negative skin friction does not result in a heave of those soil layers).

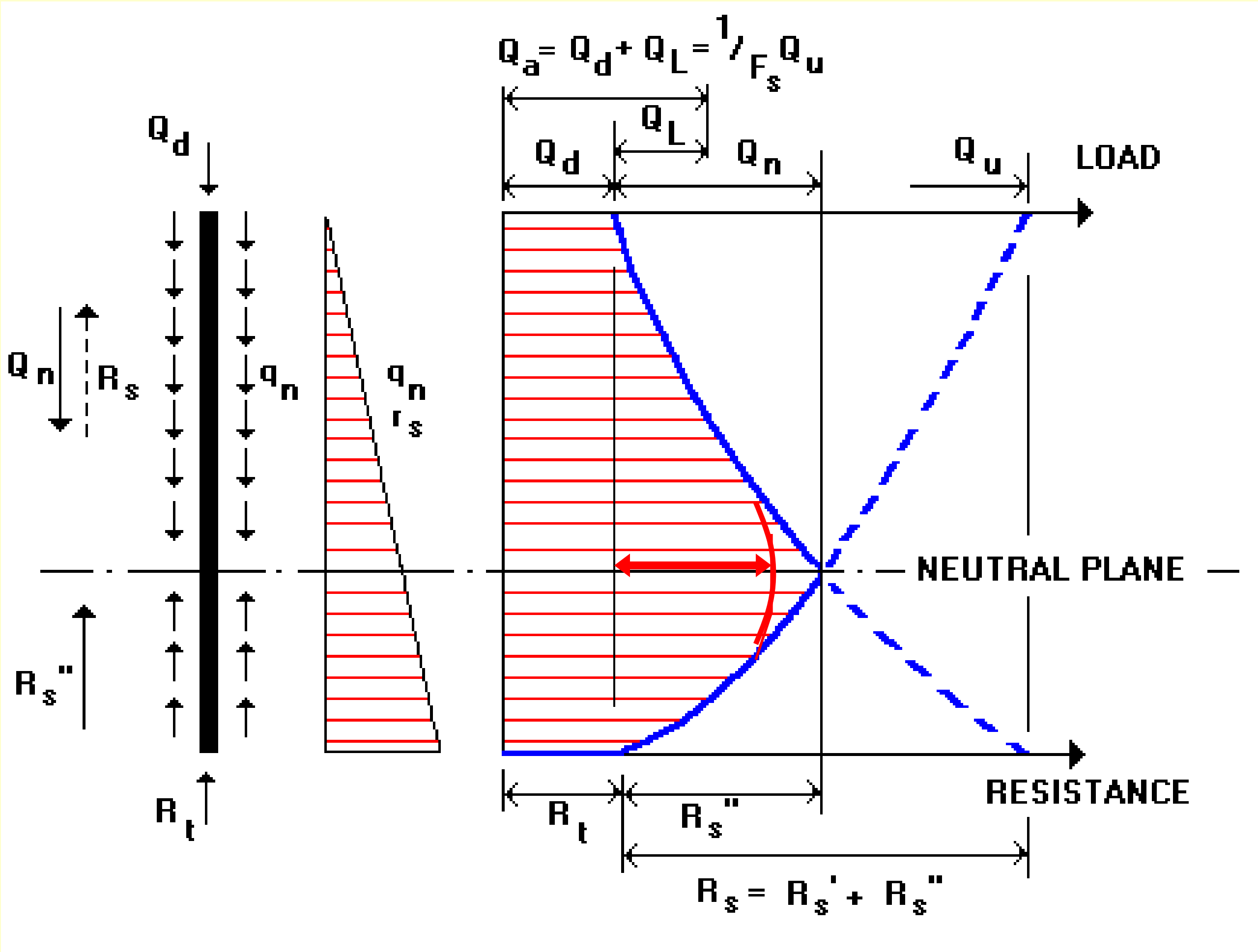
Therefore, negative-skin-friction/drag-load does not diminish capacity. Drag load (and dead load) is a matter for the pile structural strength, and the main question is if there is settlement that can cause downdrag. The approach is expressed in "The Unified Design Method".

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.

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 is not 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.

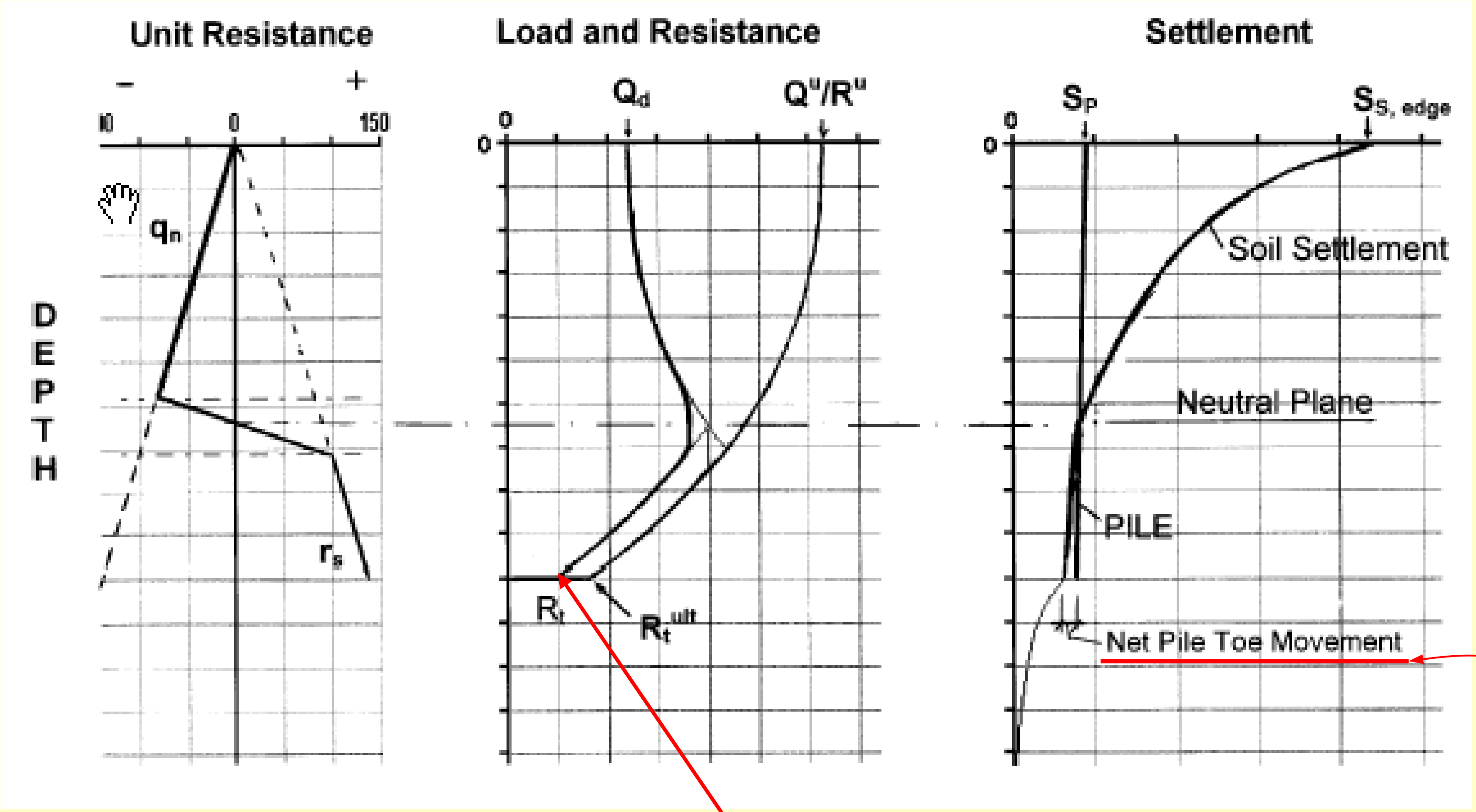


Construing the Neutral Plane and Determining the Allowable Load

The distribution of load at the pile cap is governed by the load-transfer behavior of the piles. The “design pile” can be said to be the average pile. However, the loads can differ considerably between the piles depending on toe resistance, length of piles.

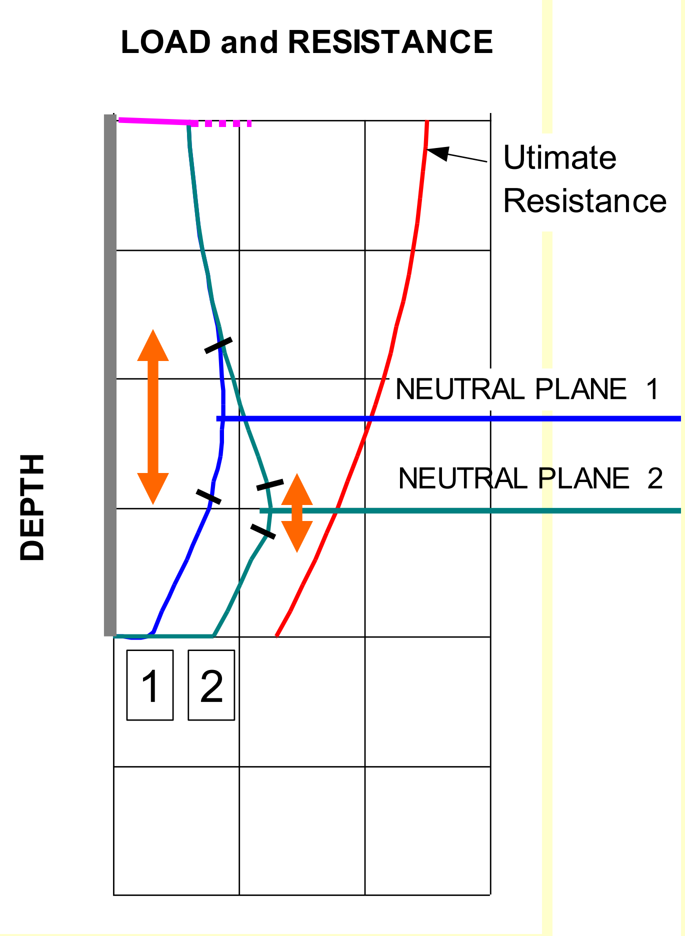
The location of the neutral plane is the result of Nature’s iterations to find the force equilibrium. If the end result — by design or by mistake — is that the neutral plane lies in or above a compressible soil layer, the pile group will settle even if the total factor of safety appears to be acceptable.

The principles of the mechanism are illustrated in the following three diagrams



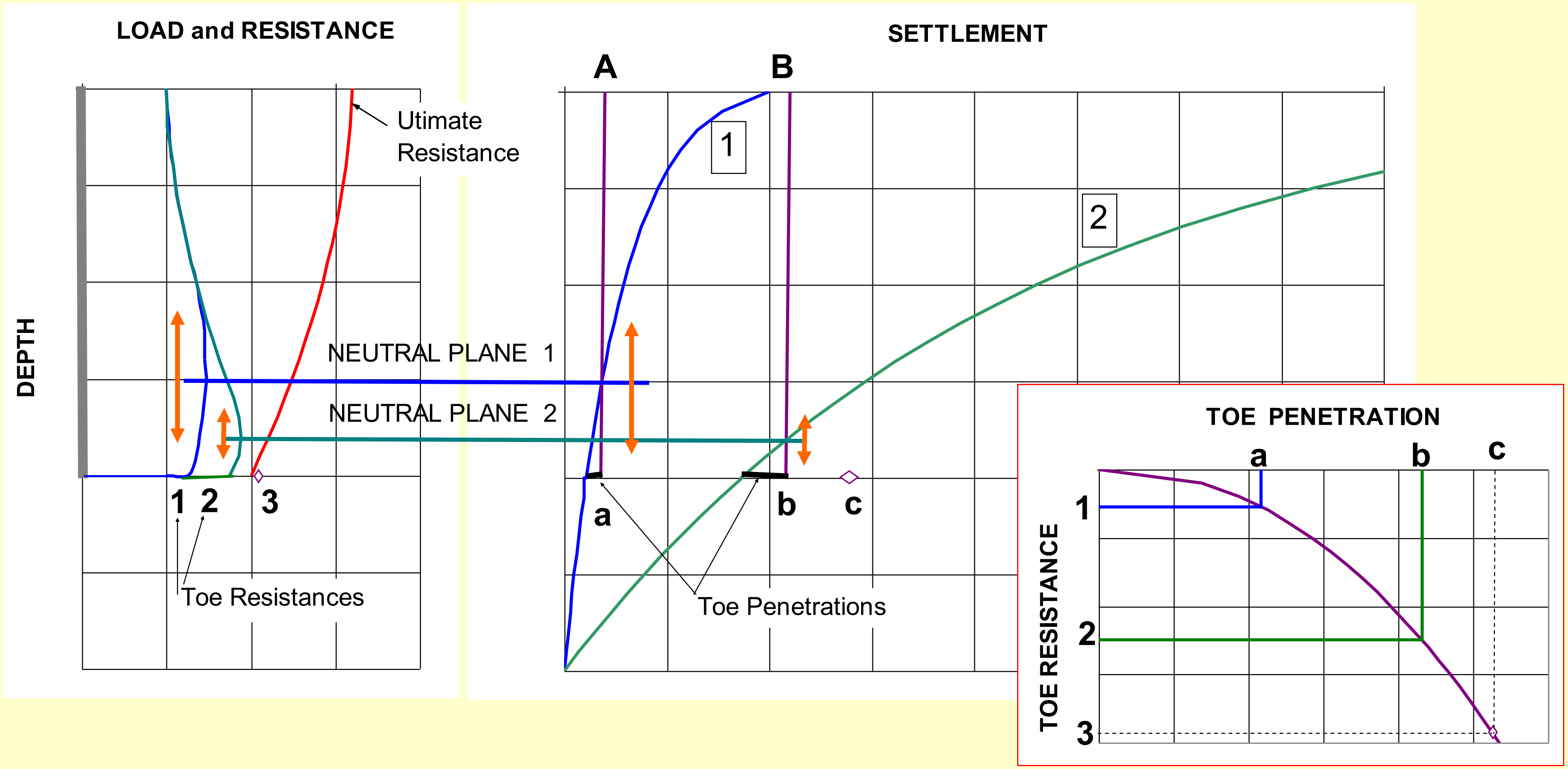
The mobilized toe resistance, $\mathbf{R_t}$, is a function of the **Net Pile Toe Movement**

Pile toe response for where the settlement is small (1) and where it is large (2)



Note, the magnitude of settlement affects not only the magnitude of toe resistance but also the length of the **Transition Zone**

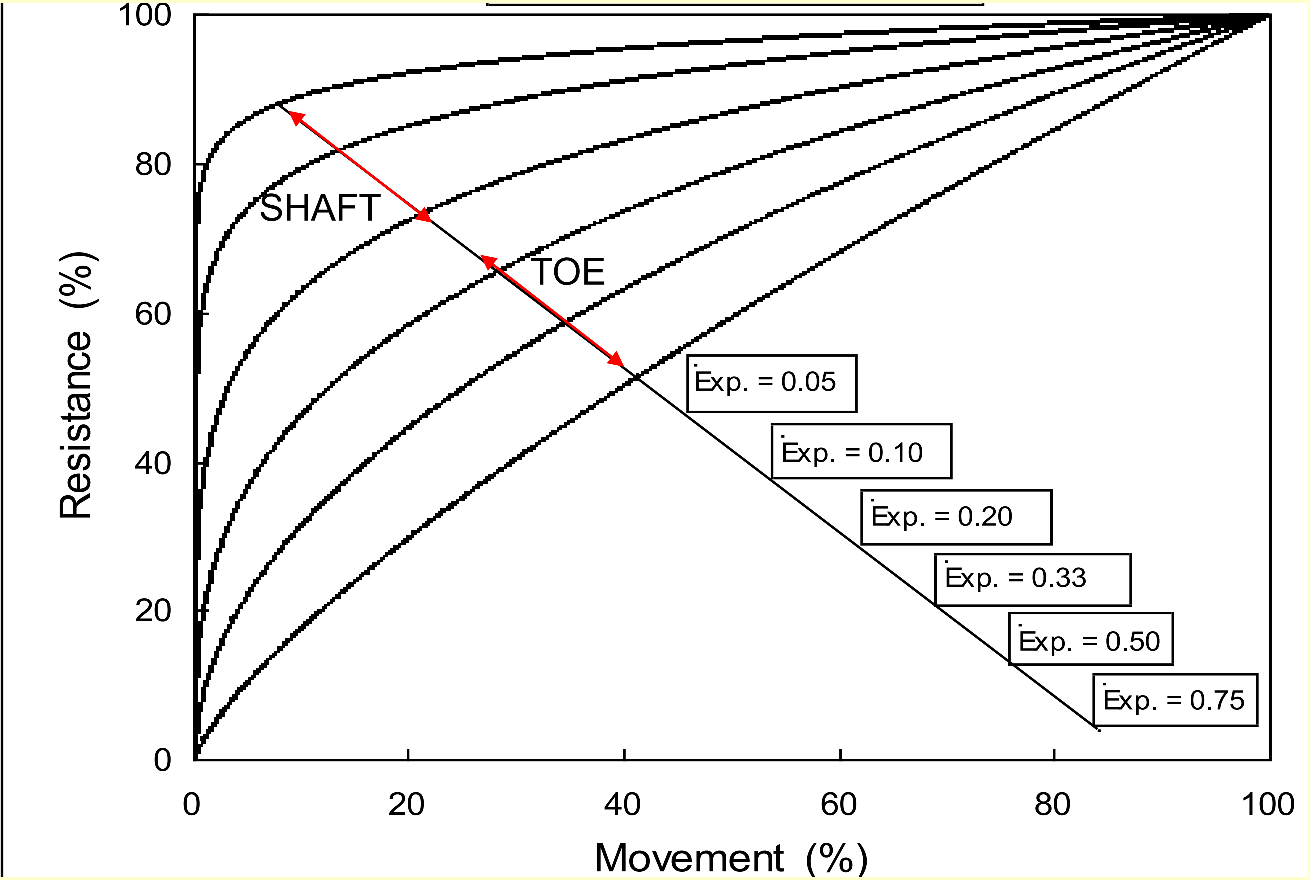
Pile toe response for where the settlement is small (1) and where it is large (2), showing <u>toe penetration</u>



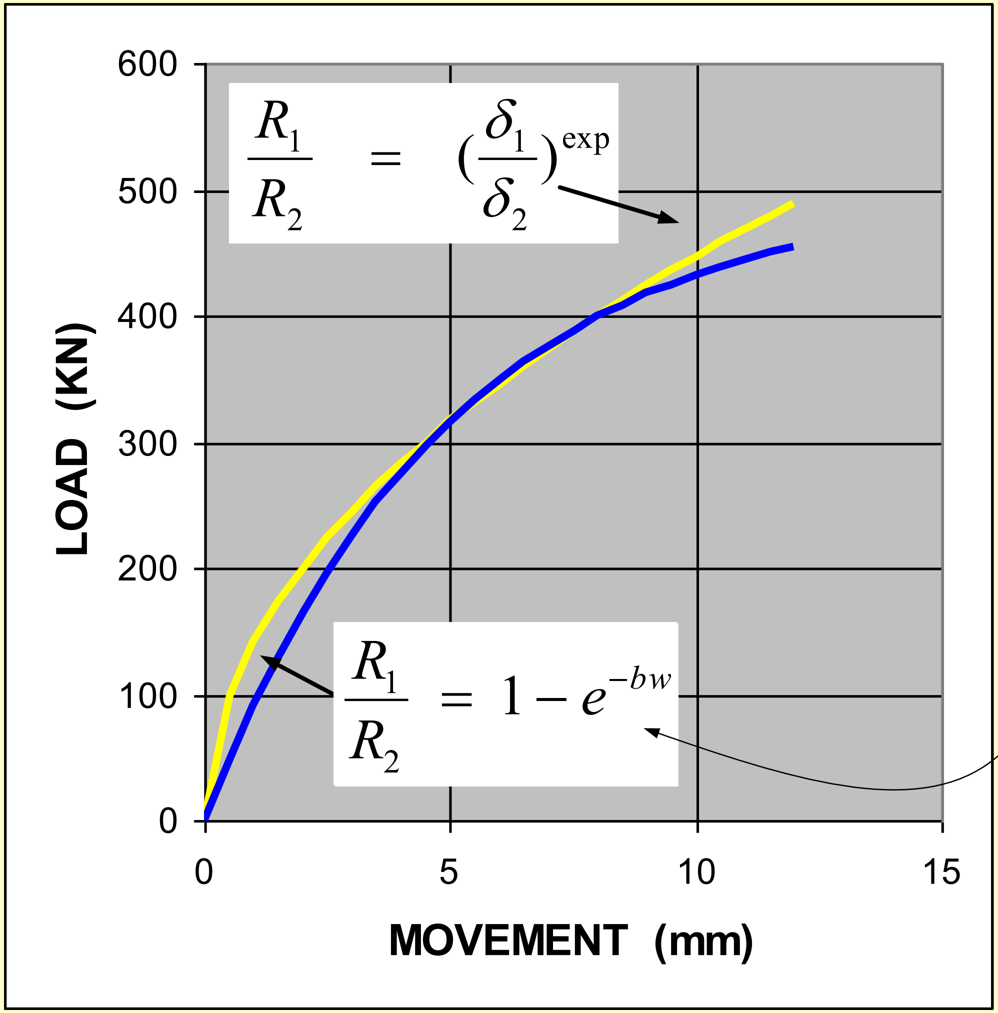
Note, the magnitude of settlement affects not only the magnitude of toe resistance but also the length of the **Transition Zone:**

Load-movement relations

Pile shaft by t-z relation
Pile toe by q-z relation



$$\frac{R_1}{R_2} = \left(\frac{\delta_1}{\delta_2}\right)^{\exp}$$



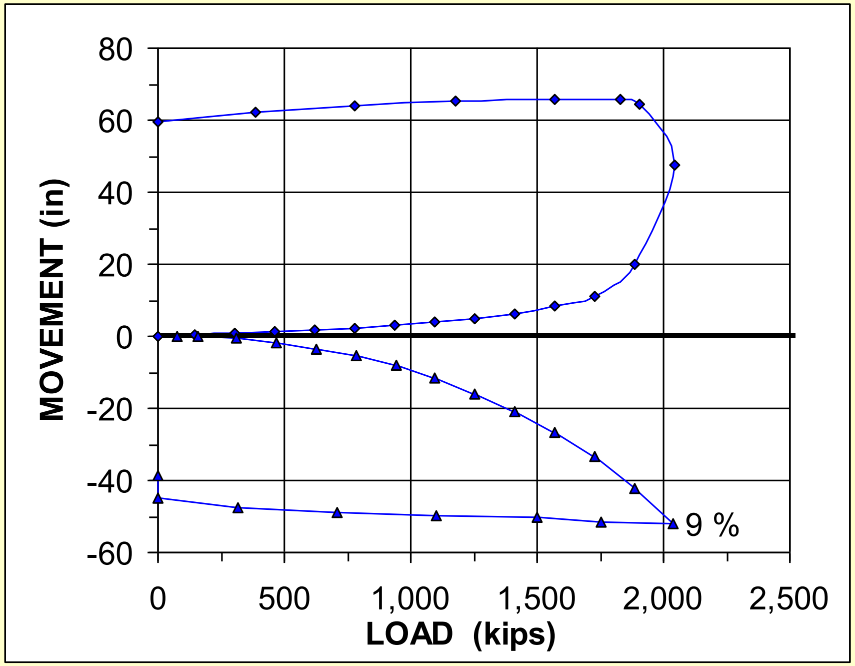
Alternative expression

b = Constant = about 0.04 – 0.15

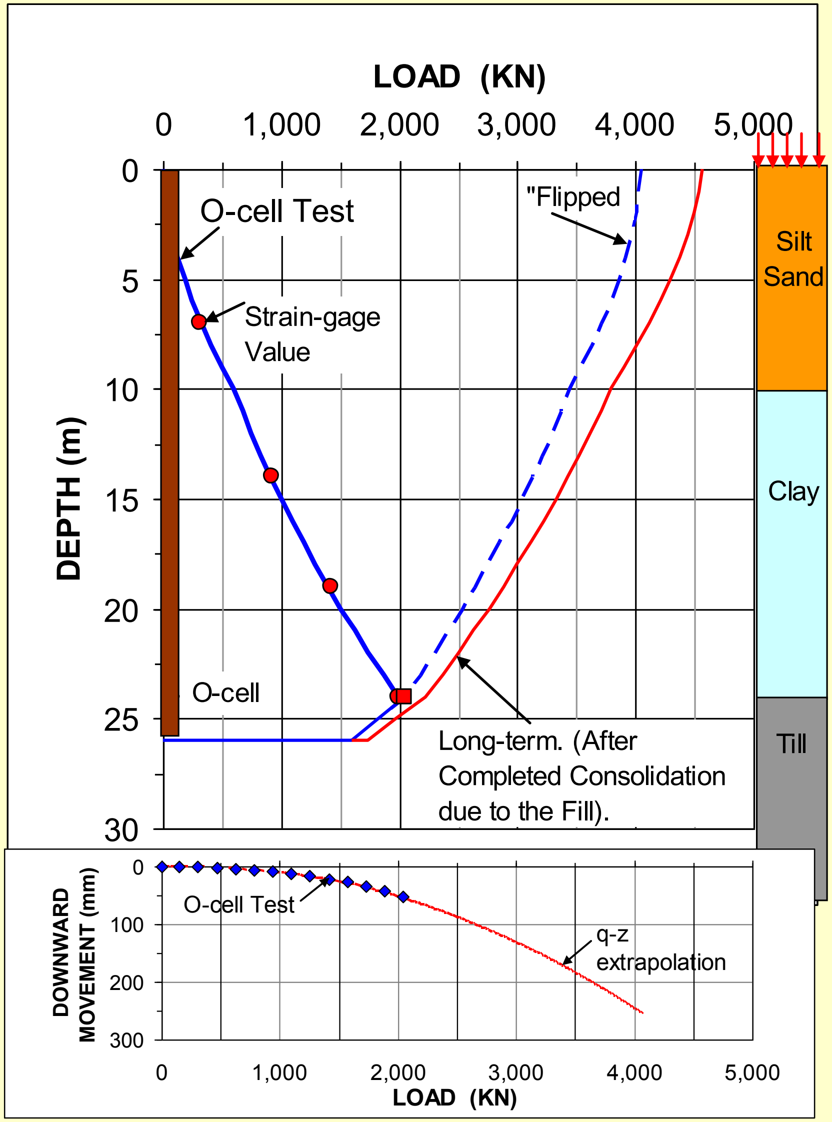
w = Penetration, δ

Example of the Unified Design Approach

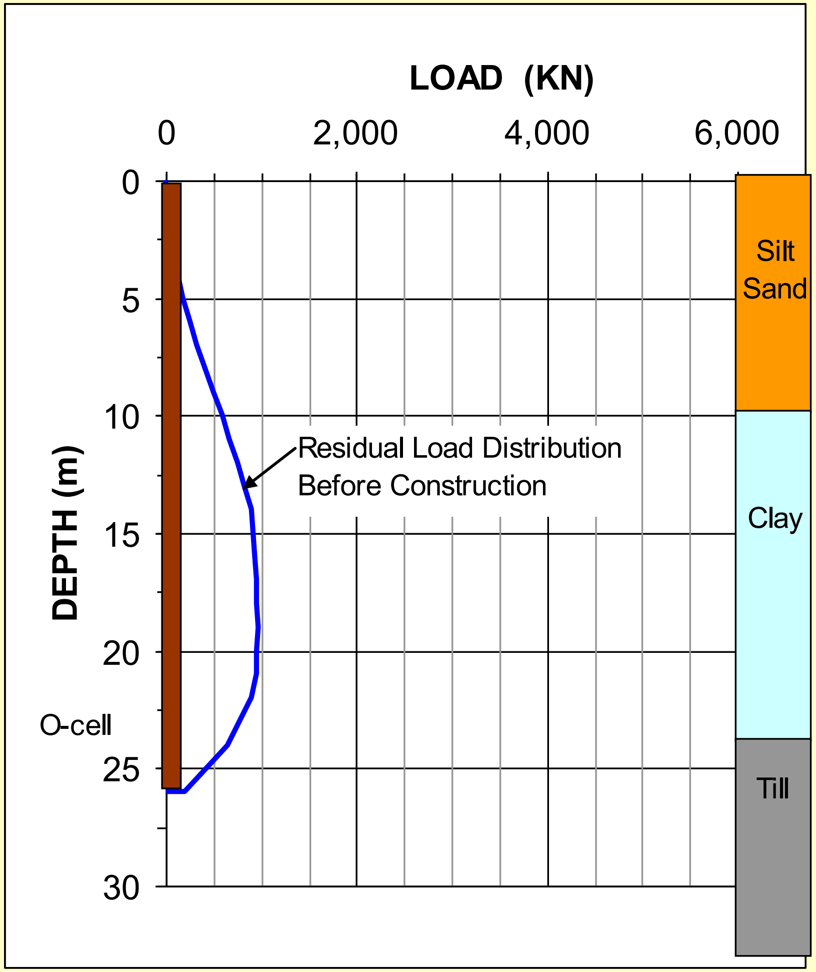
Results of an O-cell test on a 575-mm diameter, 26 m deep bored cylindrical pile. A 1.2 m thick fill will be placed over the site before construction. Piles are single or in small groups.



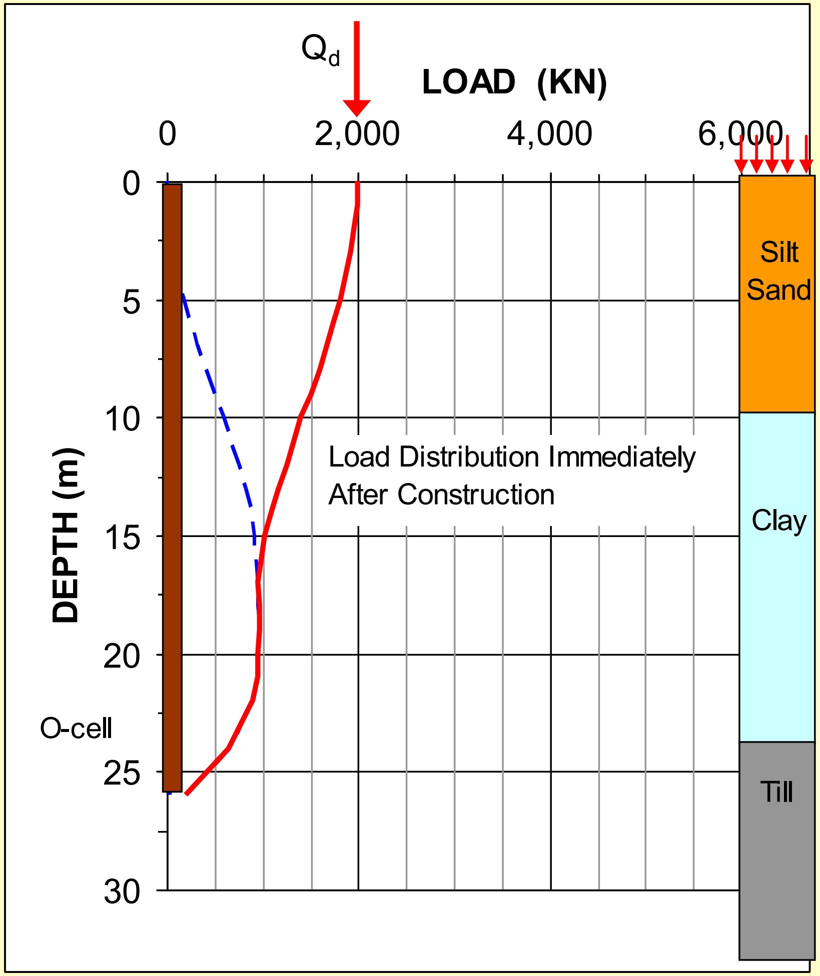
Results of analysis of test data:
Load Distributions



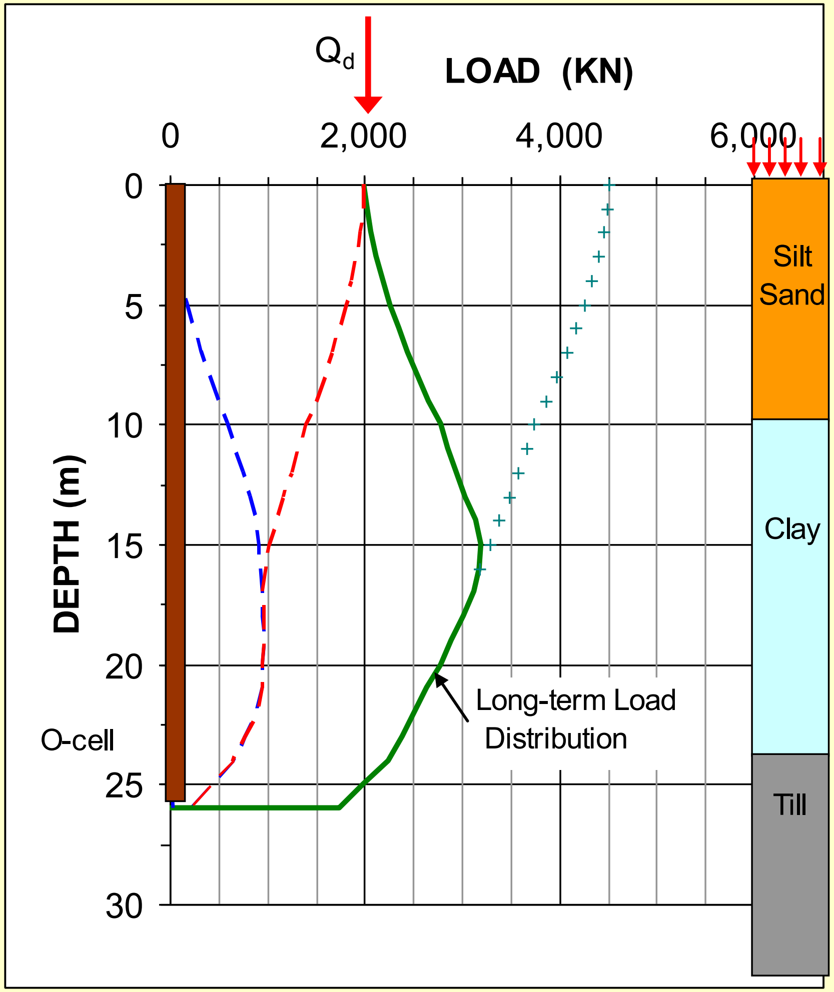
Distribution of residual load in the pile after installation, but before load is applied to the pile.



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



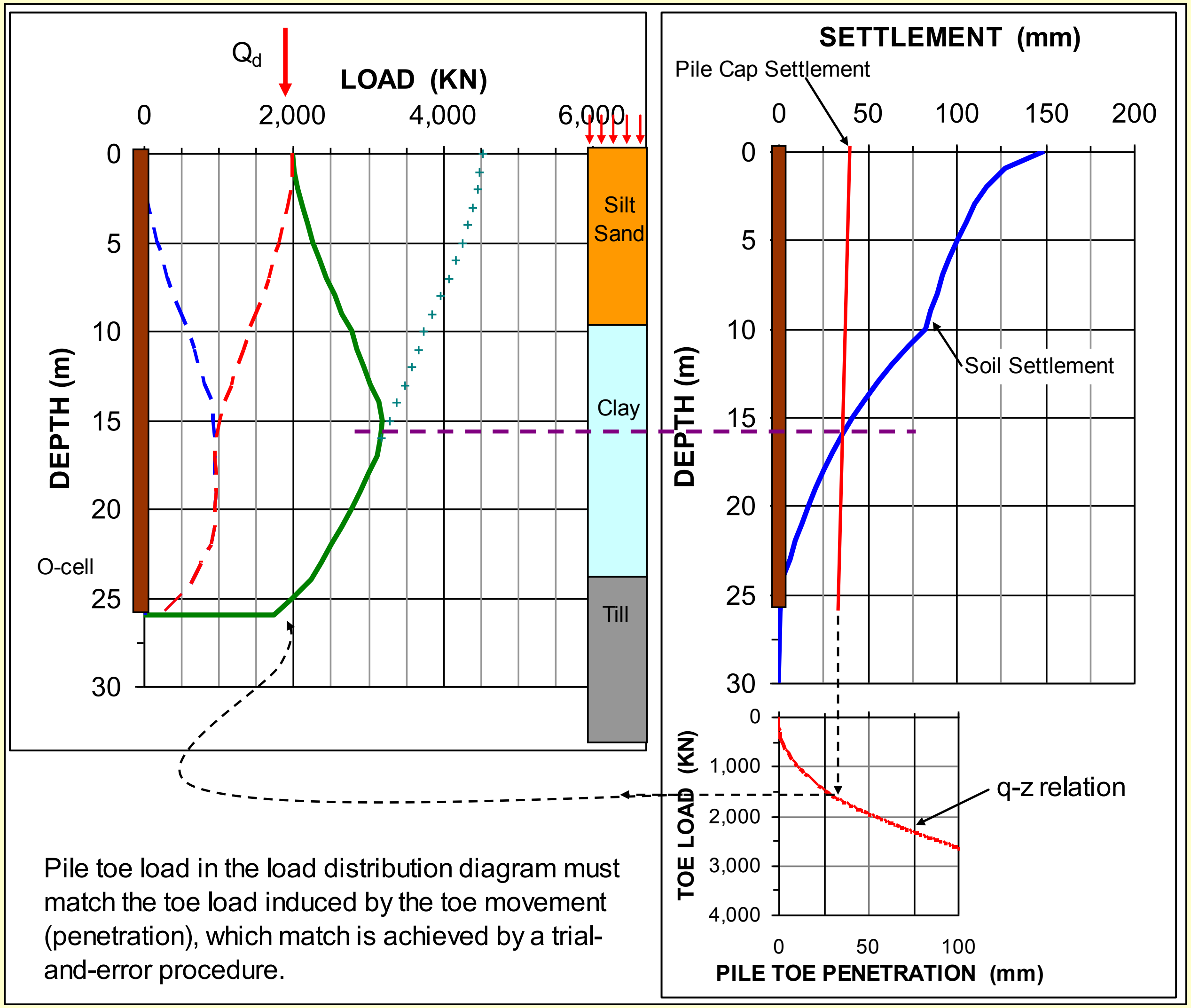
Long-term load distribution



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.

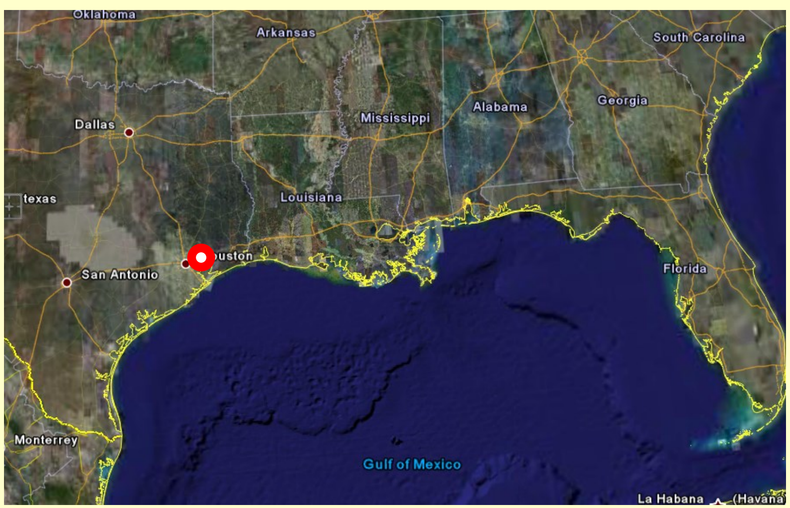
The unified pile design for capacity, drag load, settlement, and downdrag

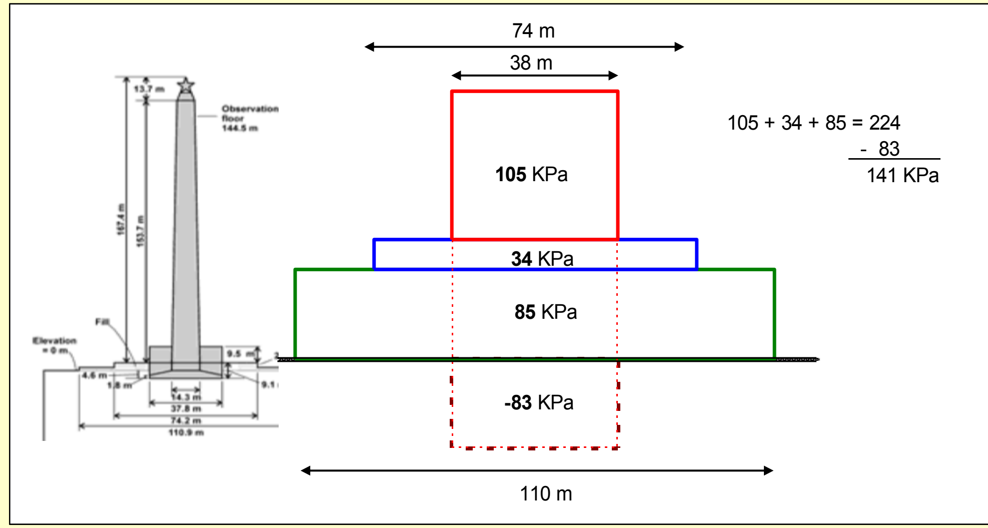


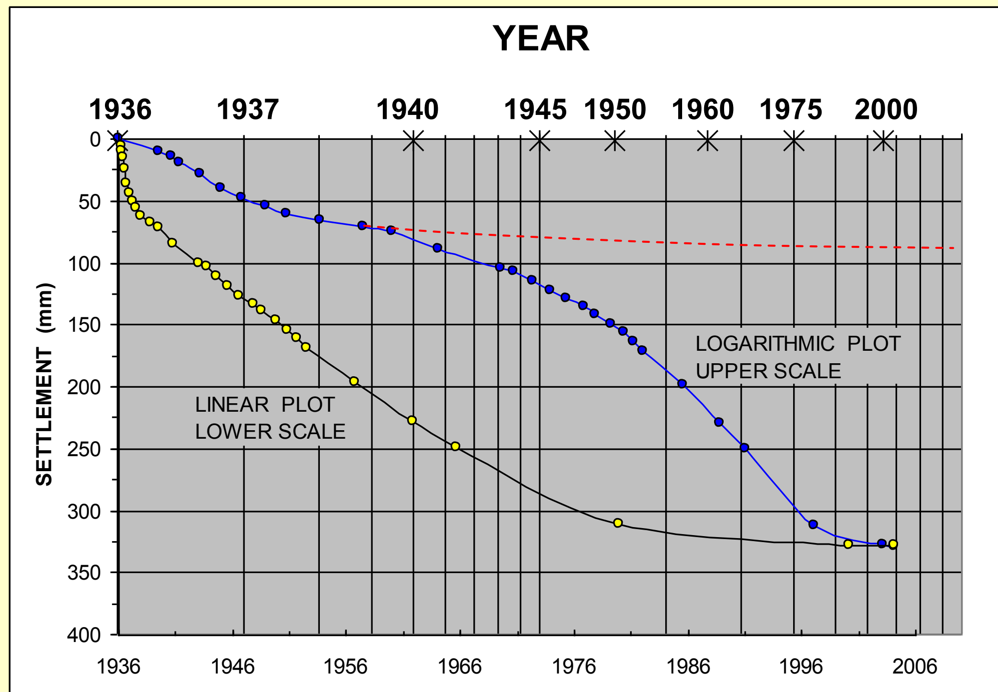
Pile toe load in the load distribution diagram must match the toe load induced by the toe movement (penetration), which match is achieved by a trial-and-error procedure.

MORE ON SETTLEMENT

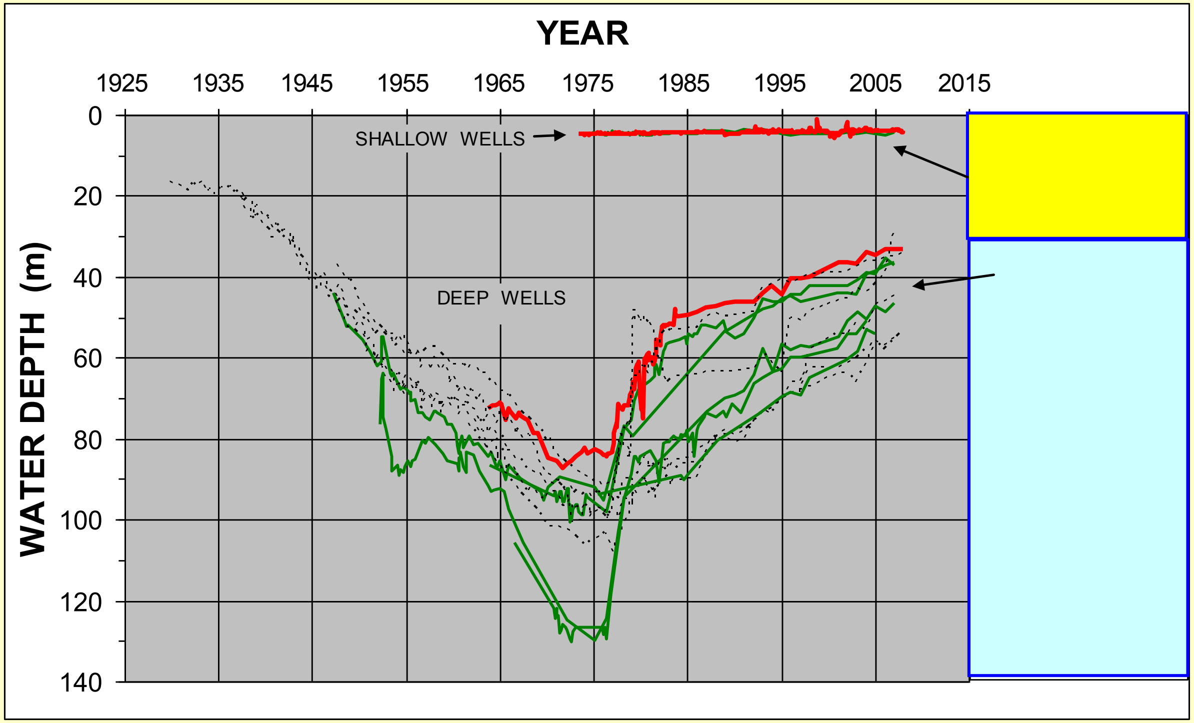
The San Jacinto Monument.





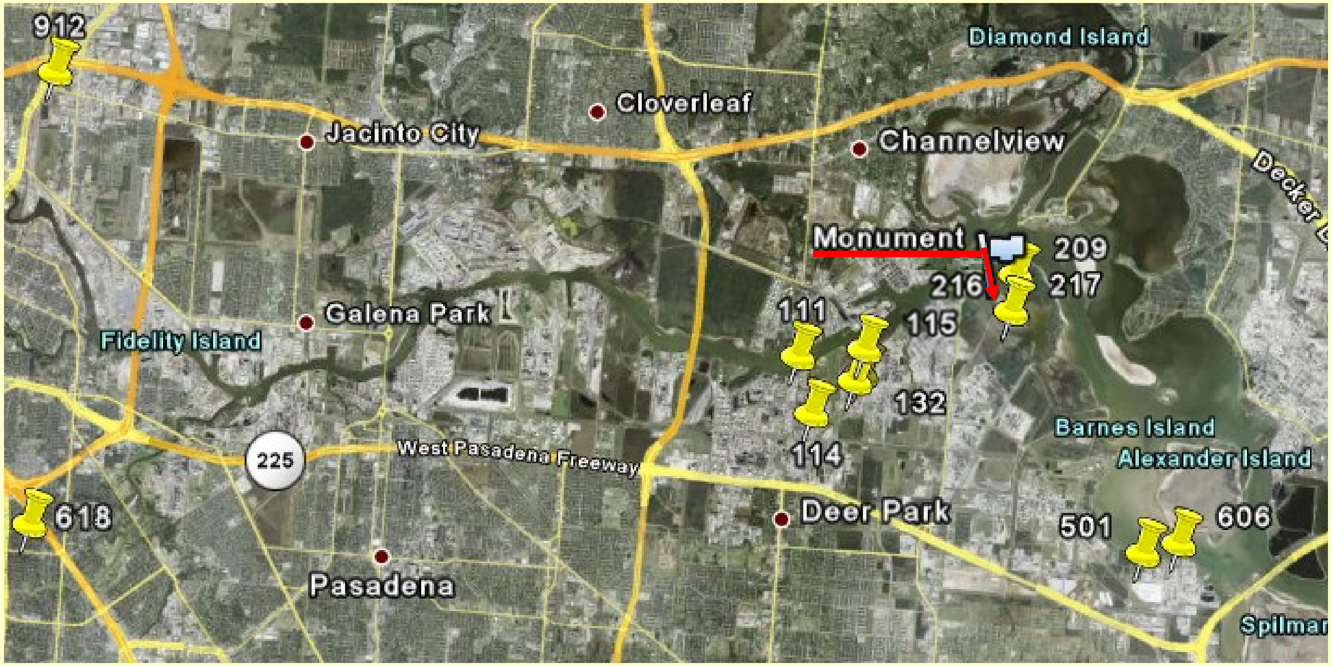


Briaud et al. 2007; Fellenius and Ochoa 2008



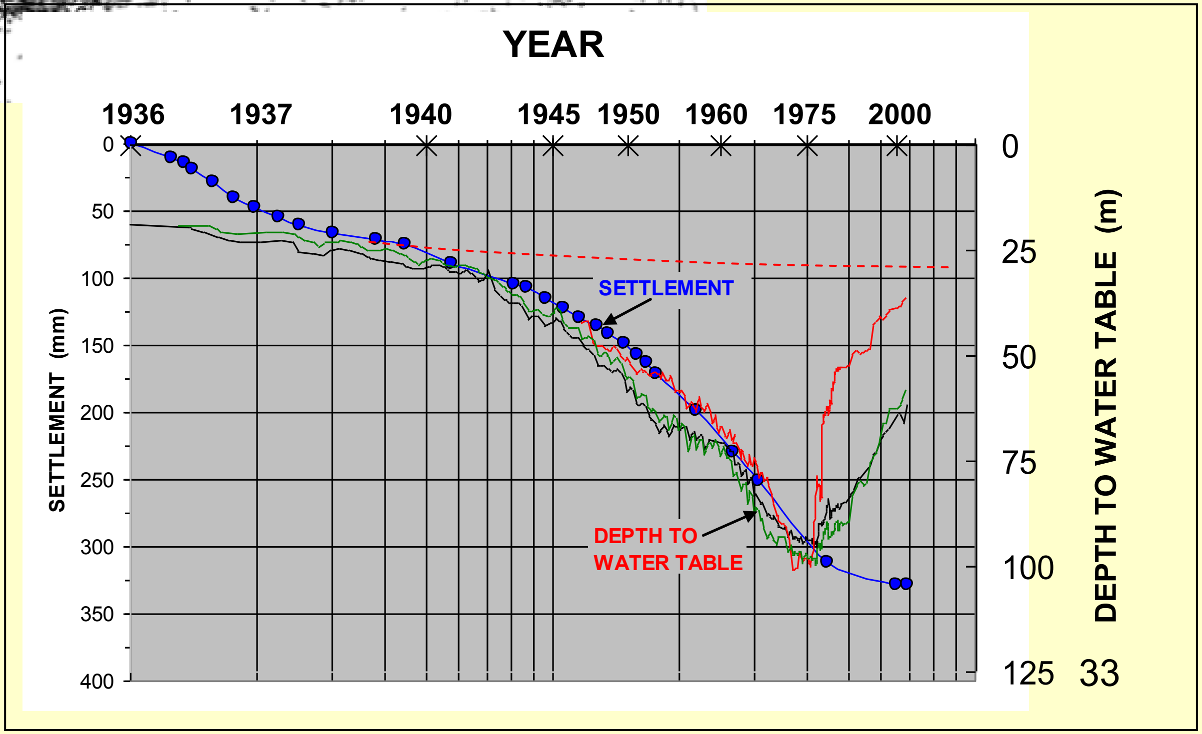
Water Depths Measured in Deep Wells

Monument and Well Locations

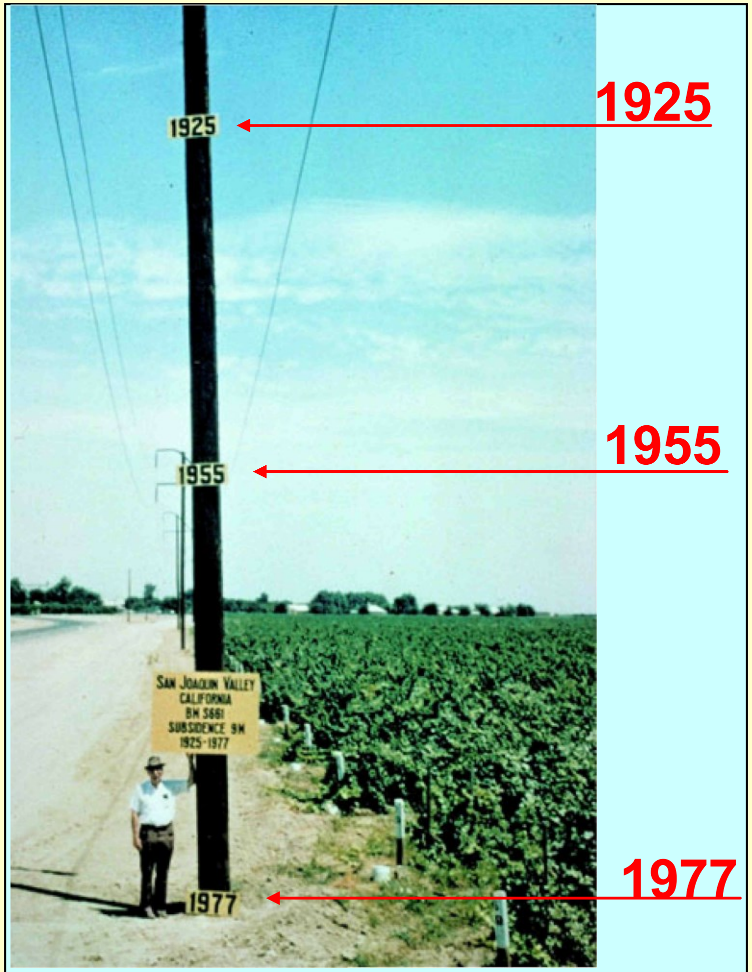


Well head at Burnett School, Baytown, Texas

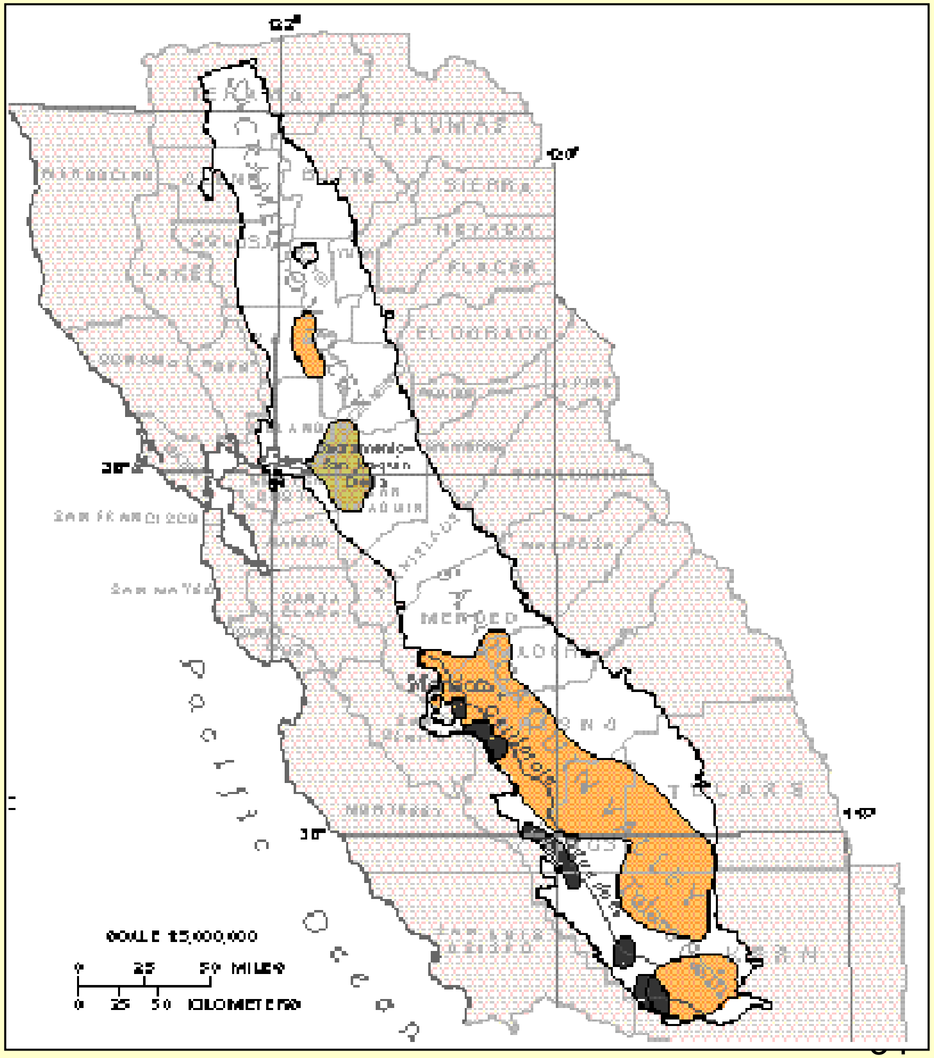
San Jacinto Monument
Settlement and Measured
Depths to Water in the
Wells Plotted Together



The lowering of the pore pressures due to mining of water and subsequent regional settlement is not unique for Texas. Another such area is Mexico City, for example. Here is a spectacular 1977 photo from San Joaquin, California.



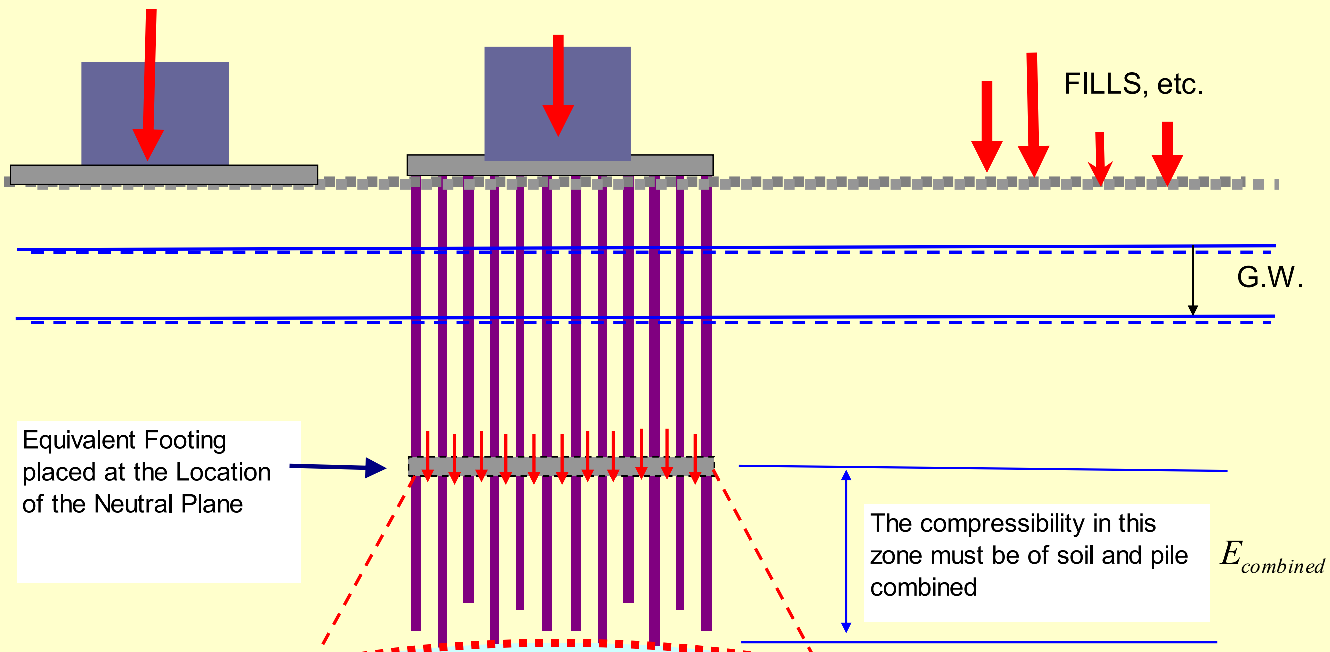
Subsidence at San Joaquin Valley, California



The **settlement** is often the most critical of the three governing aspects (*Capacity, Structural Strength, and Settlement*). It is therefore unfortunate that settlement analysis is so frequently omitted from the design of piled foundations

- The load on the piles contributes very little to the settlement of a piled foundation
- Settlement is caused by an increase of effective stress
- Settlement of a pile group is the settlement caused by the increase of effective stress in the soil layers **below the Neutral Plane** due, usually, to loads other than the load on the pile cap
- **Downdrag** is <u>not a synonym for drag load</u>, but is settlement of the pile group caused by loads from sources other than the load on the pile group
- The settlement of a large foot-print piled foundation (large pile group) can be estimated as the settlement of an **Equivalent Footing** (or **Equivalent Raft**) placed at the Neutral Plane. Note, the settlement according to this analysis occurs to the largest extent below the pile toe depth.

Settlement Analysis of <u>Large</u> Pile Groups by the Equivalent Footing Method



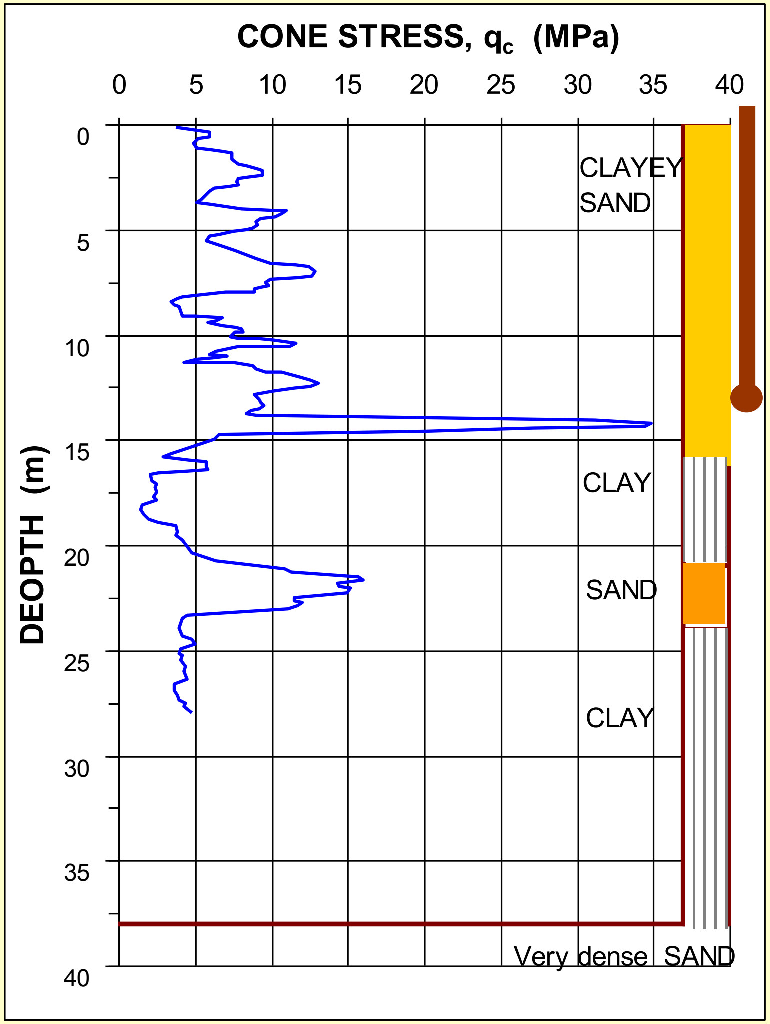
$$E_{combined} = \frac{A_{pile}\,E_{pile} + A_{soil}\,E_{soil}}{A_{pile} + A_{soil}}$$

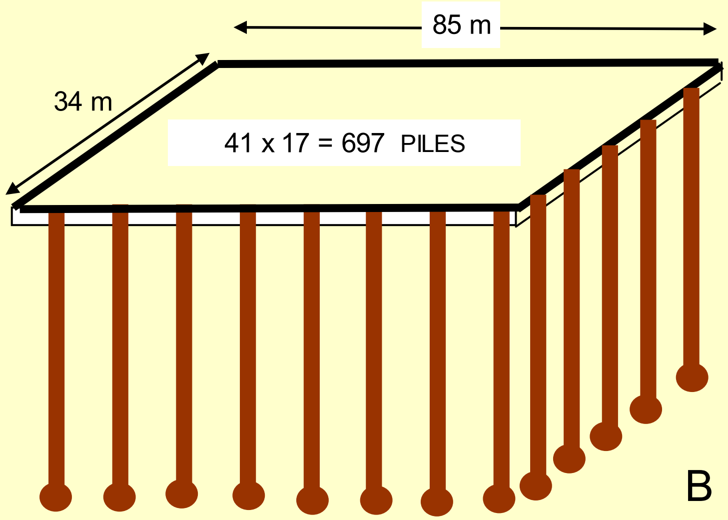
This approach is only relevant to settlement below the pile toe level! That is, it only applies to large pile groups.

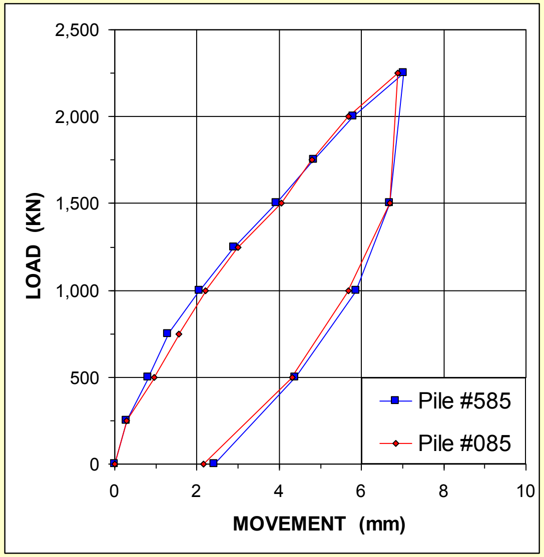
N.B., DO NOT CONFUSE THIS WITH THE LOAD-TRANSFER MOVEMENT

PILE GROUP CASE HISTORY

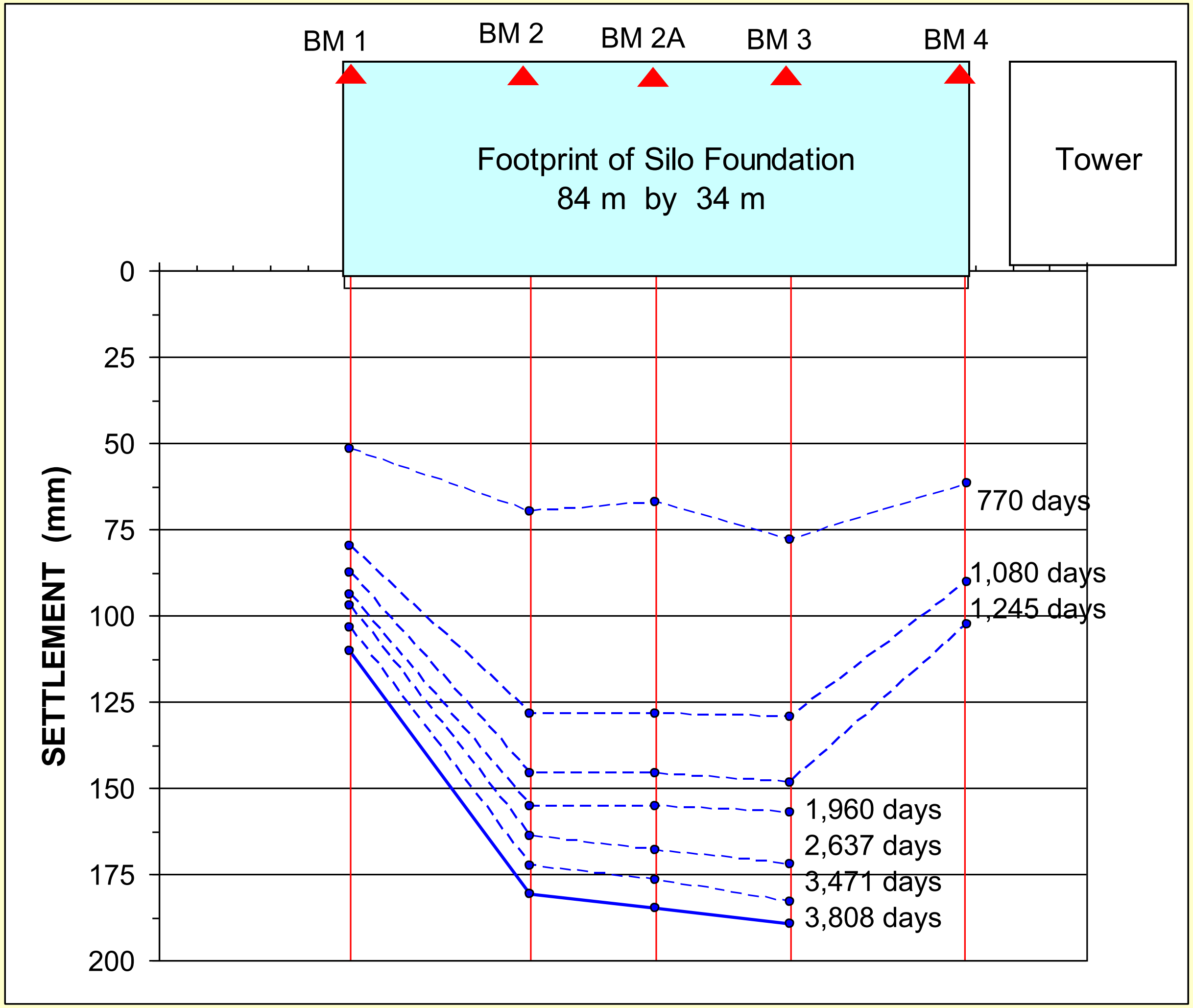
Ghent Grain Terminal — Settlement of a large pile group
(Goossens and VanImpe, 1991)







Can we use the results of the two static loading tests to estimate the settlement of the pile group?

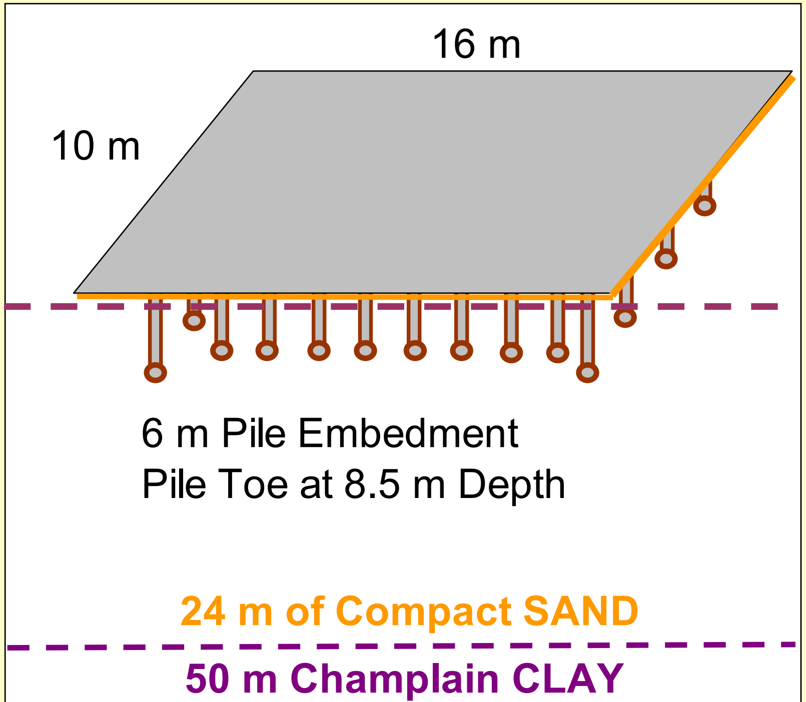


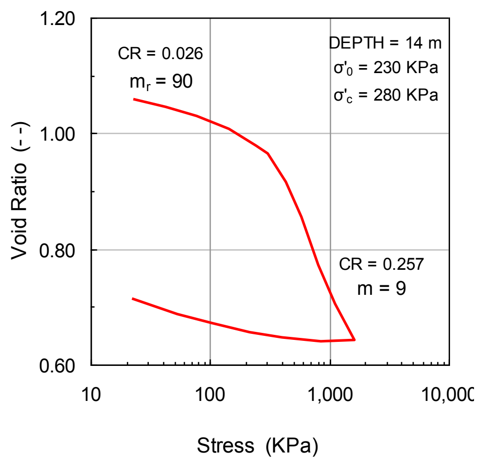
BM 1
BM 2
BM 2A
BM 3
BM 4
Footprint of Silo Foundation
84 m by 34 m
Tower
SETTLEMENT (mm)
0
25
50
75
100
125
150
175
200
770 days
1,080 days
1,245 days
1,960 days
2,637 days
3,471 days
3,808 days

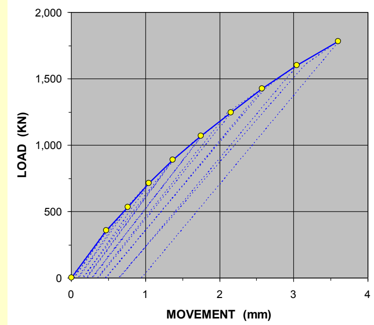
Settlement of a Pile Group Supporting Five Furnaces at QIT Plant, Sorel, Quebec

Golder, H.Q. and Osler J.C., 1968.
Settlement of a furnace foundation, Sorel, Quebec.
Canadian Geotechnical Journal, Vol. 5, No. 1, pp. 46 - 56.

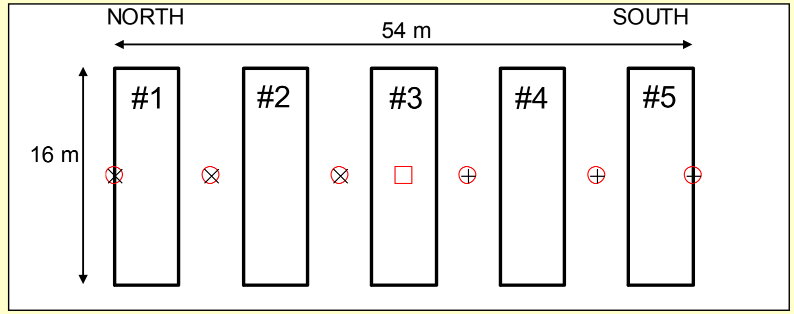
LAYOUT OF ONE FURNACE

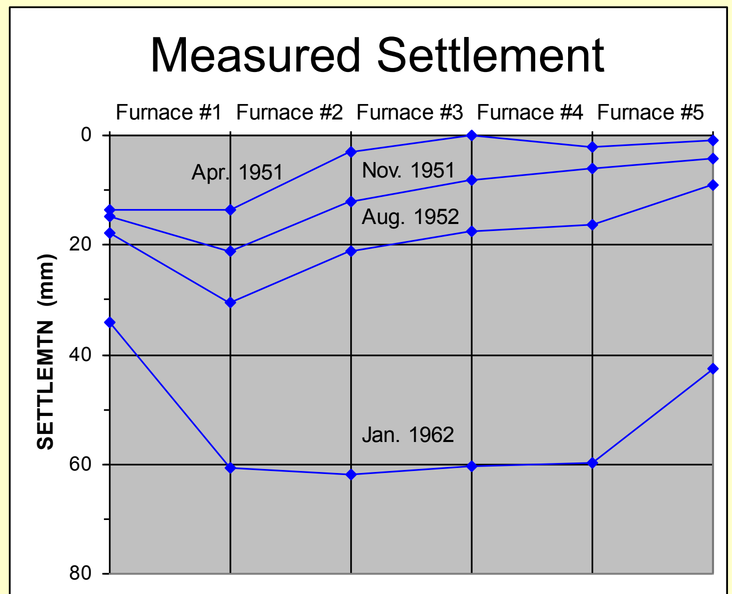


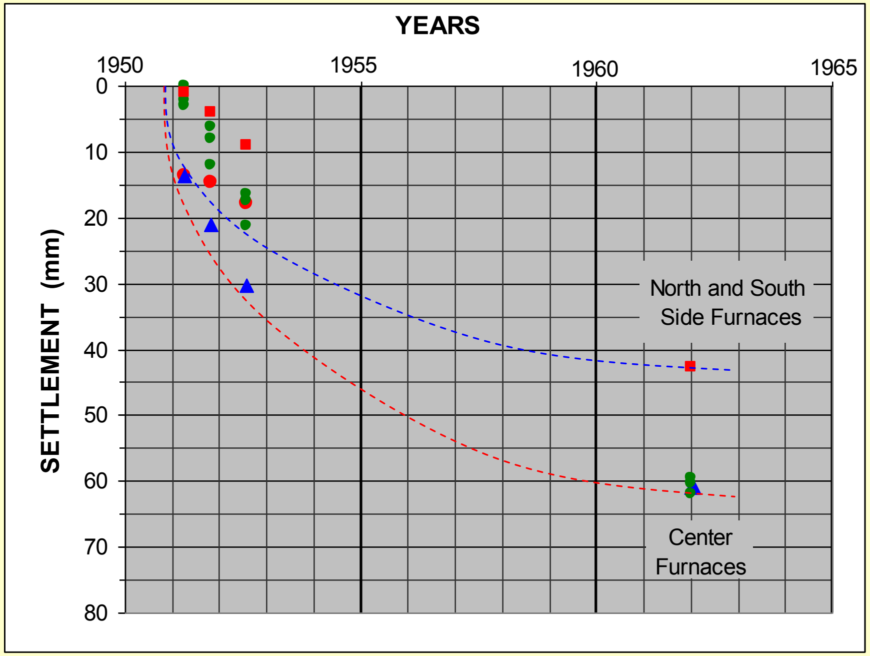


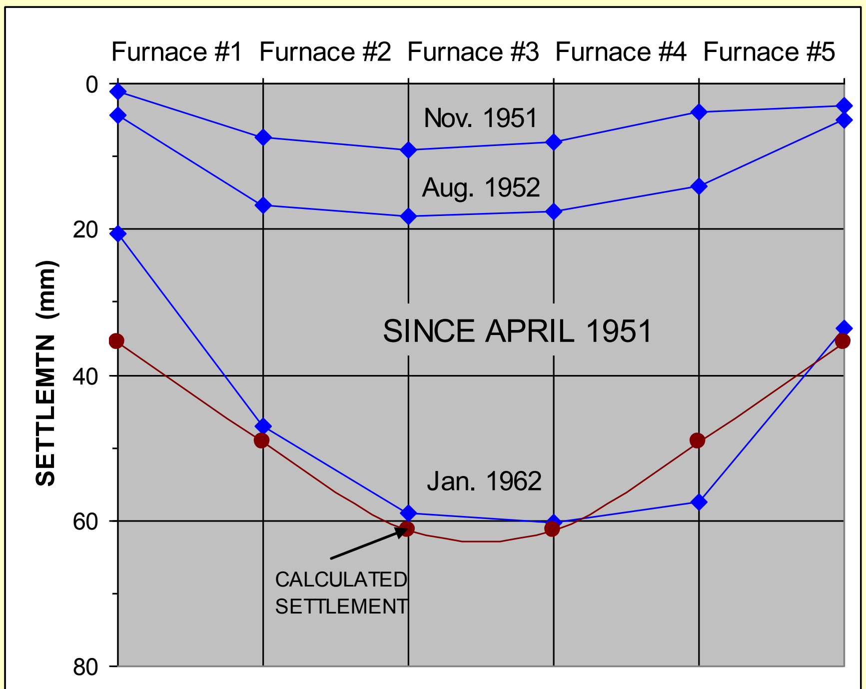


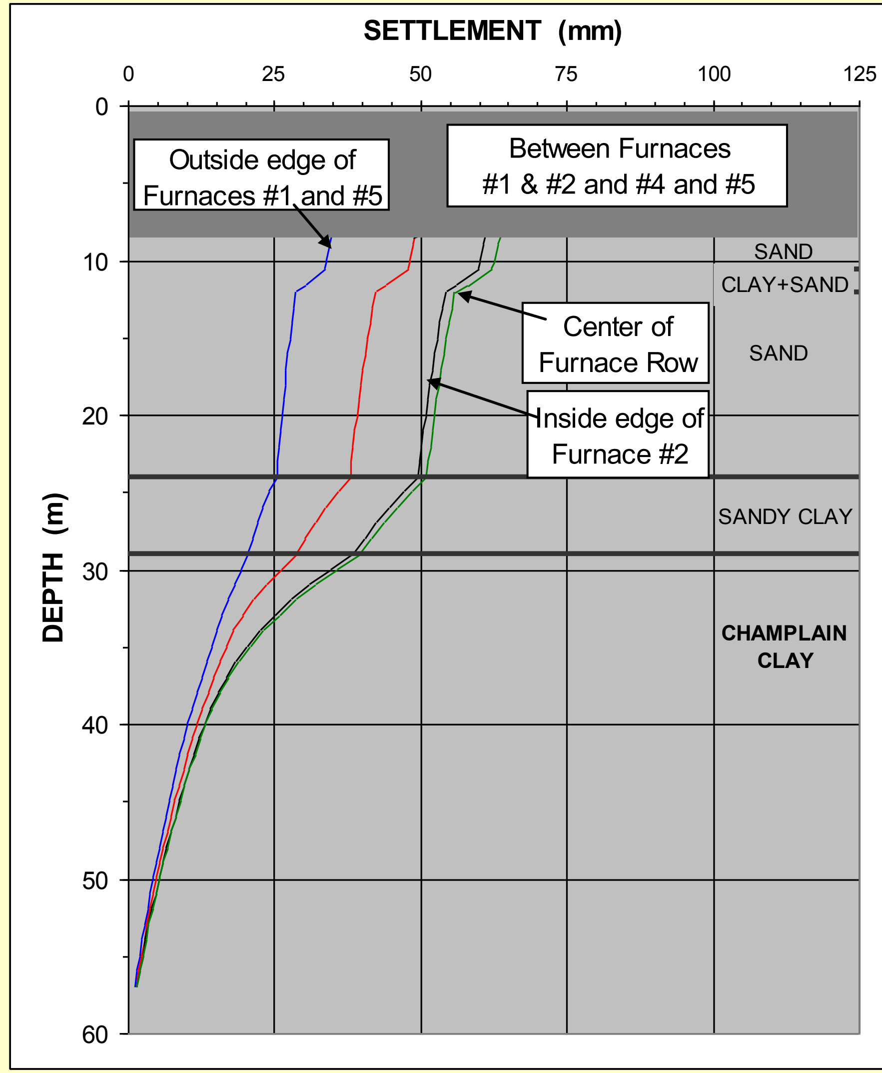
Static loading test used to predict settlement of the five furnaces: 10 mm



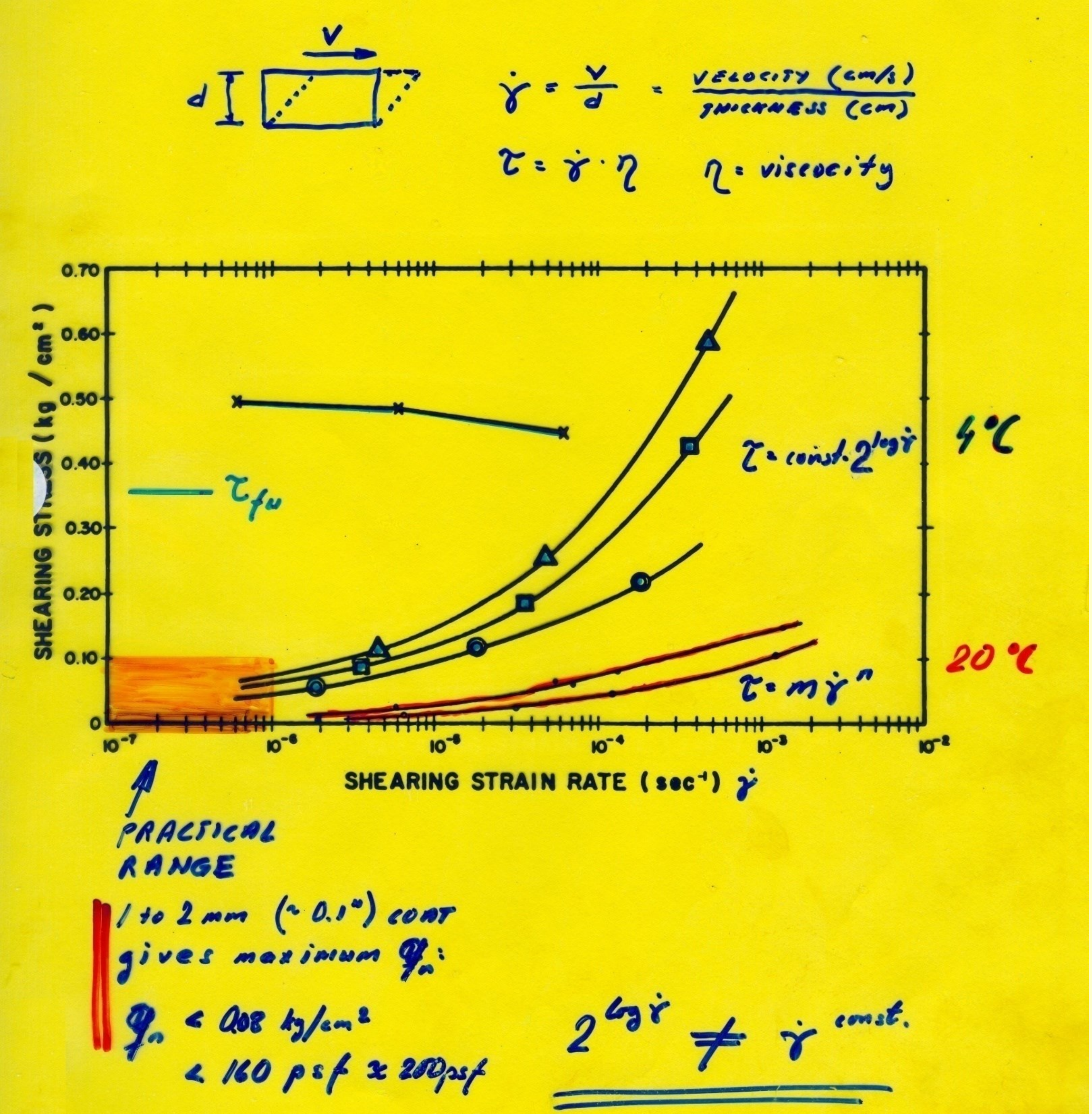



YEARS
1950
1955
1960
1965
SETTLEMENT (mm)
North and South
Side Furnaces
Center
Furnaces


Furnace #1 Furnace #2 Furnace #3 Furnace #4 Furnace #5
SETTLEMTN (mm)
Nov. 1951
Aug. 1952
SINCE APRIL 1951
Jan. 1962
CALCULATED
SETTLEMENT


SETTLEMENT (mm)
DEPTH (m)
Outside edge of
Furnaces #1 and #5
Between Furnaces
#1 & #2 and #4 and #5
Center of
Furnace Row
Inside edge of
Furnace #2
SAND
CLAY+SAND
SAND
SANDY CLAY
CHAMPLAIN
CLAY

A few words on Bitumen Coating



Laboratory tests on bitumen coats at different rates of shear

Piled foundations in current codes

The Canadian Building Code and Highway Design Code (**1992**), as well as the Hong Kong Code (Geo Guide 2006) apply the Unified Design method. That is, the drag load is only of concern for the structural strength of the pile. Indeed, the Canadian Highway Code even states that for piles with an aspect ratio (embedment depth over diameter, D/b), smaller than 80, the design does not have to check for drag load. However, the design must always check for downdrag.

The Manual of US Corps of Engineers indicate a similar approach (but less explicit), stating that the drag load constitutes a settlement problem (as opposed to a bearing capacity problem).

The ASCE “Practice for the Design and Installation on Pile Foundations (2007)” includes the following definitions:

DOWNDRAG: *The settlement due to the pile being dragged down by the settling of surrounding soil*;

DRAG LOAD: *Load imposed on the pile by the surrounding soil as it tends to move downward relative to the pile shaft, due to soil consolidation, surcharges, or other causes.*

and the following statement:
In some cases, the allowable load, as well as the pile embedment depth, is governed by concerns for settlement and downdrag, and by concern for structural strength for dead load plus drag load, rather than by bearing capacity.

The FHWA has produced one of the most extensive recent guidelines document. The full reference is: *Report No. FHWA-NHI-05-042, Design and Construction of Driven Pile Foundations - Volume I and II. National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., April 2006. 1,450 pages.*

The current issue, drag load and downdrag, is covered in about 20 of the total number of pages. in all essential parts, the FHWA document adheres to the principles of the Unified Design Method.

The FHWA document indicates the following criteria for identifying a drag load and/or downdrag problem. If any one of these criteria is met, drag load or downdrag shall be considered in the design.

The criteria are:

1. The settlement of the ground surface (after the piles are installed) will be larger than 10 mm (0.4 in).
2. The piles will be longer than 25 m (82 ft).
3. The compressible soil layer is thicker than 10 m (33 ft).
4. The water table will be lowered more than 4 m (13 ft).
5. The height of the embankment to be placed on the ground surface exceeds 2 m (6.5 ft).

Note however, that negative skin friction is usually fully mobilized at a movement between the pile and the soil of about 1 mm, not 10 mm!

The trend is toward Load and Resistance Factor Design (LRFD). The Canadian Highway Code has been based on LRFD for about 20 years. With regard to the drag load and downdrag issue, the Canadian Code follows the unified design method.

Since 1995, the Australian Piling Standard is also a Limit States Design Code (LRFD), and, like the Canadian Code, the recommendation for the design of piled foundations is according to the Unified Method, as quoted in the following.

The Australian Piling Standard, AS 2159—1995

3.3.2 Load combinations for strength design The load combinations for strength design shall be as follows:

(a) The design load for ultimate strength design of piles shall be the combination of factored loads which produces the most adverse effect on the pile in accordance with AS 1170.1

(b) If there are loads induced by soil movement (see Clause 3.3.1.2), they shall be computed as follows:

(i) *Design* ***structural strength*** (see Clause 4.3.5)—determined as follows:
(A) 1.2 F_{nf} — negative friction loads (*i.e., drag load*).
(B) 1.5 F_{es} — compressive and tensile loads
(C) 1.5 F_{em}— bending moments, shear forces, and axial loads.

(ii) *Design* ***geotechnical strength***—loads induced by soil movement shall not be taken into account.

The Australian Piling Standard, AS 2159—1995

4.3.5 Negative friction In the absence of other information, the **geotechnical** strength in compression or uplift shall be assumed to be unaffected by negative friction and shall be computed as set out in Clauses 4.3.1 and 4.3.2 for a single pile, and Clause 4.3.3 for a pile group.

The additional axial forces induced in a pile by negative friction shall be considered in the **structural design** of the pile.

4.5.3 Settlement Consideration shall be given to the settlement of both a pile and a pile group resulting from effects caused by settlement of the surrounding ground. NOTE: In the absence of an analysis in which pile-soil interaction is allowed for, the settlement of a pile or pile group subjected to negative friction may be approximated as the greater of the following:

(**a**) The settlement of the ground at the 'neutral plane' in the ground, that is the depth at which the shaft friction on the pile changes from negative (downward) to positive (upward). Applied compressive loading tends to raise the 'neutral plane' and increase the settlement of the pile or pile group.

(**b**) The sum of the following three components:

(i) the compression of the pile shaft due to the design action;

(ii) the compression of the pile shaft due to the computed forces arising from negative friction;

(iii) the settlement of the portion of the pile shaft in the 'stable' soil (the part of the soil profile not subjected to movement) under the sum of the design action and the maximum computed force in the pile arising from negative friction.

As many other codes and standards, the Australian Standard can go overboard with some details

TABLE 4.1
RANGE OF VALUES FOR GEOTECHNICAL STRENGTH REDUCTION FACTOR

Method of assessment of ultimate geotechnical strength

Static load testing to failure

Static proof (not to failure) load testing

Dynamic load testing to failure supported by signal matching

Dynamic load testing to failure not supported by signal matching

Dynamic proof (not to failure) load testing supported by signal matching

Dynamic proof (not to failure) load testing not supported by signal matching (**!**)

Static analysis using CPT data

Static analysis using SPT data in cohesionless soils (**!**)

Static analysis using laboratory data for cohesive soils

Dynamic analysis using wave equation method (**!**)

Dynamic analysis using driving formulae for piles in rock (**!**)

Dynamic analysis using driving formulae for piles in sand (**!**)

Dynamic analysis using driving formulae for piles in clay (**!**)

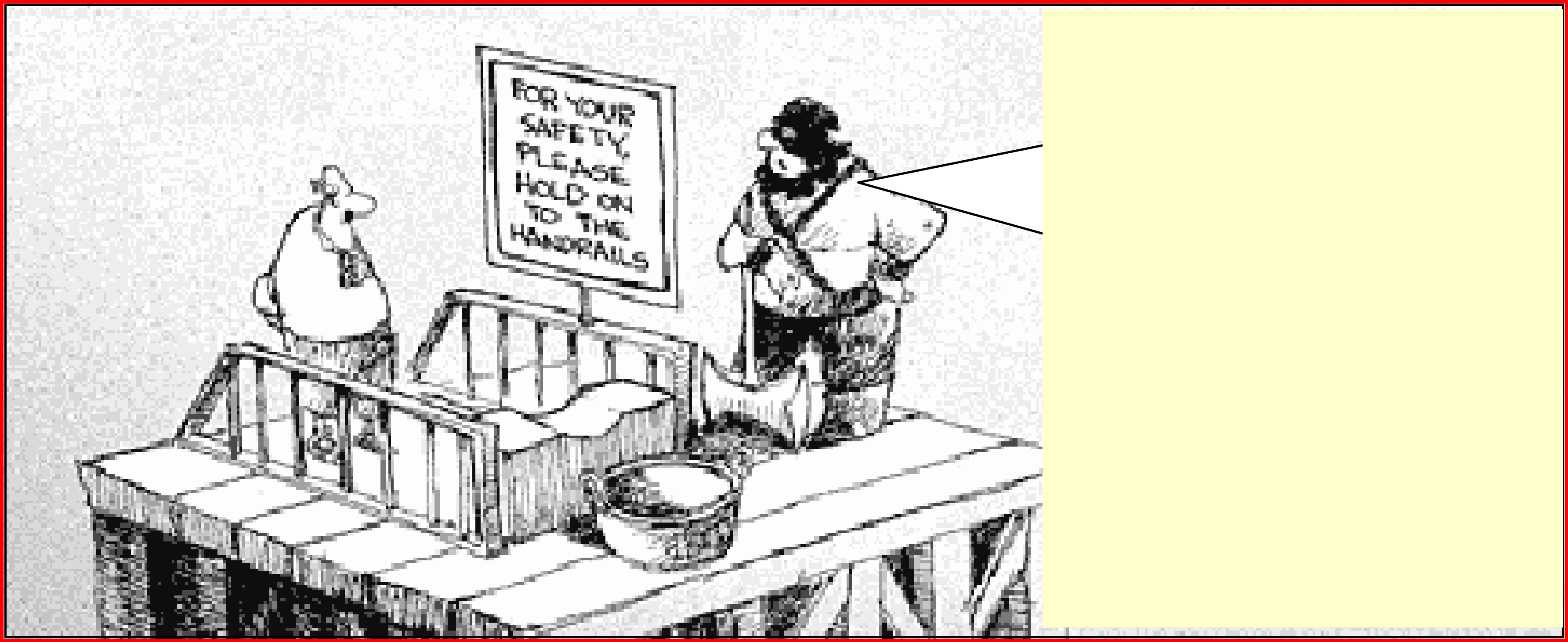
Measurement during installation of proprietary displacement piles,
using well established in-house formulae

As many other codes and standards, the Australian Standard can go overboard with some details

TABLE 4.1
RANGE OF VALUES FOR GEOTECHNICAL STRENGTH REDUCTION FACTOR

Method of assessment of ultimate geotechnical strength	Range of values
Static load testing to failure	0.70–**0.90**
Static proof (not to failure) load testing	0.70–0.90
Dynamic load testing to failure supported by signal matching	0.65–0.85
Dynamic load testing to failure not supported by signal matching	0.50–0.70
Dynamic proof (not to failure) load testing supported by signal matching	0.65–**0.85**
Dynamic proof (not to failure) load testing not supported by signal matching (**!**)	0.50–0.70
Static analysis using CPT data	0.45–0.65
Static analysis using SPT data in cohesionless soils (**!**)	0.40–0.55
Static analysis using laboratory data for cohesive soils	0.45–0.55
Dynamic analysis using wave equation method (**!**)	0.45–0.55
Dynamic analysis using driving formulae for piles in rock (**!**)	0.50–0.65
Dynamic analysis using driving formulae for piles in sand (**!**)	0.45–0.55
Dynamic analysis using driving formulae for piles in clay (**!**)	
Measurement during installation of proprietary displacement piles, using well established in-house formulae	0.50–0.65

Factors of safety and LRFD



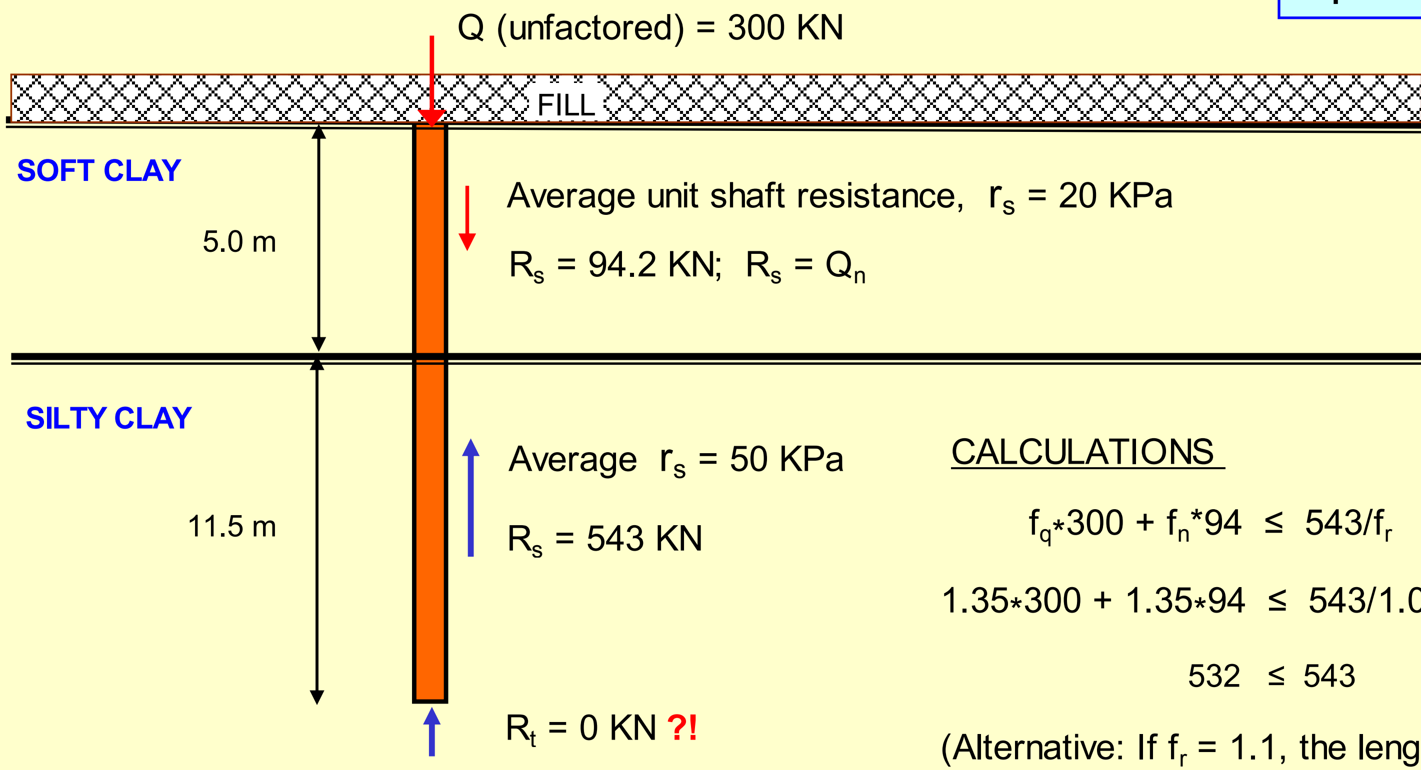
The Euro Code

The European Community has recently completed EuroCode 7, which is supposed to be adopted by all member states. The EuroCode treats the drag load as a load acting similarly to the load from the structure, and requires it to be added to that load (or subtracted from the pile capacity). Moreover, the shaft resistance in the soil layer that contributes to the drag load is disregarded when determining the pile resistance. That is, when a capacity has been determined in a static loading test to, say, 1,000 and the drag load is expected to be, say, 400, the usable resistance is 1,000 – 2*400 = 200! After applying the resistance factor, what is left? What "saves" the economy of some designs is that the EuroCode clauses advocate that the designer maintain the faithful approach that "*the drag load cannot really be that large, can it, please?*" to determining the magnitude of the drag load. Incredibly, the EuroCode says little on how to calculate settlement of piled foundations and nothing is stated about downdrag!

Unfortunately, the recently issued 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)

and AASHTO Specs example



CALCULATIONS

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

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

$$532 \leq 543$$

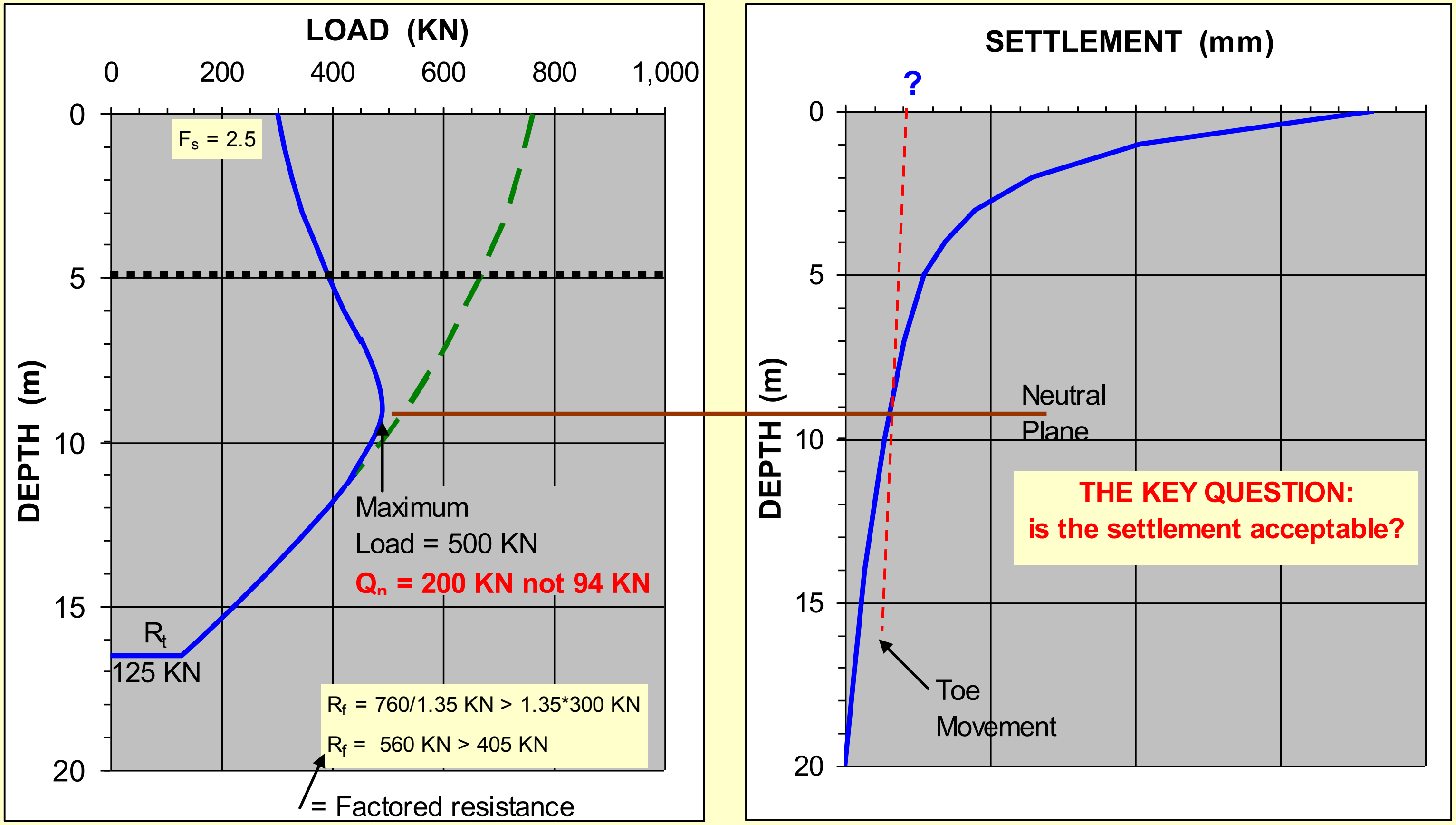
(Alternative: If f_r = 1.1, the length in the silty clay becomes 12.4 m)

"*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 **in both layers** with groundwater table at ground surface, and a fill stress of 30 KPa.

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

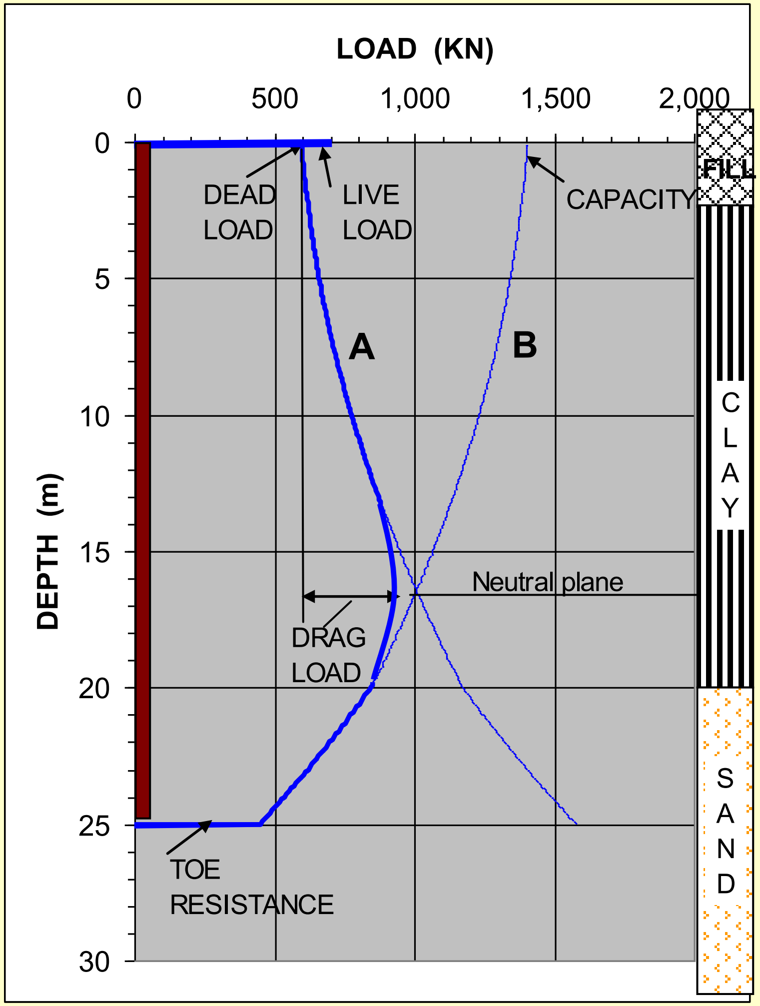
Analysis using the same numerical values for the pile shaft, but including the benefit of a small toe resistance



If the settlement is acceptable, there is 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?

Example from an actual project somewhere in Europe

A 300 mm diameter pile installed to a depth of 25 m through a surficial 2 m thick fill placed on a 20 m thick layer of soft clay deposited on a thick sand layer.

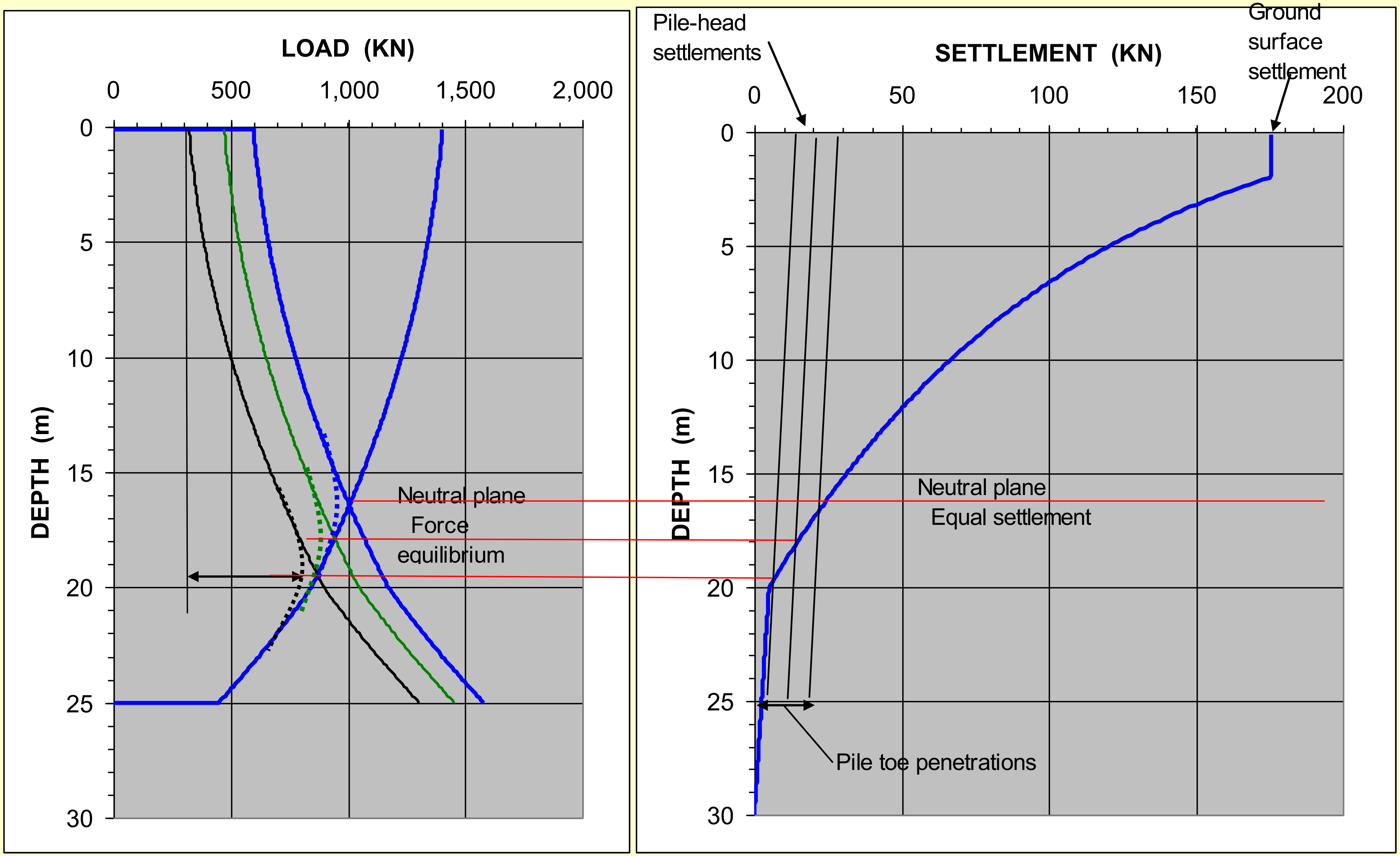


A static loading test has been performed and the evaluation of the test data has established that the pile capacity is 1,400 KN. Applying a factor of safety of 2.0 results in an allowable load of 700 KN (dead load 600 KN and live load 100 KN). The drag load is 300 KN.

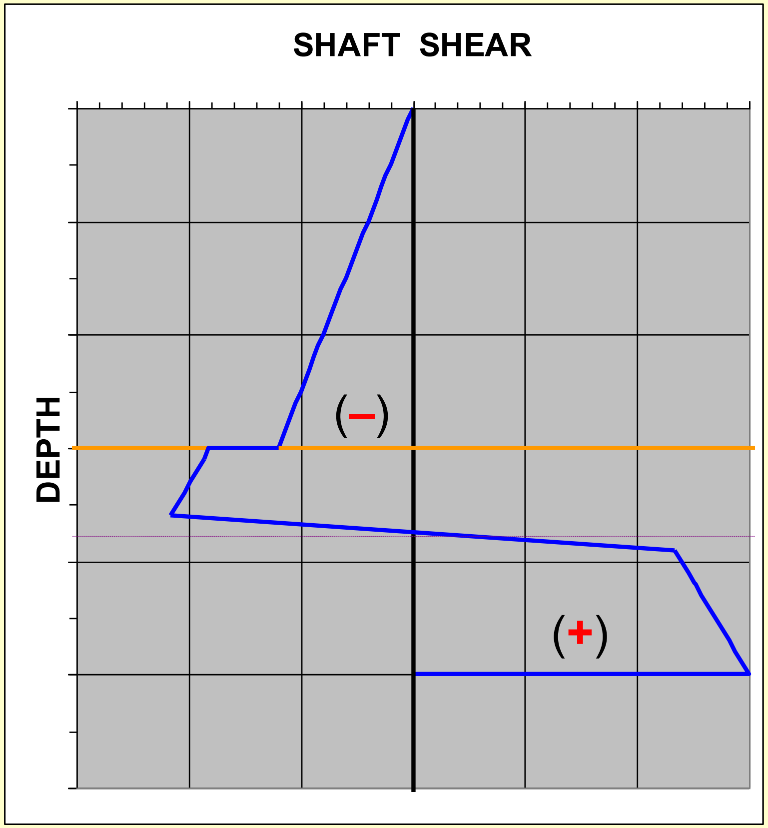
The designer insisted on subtracting the drag load from the capacity (considered available only from below the neutral plane) before determining the factored resistance (then = 900 KN). The "action" load was considered to be the sum of dead load, live load, and drag load, which sum already before multiplication by the load factor was larger than the factored resistance! The test results were stated to show that the 1,400 KN capacity pile piles was inadequate to support the 700 KN load. The designer required longer piles and a considerably increased number of piles.

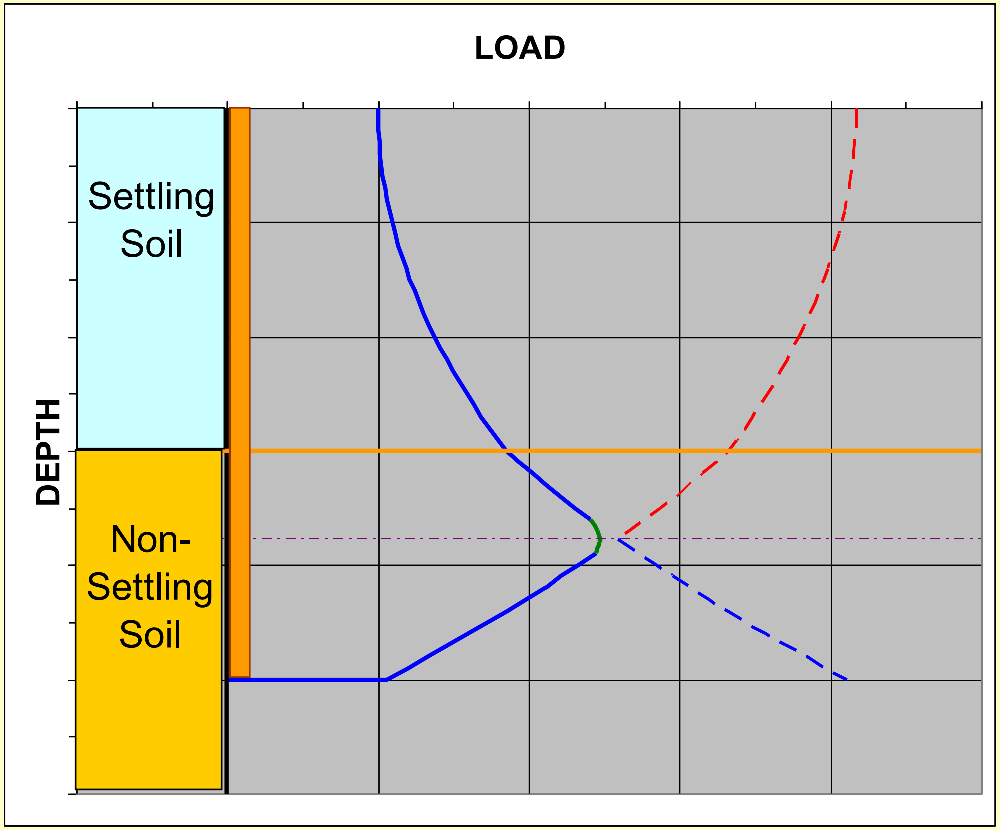
!! $$$!!

Graphic Illustration of the Case



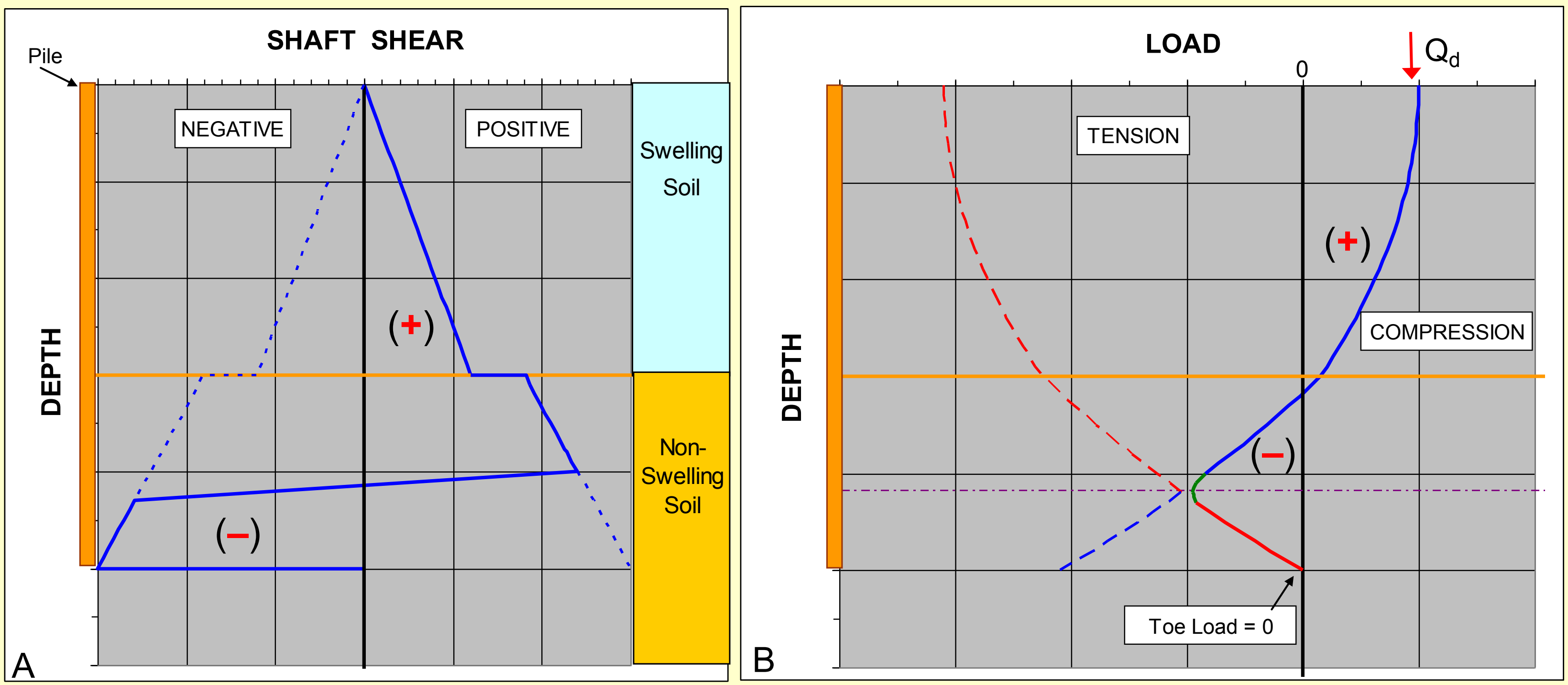
A repeat: Distribution of unit shaft shear and of load and resistance





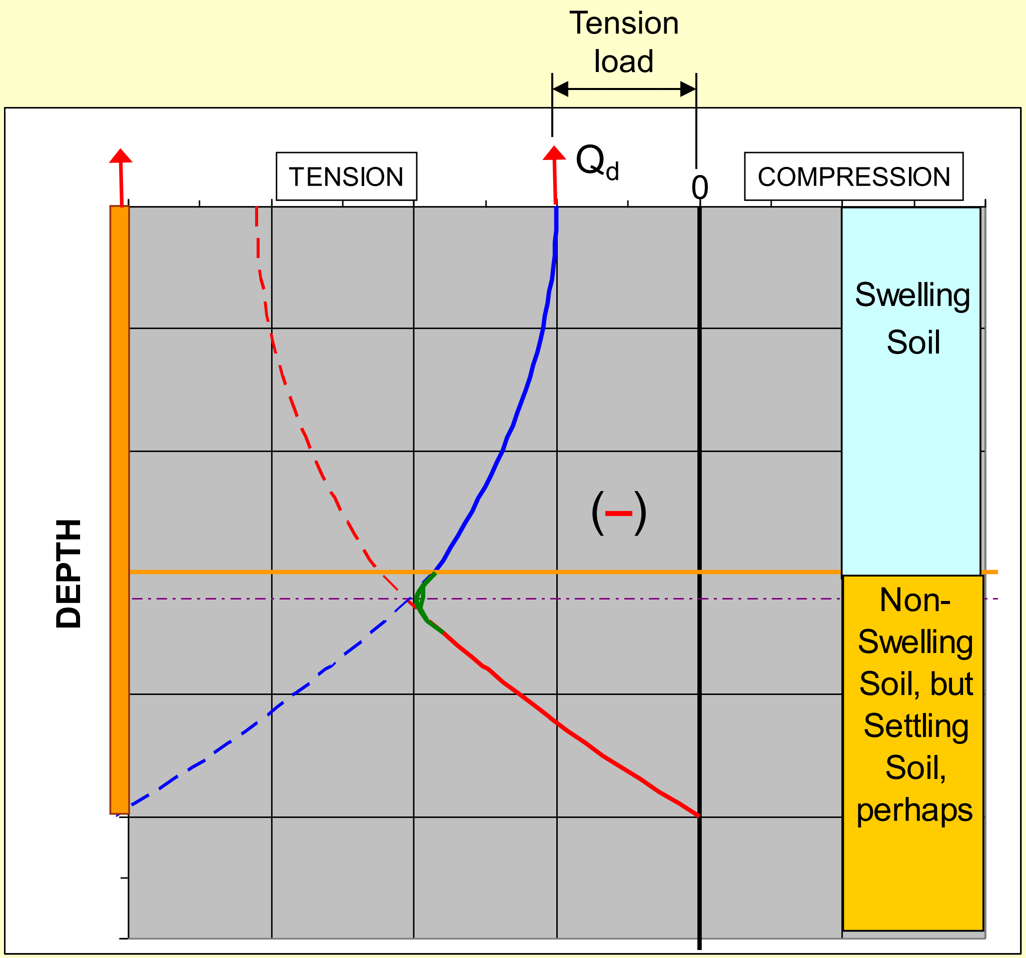
How would the distributions look for a pile in a swelling soil?

The shear force along the pile in a swelling soil is the opposite to that in settling soil, of course — "positive skin friction" as opposed to "negative skin friction". But the same analysis method applies.



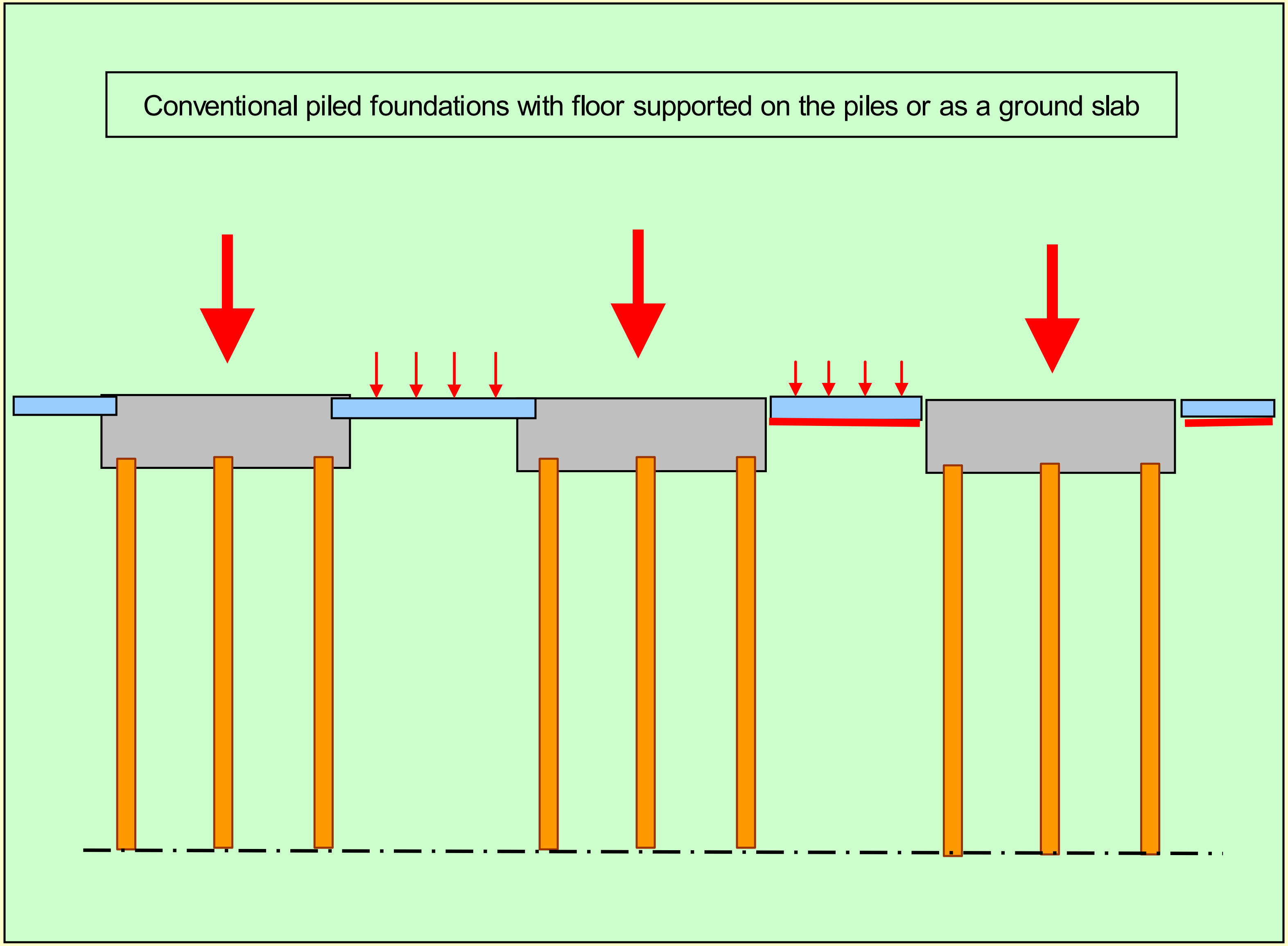
Distributions of unit shaft shear and load for a pile in swelling soil

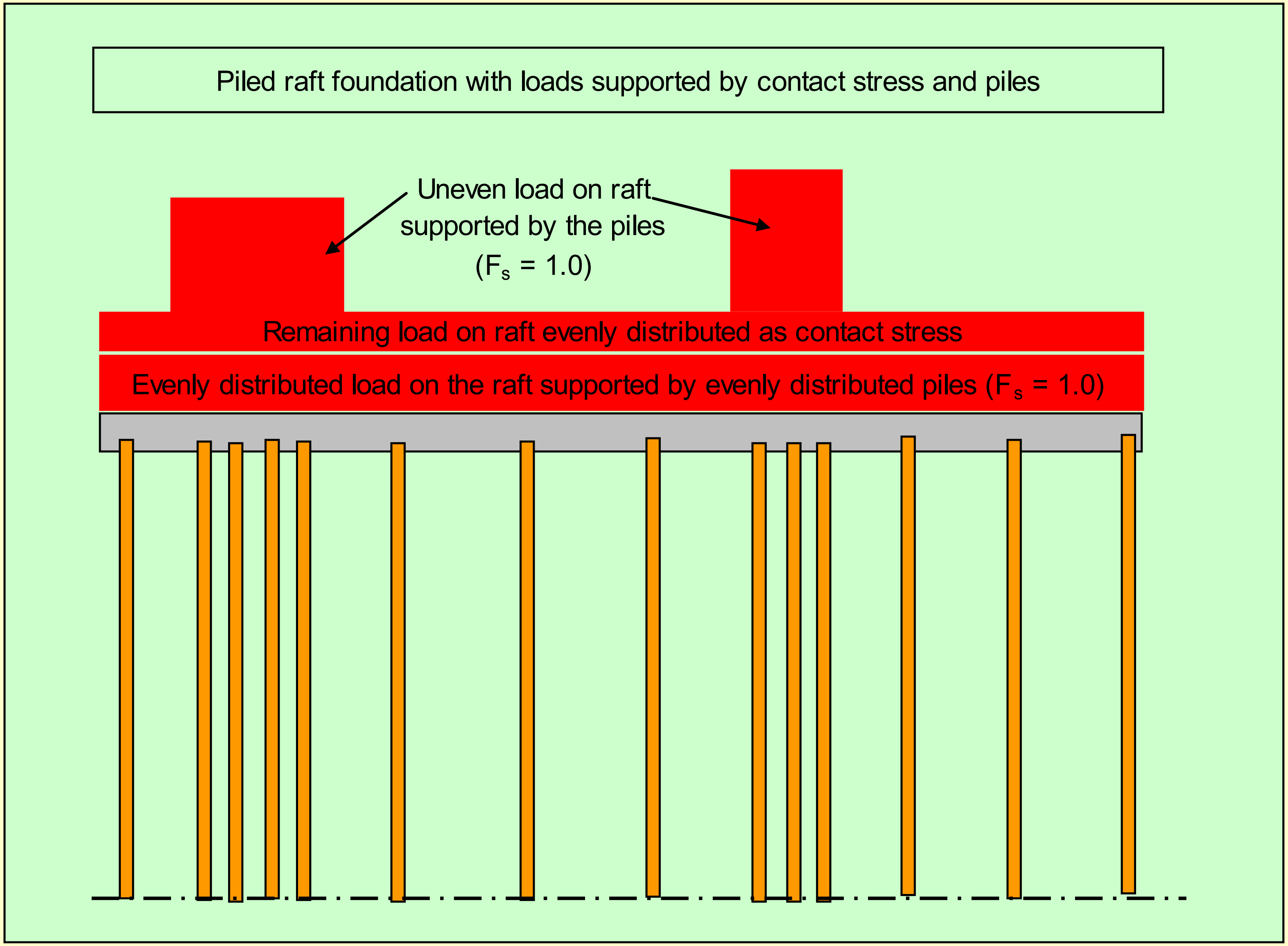
Pile is loaded in uplift (tension)



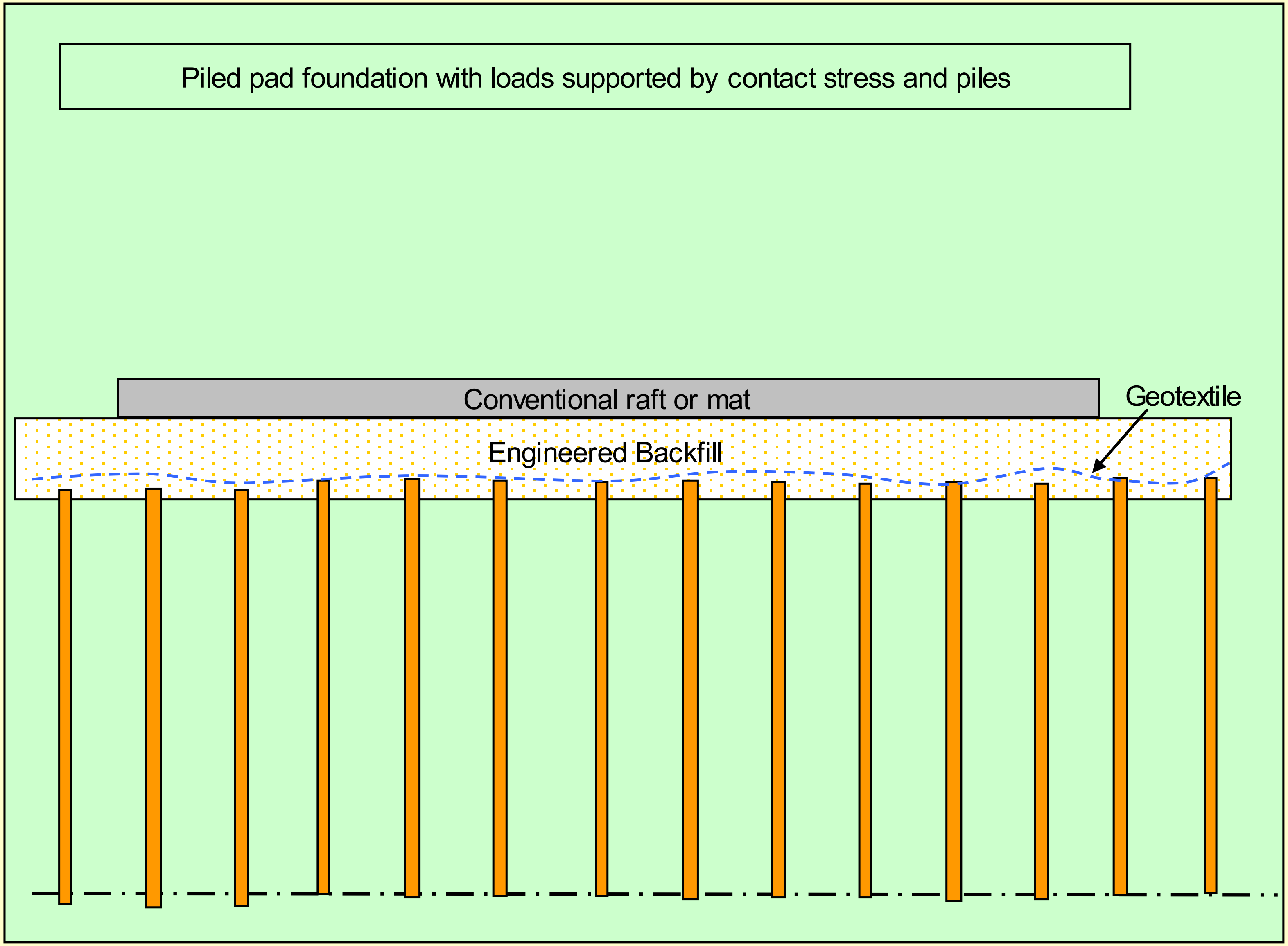
So, what does it mean that a pile loaded in tension in swelling soil has a neutral plane in settling soil below the swelling soil?

Piled Raft and Piled Pad Foundations



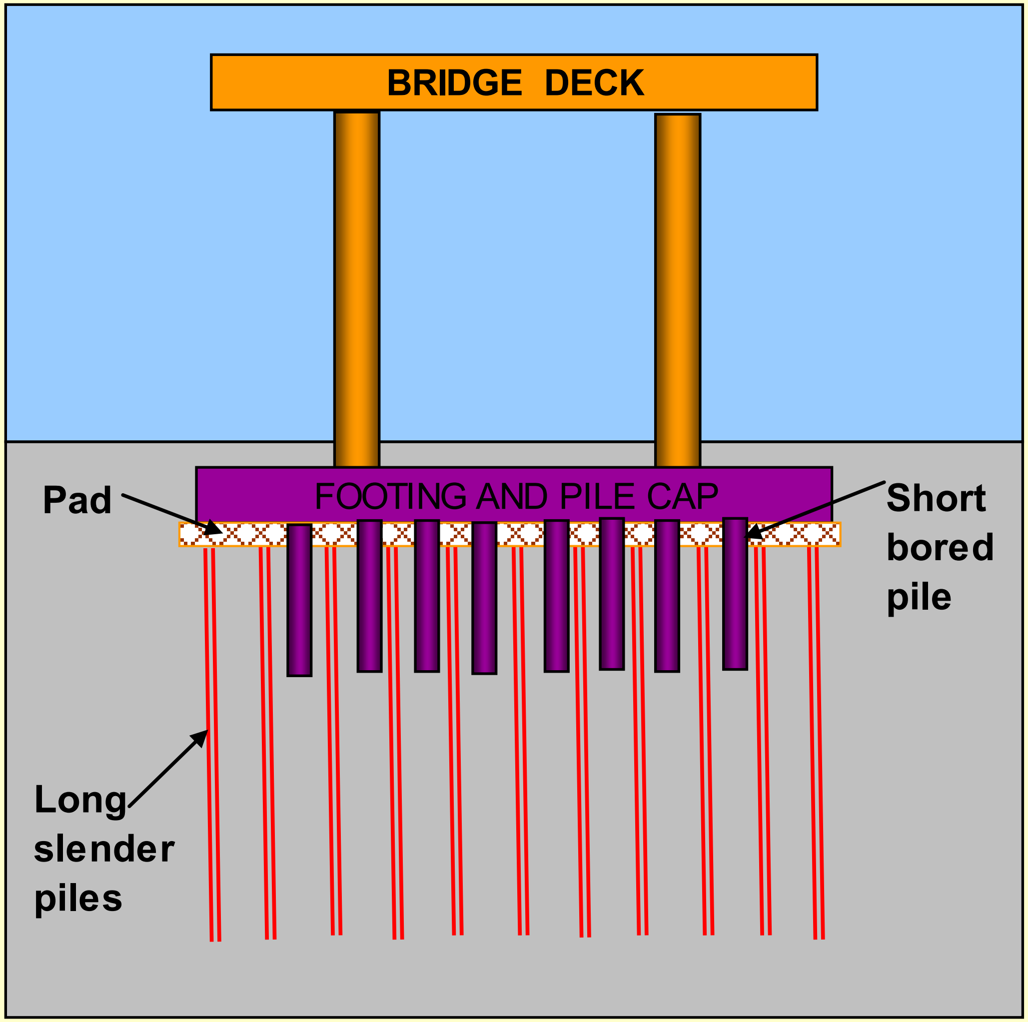


Piled raft foundation with loads supported by contact stress and piles
Uneven load on raft supported by the piles (F_s = 1.0)
Remaining load on raft evenly distributed as contact stress
Evenly distributed load on the raft supported by evenly distributed piles (F_s = 1.0)


Piled pad foundation with loads supported by contact stress and piles
Conventional raft or mat
Geotextile
Engineered Backfill

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 2008), illustrated below.

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



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

and

driven (300 mm; 30 m) piled-pad piles.

over an about 100 m thick deposit of soft compressible clay

Axial Design for Seismic Condition

Liquefaction (Adapazari, Turkey)

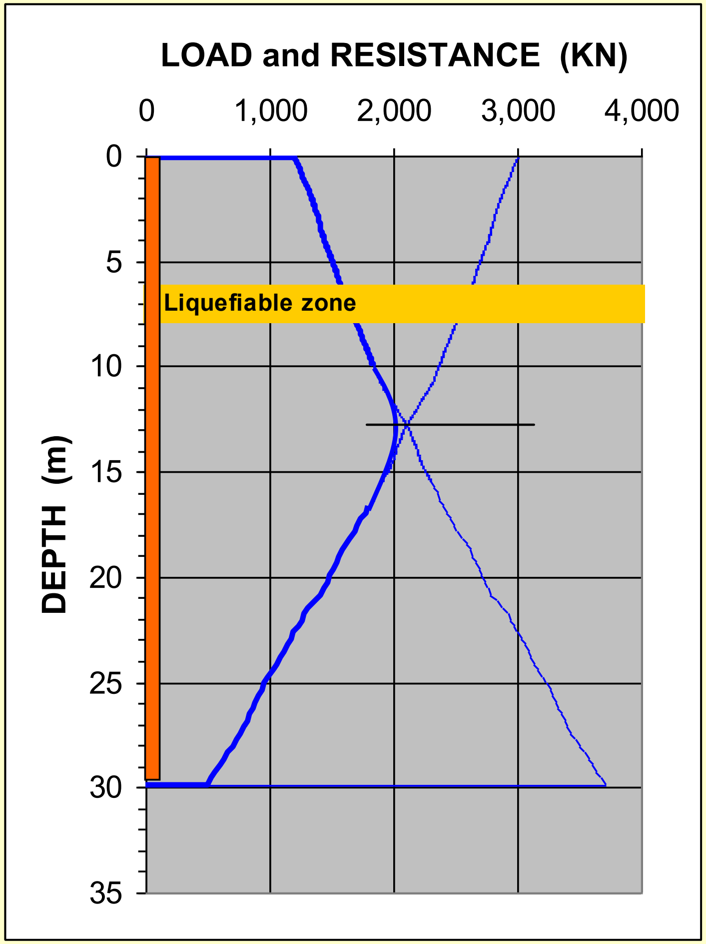
Photo courtesy of Noel J. Gardner, Ottawa

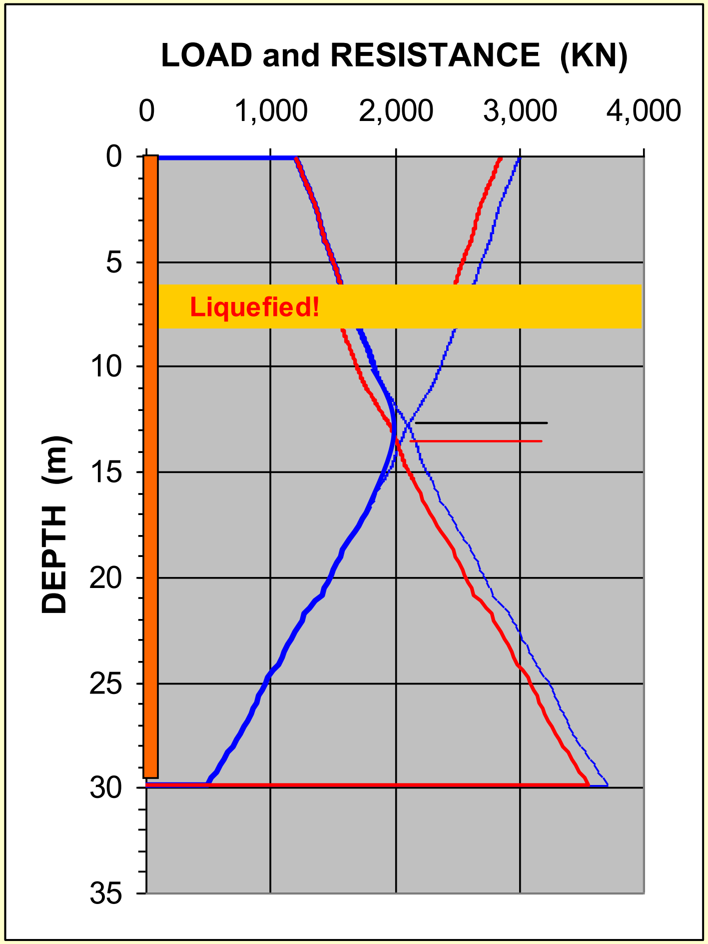
Liquefaction (Adapazari, Turkey)

Photo courtesy of Noel J. Gardner, Ottawa

The Unified Method Applied to Seismic (Liquefaction) Design

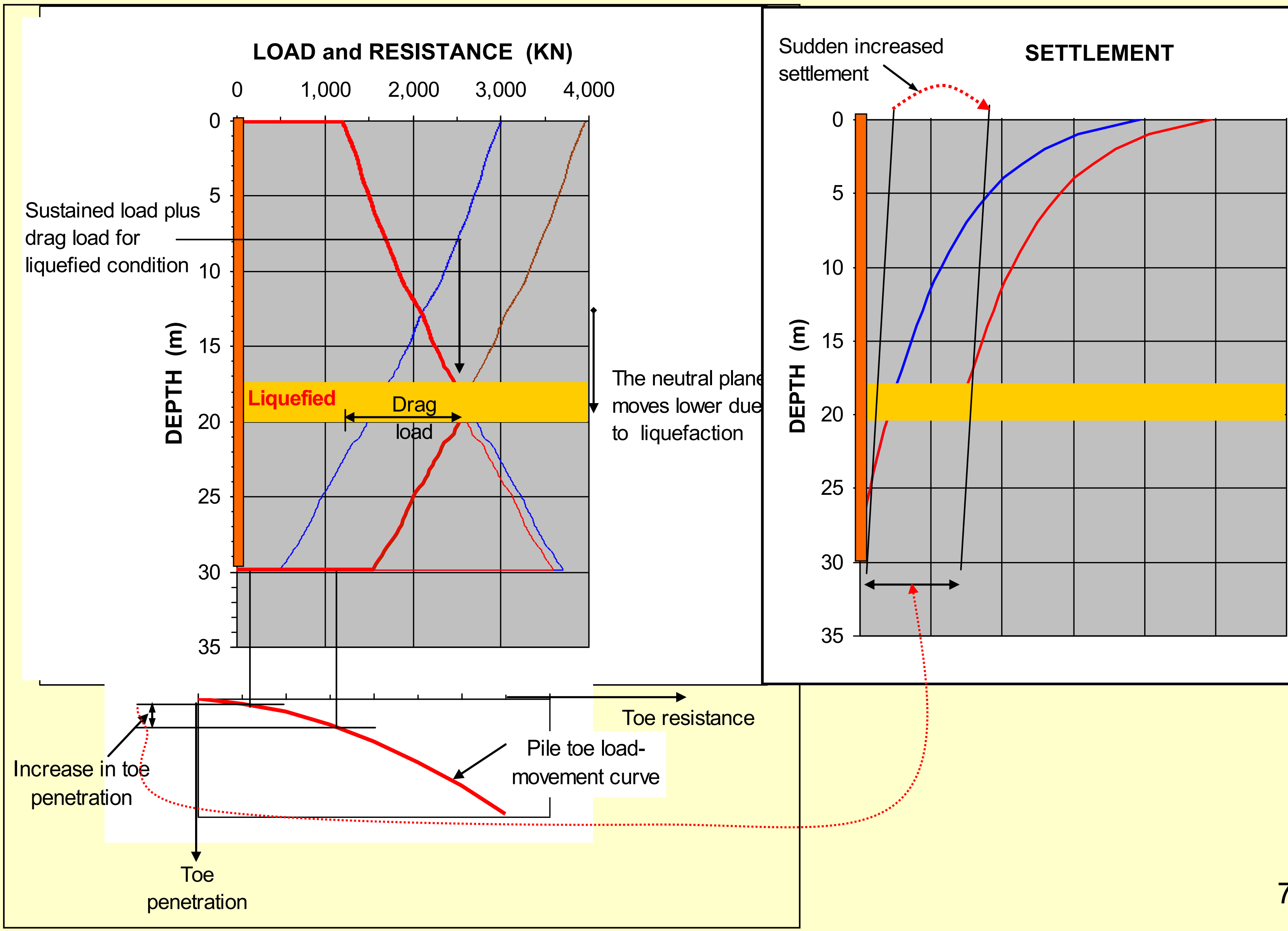
(Fellenius and Siegel 2008)





Liquefaction occurring above the neutral plane is of no practical consequence for the piles.

What about liquefaction occurring below the neutral plane?



Frequently, a design may require full-scale testing. (Note, a so-called routine static loading test with only applying load to the pile head is mostly a waste of money). If testing is necessary, then, the test should have some instrumentation to determine the load movement response of the pile and be properly planned and executed. An O-cell test is an invaluable tool for the designer at this stage.

However, if the soils are expected to settle, then, it is important that design is such that the neutral plane lies below the settling soil — is located in the non-settling layers. Then, the piled foundation will not settle. For piles shorter than about 30m (the 1992 Canadian Highway Bridge Design Code states that for piles shorter than 80b, where b is the pile diameter), the drag load is not going to exceed the safe structural load (stress) for most pile cases. Note, drag load is totally a matter for the structural strength of the pile. Drag load has no relevance for the pile bearing capacity and must not be combined with the load on the pile when determining the factor of safety on pile capacity.

Downdrag is the settlement of piles where the neutral plane lies in compressible layers that are settling. While drag load is of little concern for a pile foundation (provide the structural strength is sufficient), downdrag is not desirable. Downdrag is settlement and the “inverse” of drag load and the two definitions must be understood as separate.

The settlement analysis involves the loads from the structure, of course, but the important loads are the fills, footings, changes in pore pressure distribution, etc. around the pile(s).

The analysis requires applying a **load-movement relation** for the pile toe and the shaft resistance distribution in a trial-and-error approach to determine the mobilized toe resistance and the location of the neutral plane (they are mutually dependent).

The **settlement of the pile cap** is the soil settlement at the neutral plane plus the pile shortening for the combination of dead load on the pile cap and the drag load.

The Unified Design Method applies basic soil mechanic principles of effective stress while relying on soil parameters determined from well-analyzed tests on instrumented piles, realizing that movements and deformations are what govern the pile response to axial load, and understanding that foundations care about settlements, not about factors of safety on some capacity value.

Soil parameters should not be taken from a textbook or some published paper. If the parameters have to be assumed, then, use input of not just the most probable value, but also values representative for the upper and lower range of potential values.

If you are still unsure about not including the Drag Load in determining the allowable load from the bearing capacity, please recognize the fundamental principle of that no other loads than those present for the case of a factor of safety of unity (i.e., 1.0) can be included in a calculation of factor of safety as capacity divided by the applied loads. The drag load does not then exist, as it is not a load to be supported by the pile in contrast to the loads from the structure.

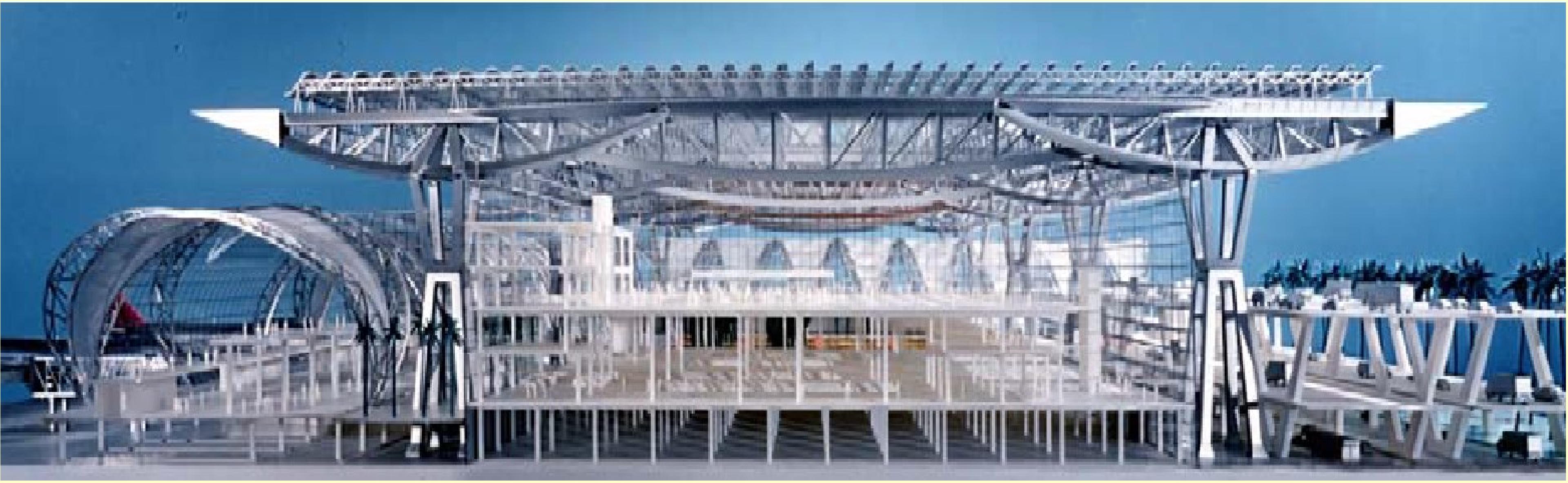
To rephrase and repeat:

The imperative requirement for the design approach of dividing capacity with a factor of safety is that only the loads present at a factor of safety of unity (1.0) can be included in a design analysis (then, using a more reasonable factor of safety, of course). Those loads are the dead and live loads. Drag load does not exist when F_s is 1.0 and should therefore not be included when F_s is, say, 2.0).

Design for drag load is akin to prestressed concrete where one must not apply a prestress that can risk overstressing the concrete (together with other stresses (axial and bending), which is a structural problem. When that prestressed "beam" is in the ground serving as a pile, structural strength is still an issue. The drag load is nothing but an add-on prestress load of a sort.

However, the geotechnical capacity is independent of any soil force acting on the "beam", as large prestress or small, "old" or "add-on", is irrelevant to the geotechnical capacity. The geotechnical issue is settlement, which again is independent of any kind of prestress present in the "beam", aqua "pile".

CASE HISTORY EXAMPLES



The New International Airport, Bangkok Thailand

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


THAILAND
(Bangkok) Krung Thep
Ko Si Chang
Ko Phai
Ko Khram Yai
Ko Samae San
Magan Islands
Auk Bok
Mali Kyun
Mali Nge
Thamihla Kyun
Kala Kyun
Grants Island
Mergui Islands
Sellore Island
Parker Island
Kanmaw Kyun
Eliza Island
Pawe Gyi Kyun
Ale-man Kyun
Kau-ye Kyun
Ko Thalu
Image © 2007 TerraMetrics
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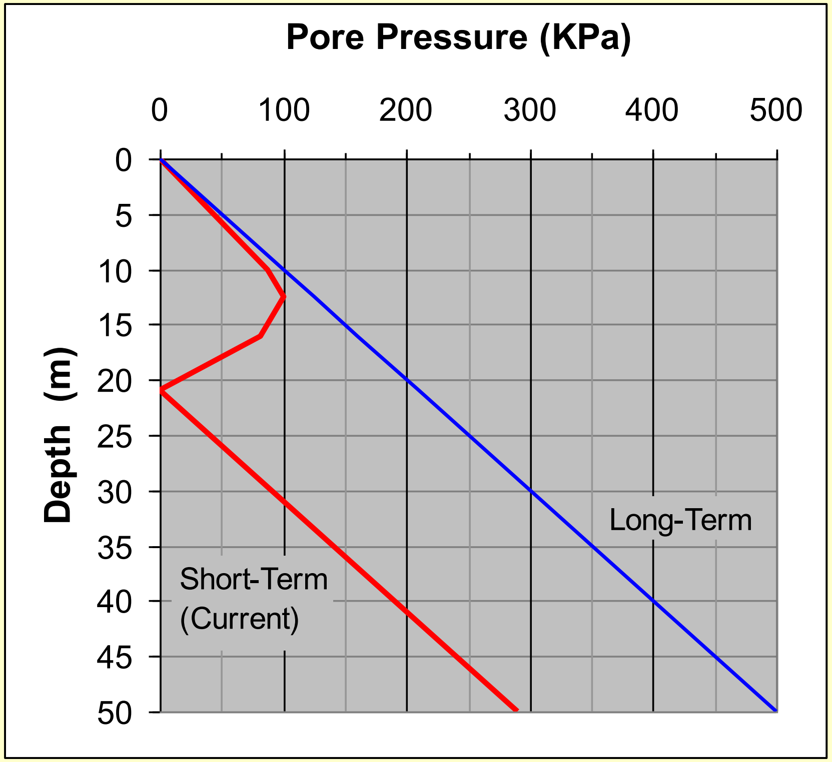
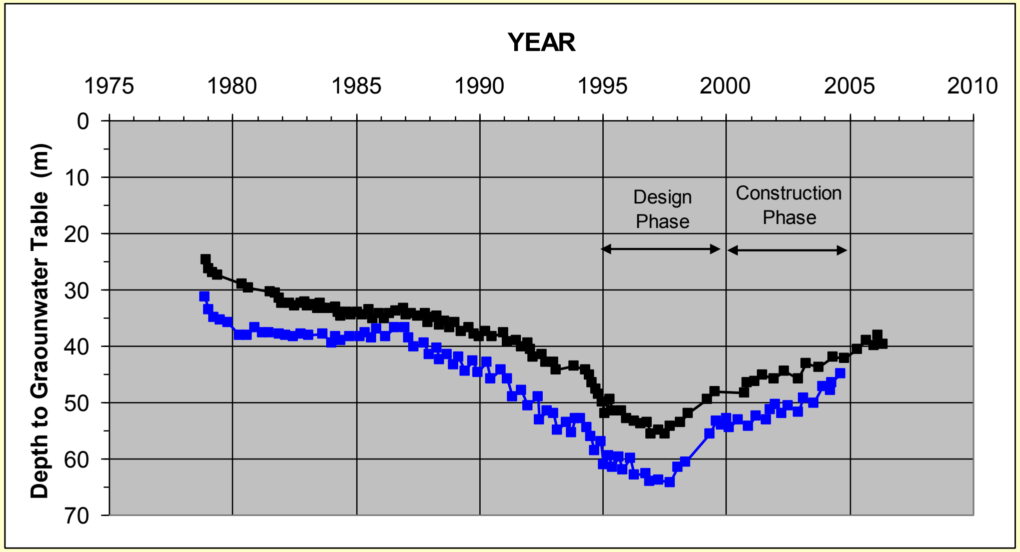
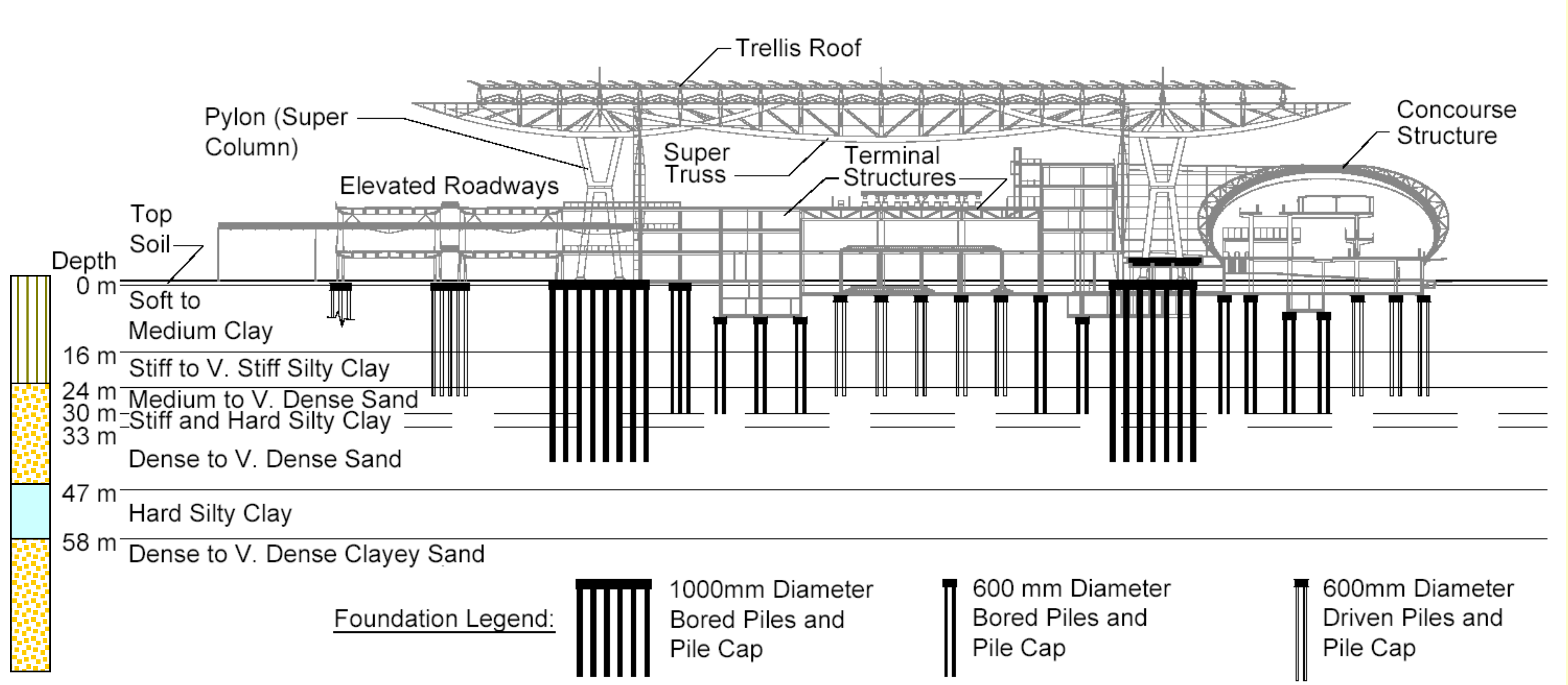
Current and Future Pore Pressure Distribution



Nearby Observations of Groundwater Table



Pumping (mining) of groundwater has reduced the pore pressures. At the start of the design process, pumping in the area was stopped.



Schematic foundation section and typical soil profile

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

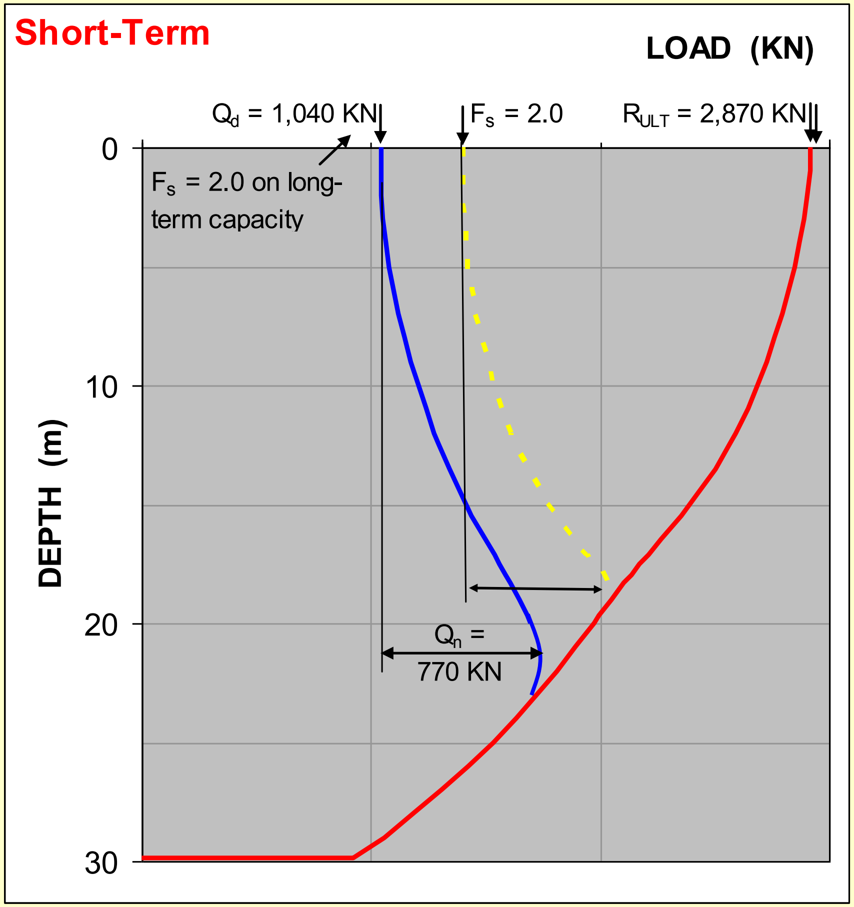
All foundations — the trellis roof, terminal buildings, concourse, walkways, etc. — are placed on piles. The stress-bulbs from the various foundations will overlap each other's areas resulting in a complicated settlement analysis.

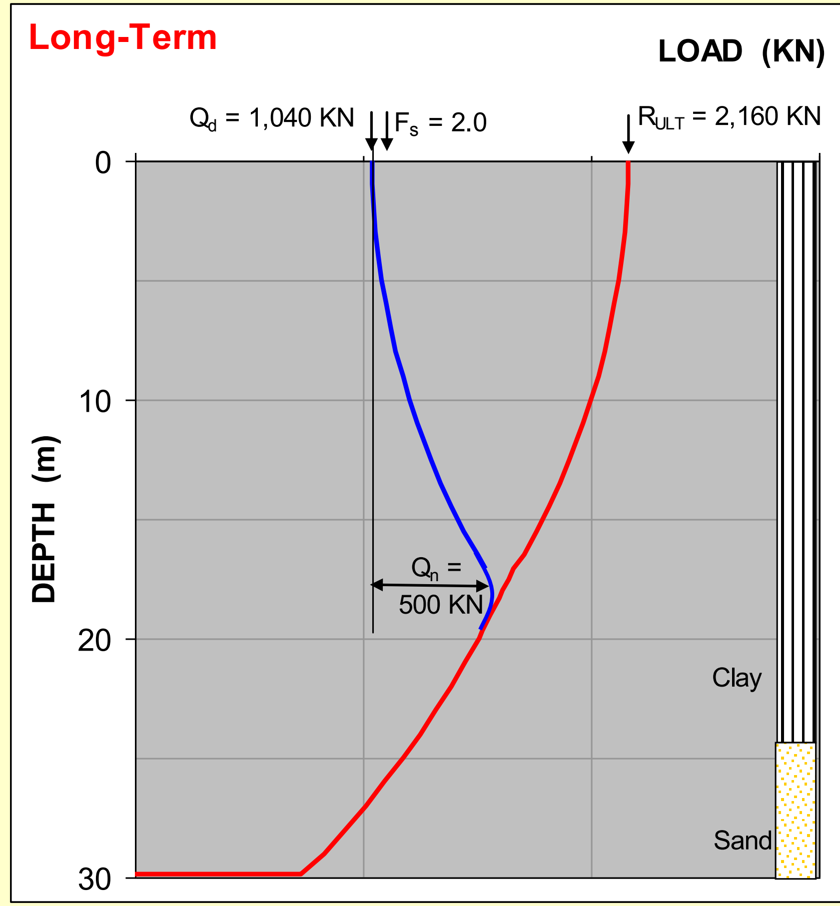
Several static loading tests on instrumented piles were performed to establish the load-transfer conditions at the site at the time of the testing, i.e., short-term conditions. Effective stress analysis of the test results for the current pore pressures established the coefficients applicable to the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

A total of **25,000+** piles were installed.

The design employed the unified pile design method.

Example of 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**

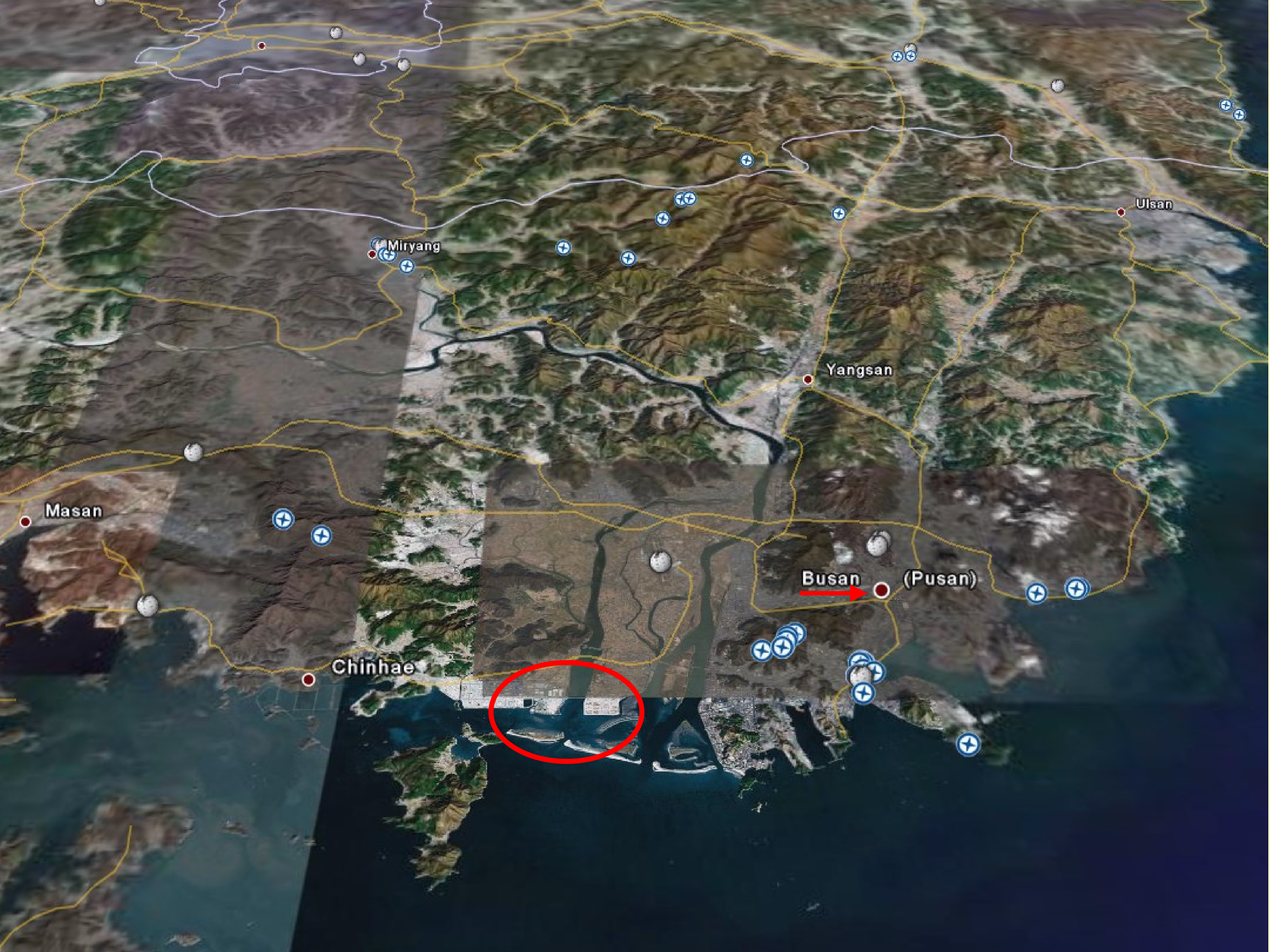
* * *

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
South Korea
Daejeon
Daegu
Kwangju
Busan
(Pusan)
Cheju do
Fukuoka
Hirado-shima
Kumamoto
Fukue-jima
Kyushu
Jiangsu
Yangzhou
Nanjing
Wuxi
Suzhou
Shanghai
Kagoshima


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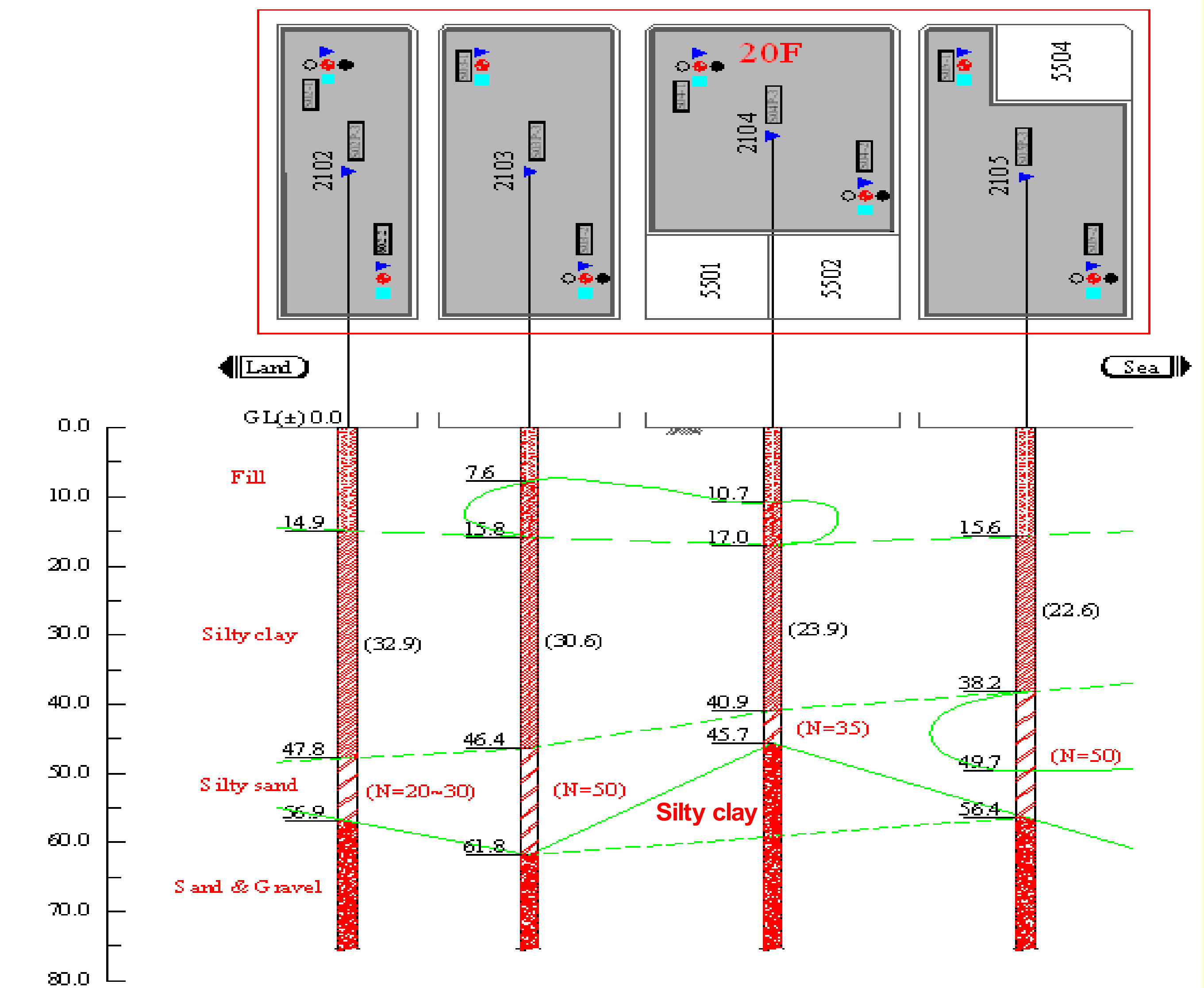
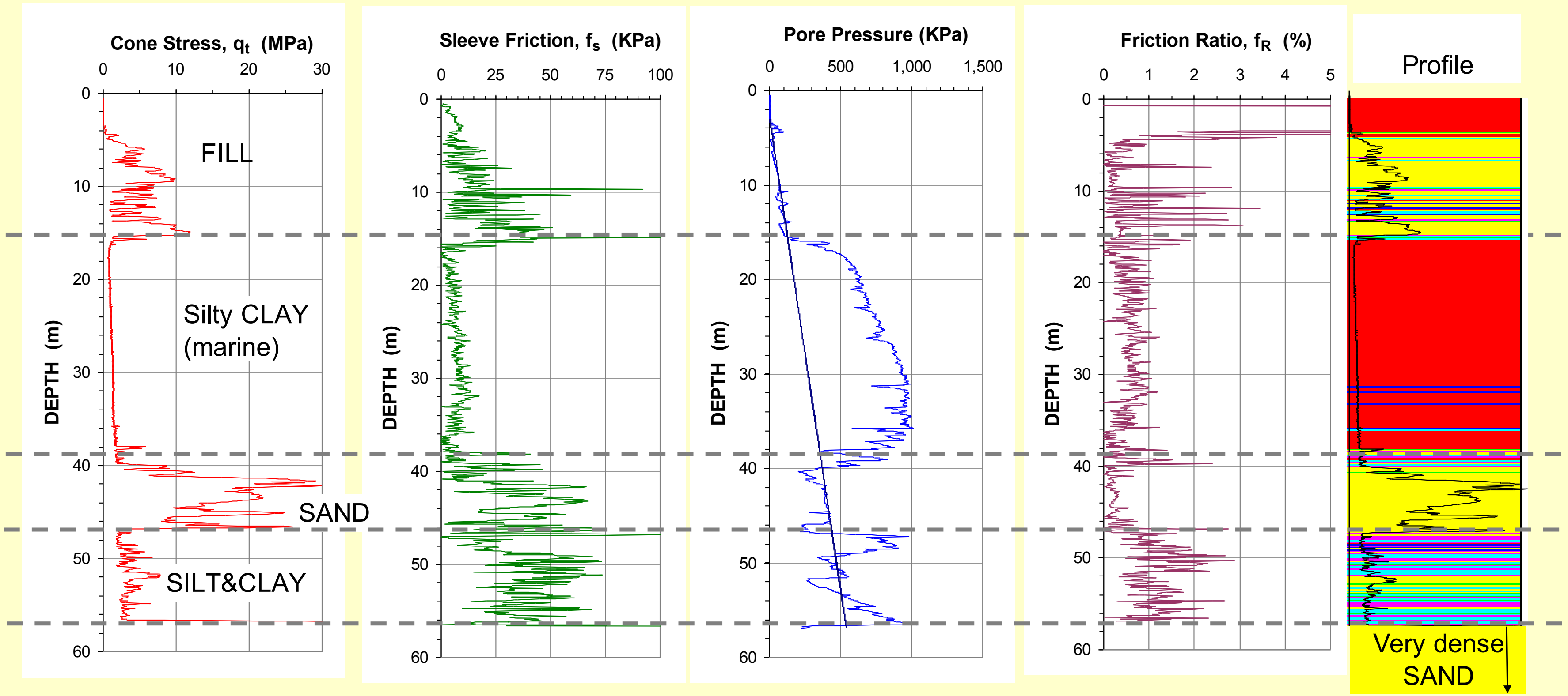
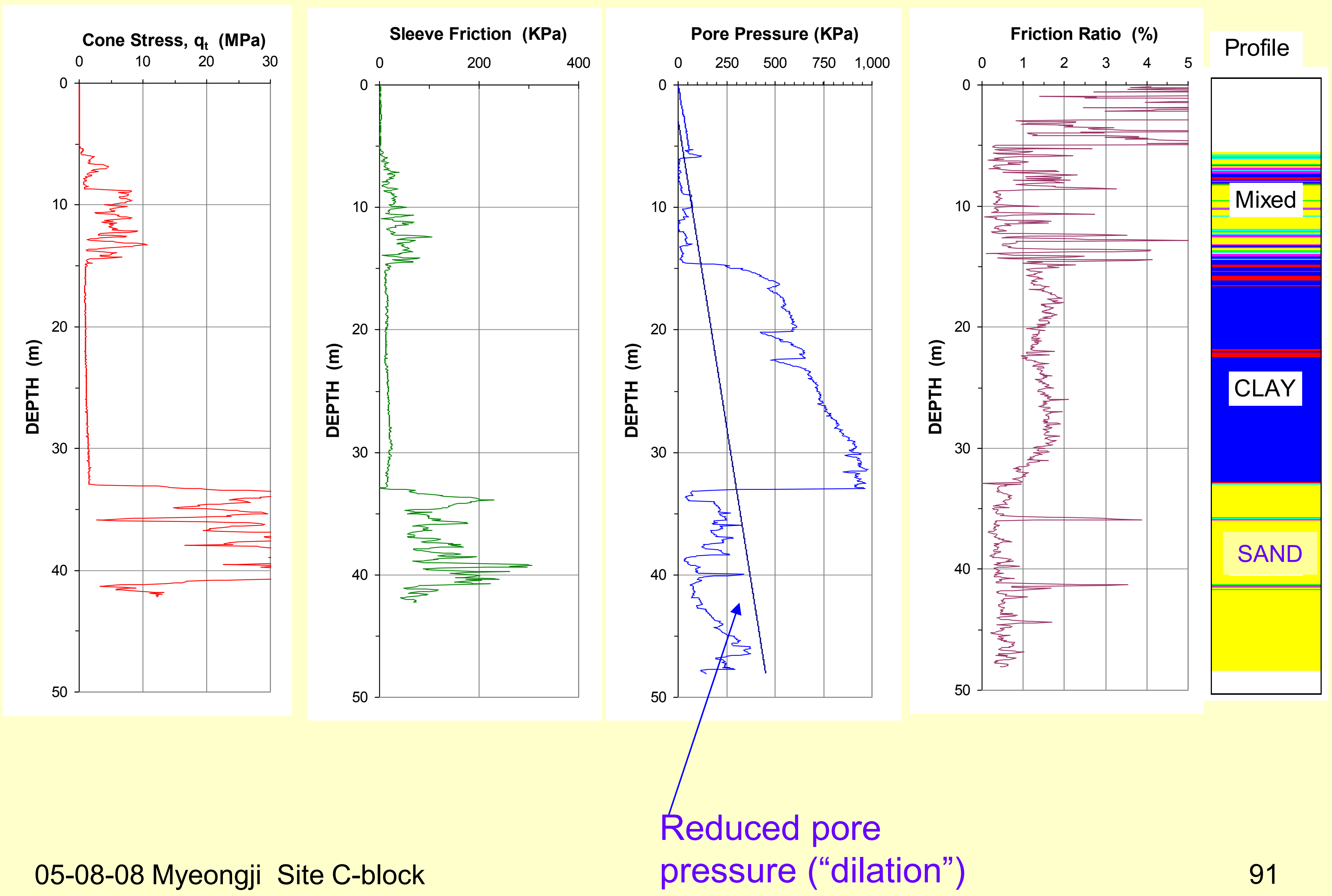
AIR VIEW (Shinho Site)

SITE PLAN (SH Site)

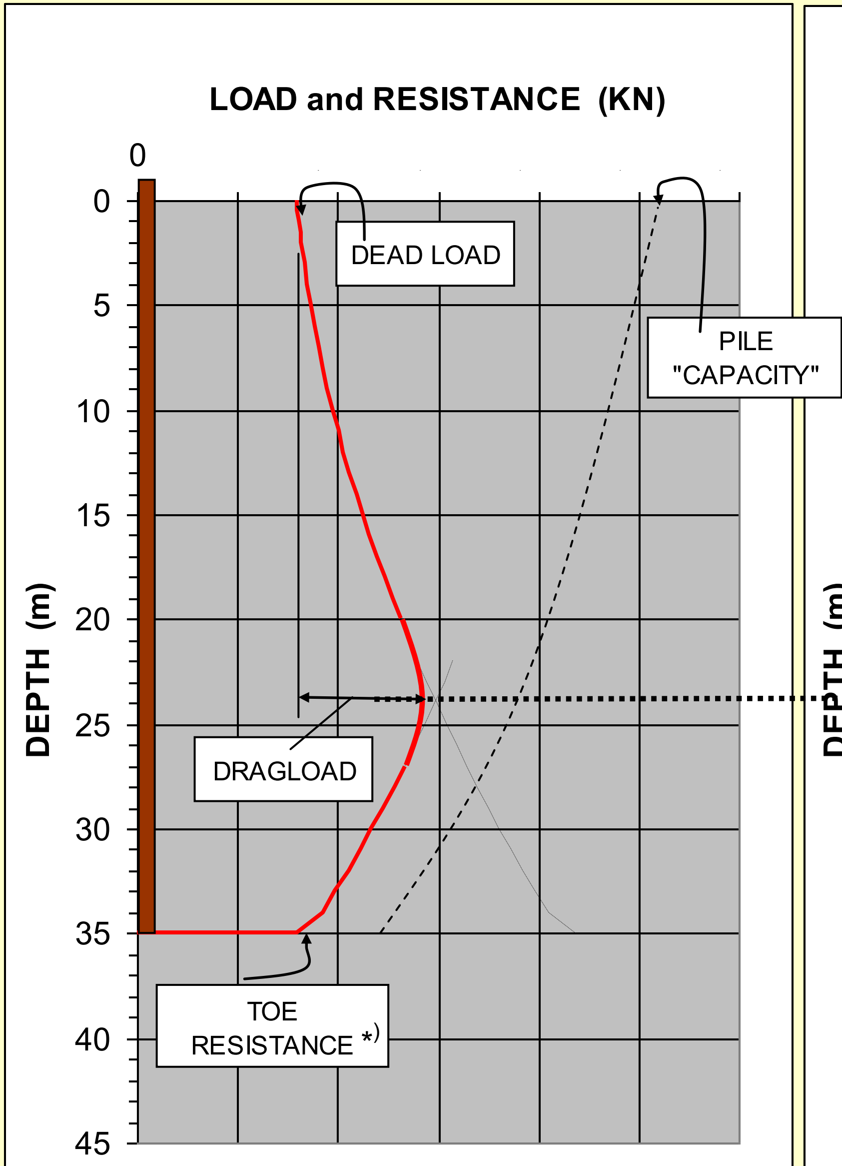




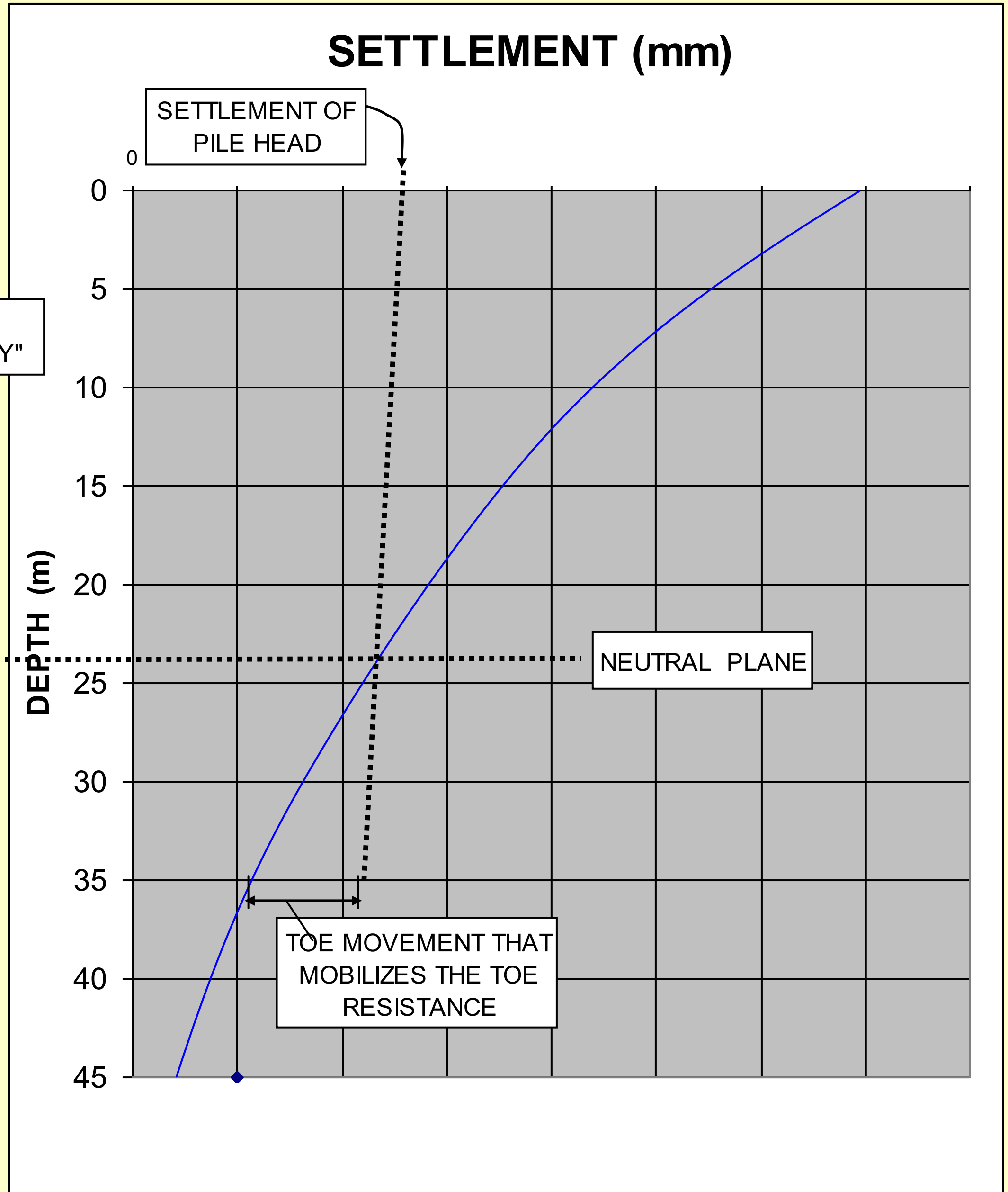
CPTU sounding
at the location
of the O-cell


Cone Stress, q_t (MPa)
0 10 20 30
DEPTH (m)
0 10 20 30 40 50
Sleeve Friction (KPa)
0 200 400
Pore Pressure (KPa)
0 250 500 750 1,000
Friction Ratio (%)
0 1 2 3 4 5
Profile
Mixed
CLAY
SAND
Reduced pore pressure ("dilation")

The Unified Method for Design of Piled Foundations
(typical only ; the numbers are not applicable to this site)



*) Portion of the toe resistance will have developed from the driving



The pile considered is a 600 mm diameter cylinder pile with a 100 mm wall driven closed-toe

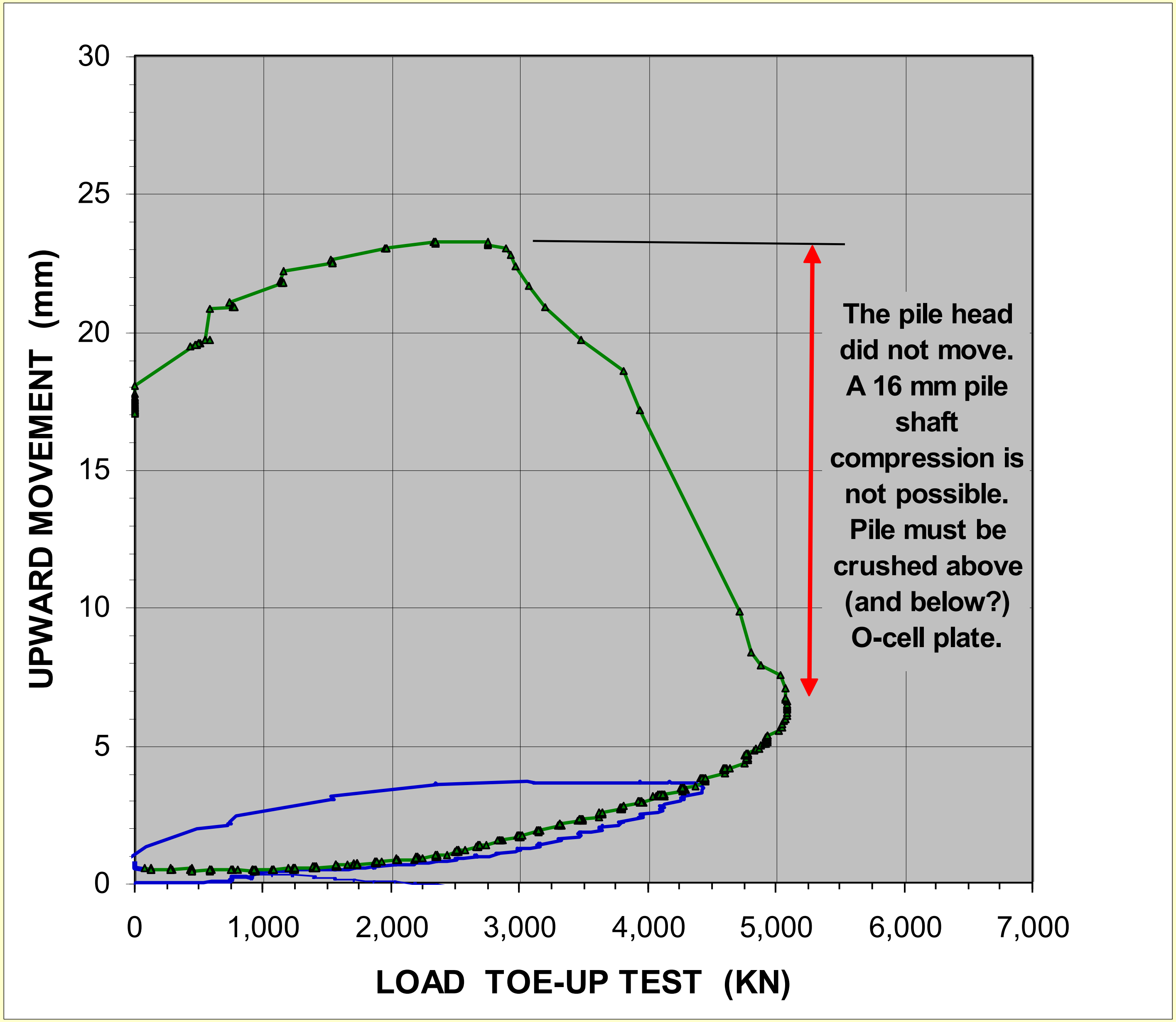
The questions to resolve in the design are

1. What is the capacity in the different layers?

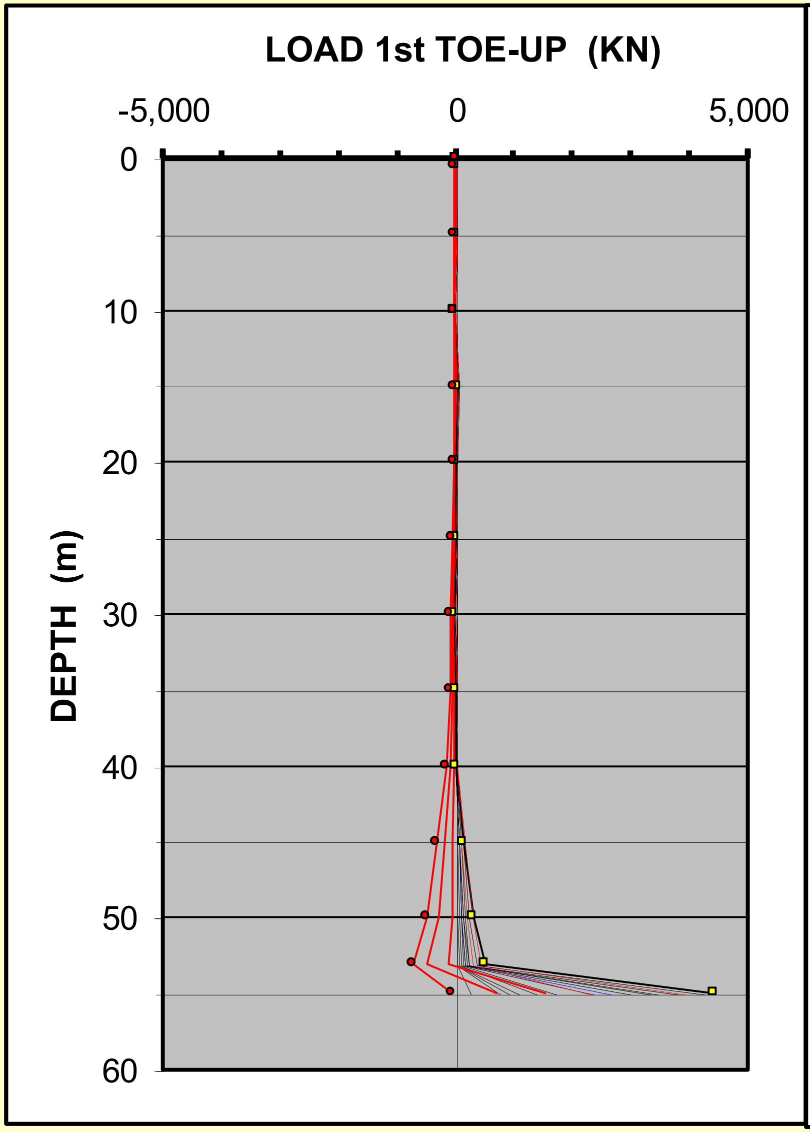
2. What is the depth to the force equilibrium/settlement equilibrium, i.e., the neutral plane

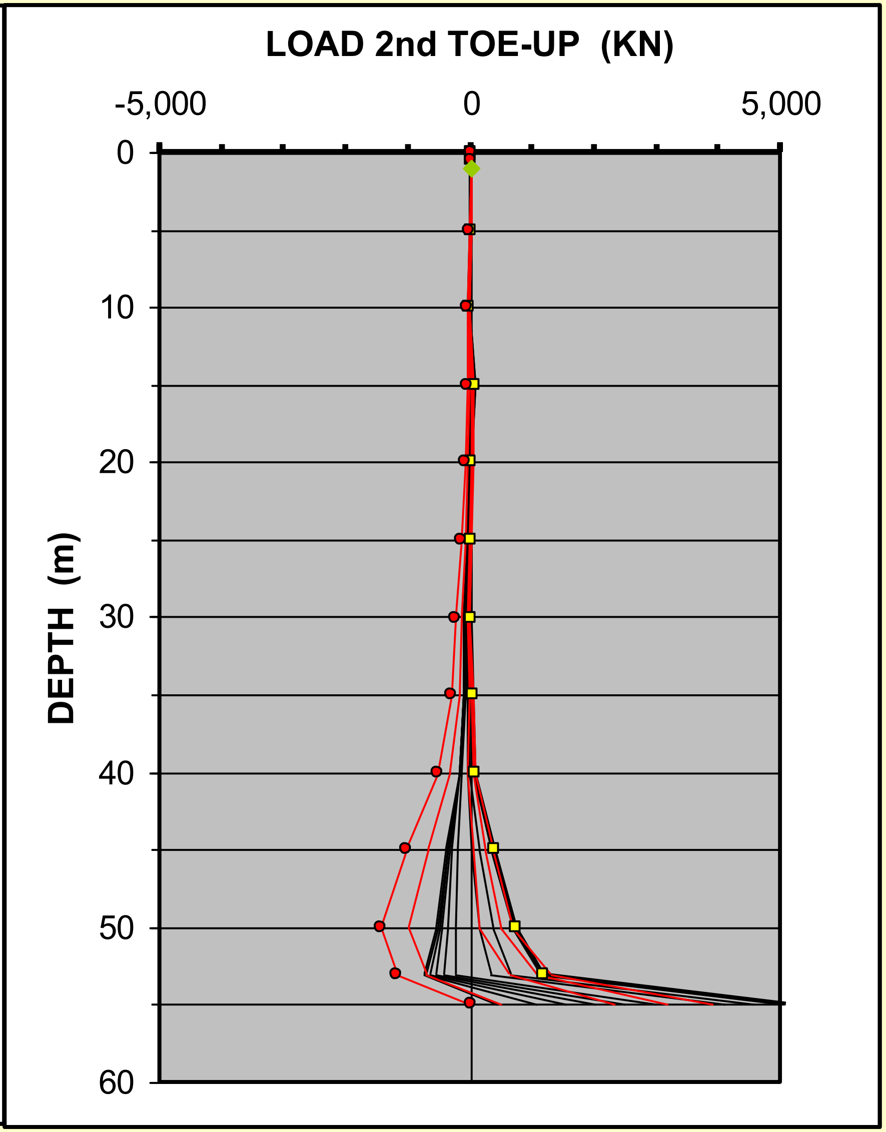
3. What will be the maximum load in the pile? Is the structural strength adequate?

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

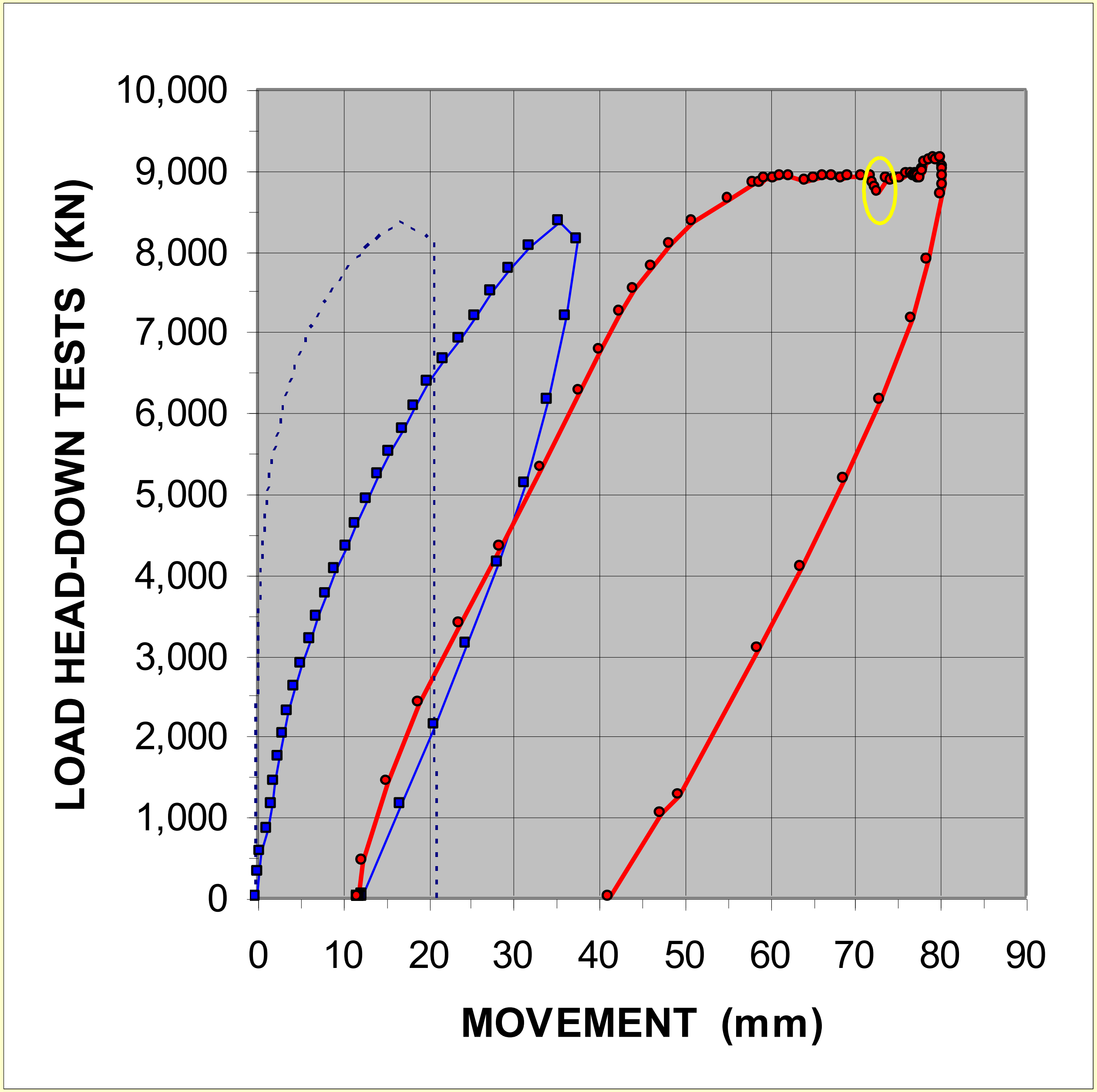


UPWARD MOVEMENT (mm)
LOAD TOE-UP TEST (KN)
30
25
20
15
10
5
0
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
The pile head did not move. A 16 mm pile shaft compression is not possible. Pile must be crushed above (and below?) O-cell plate.

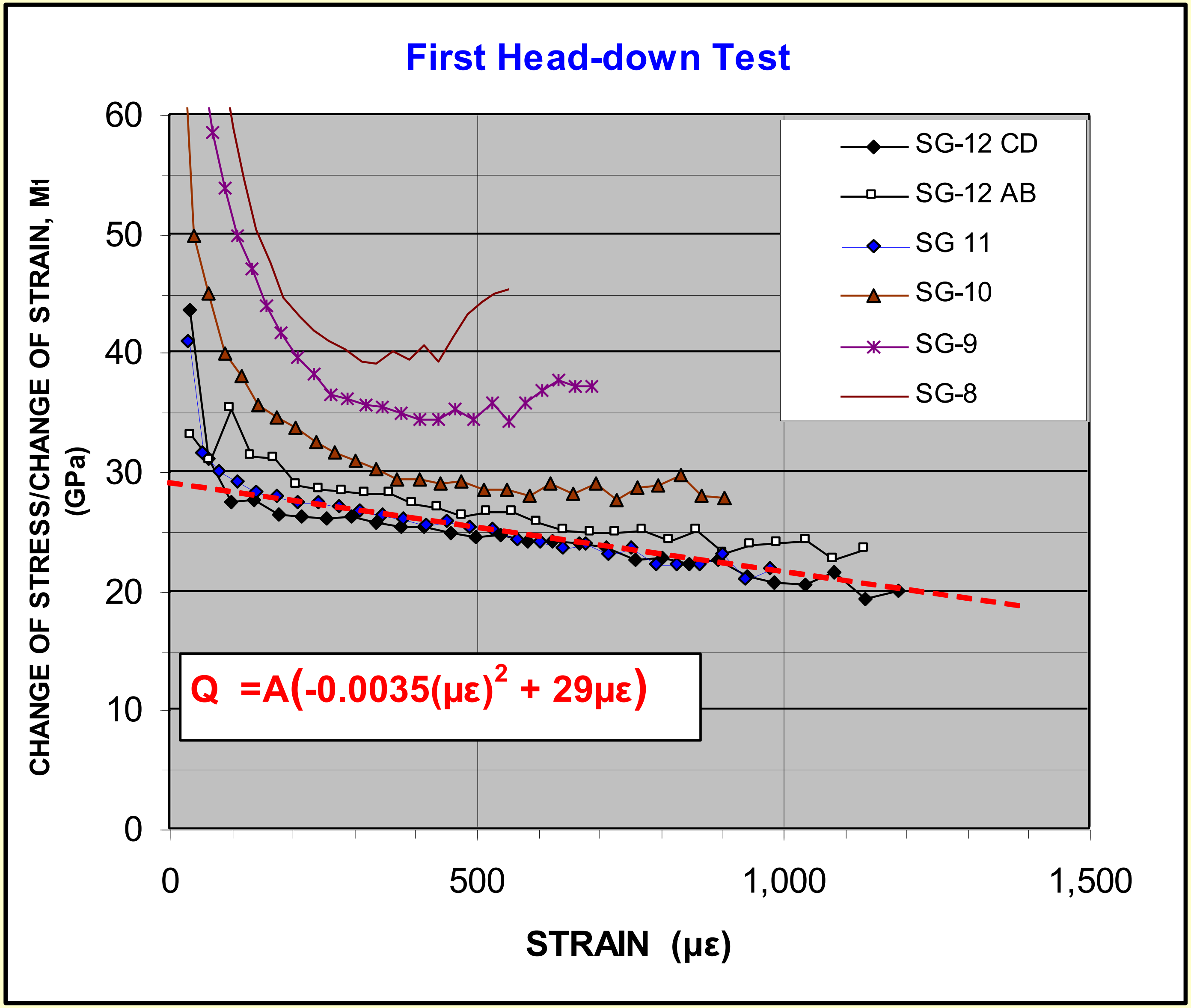

LOAD 1st TOE-UP (KN)
-5,000
0
5,000
0
10
20
30
40
50
60
DEPTH (m)


LOAD 2nd TOE-UP (KN)
-5,000
0
5,000
0
10
20
30
40
50
60
DEPTH (m)

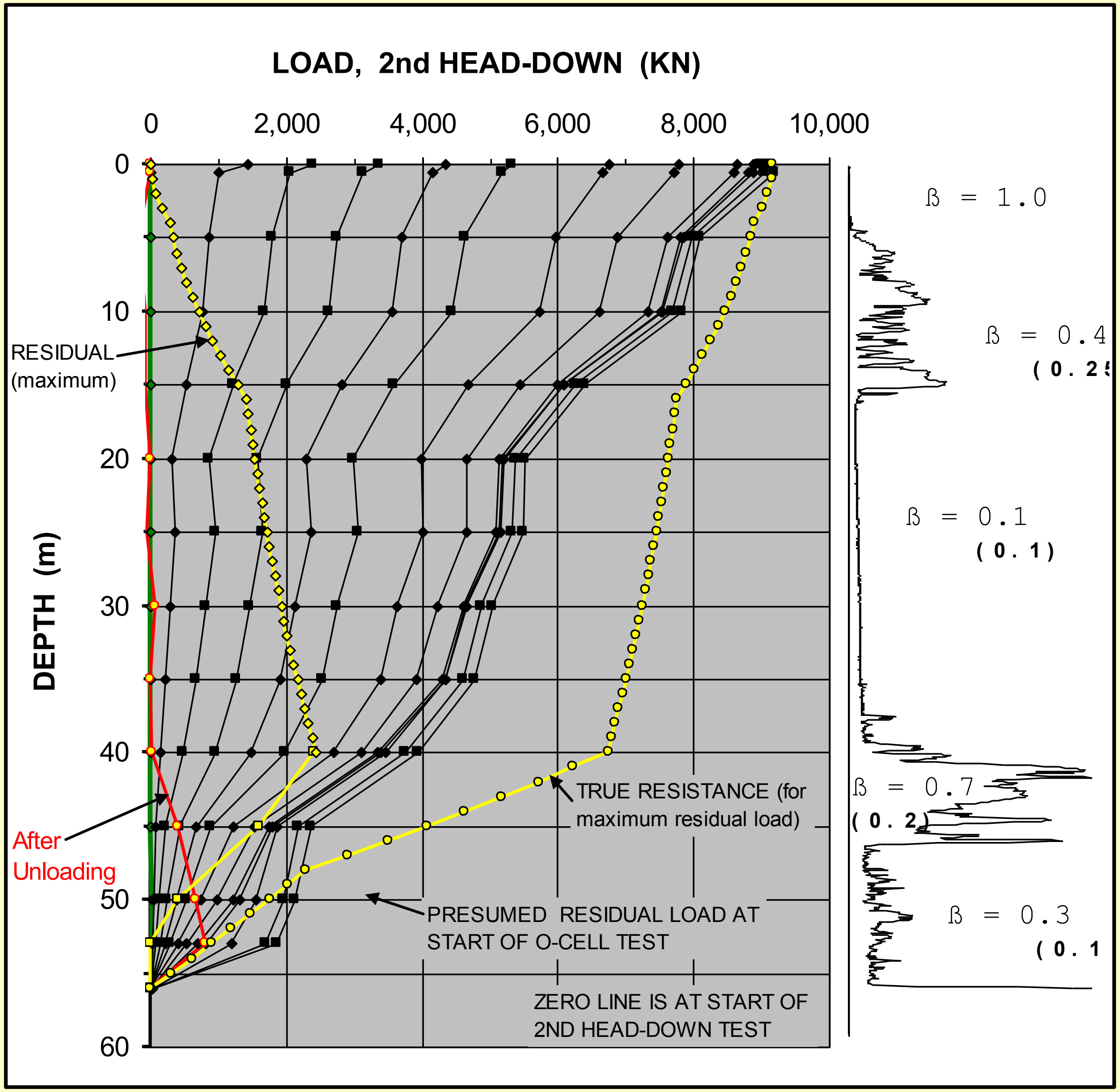
Now The Head-down Test



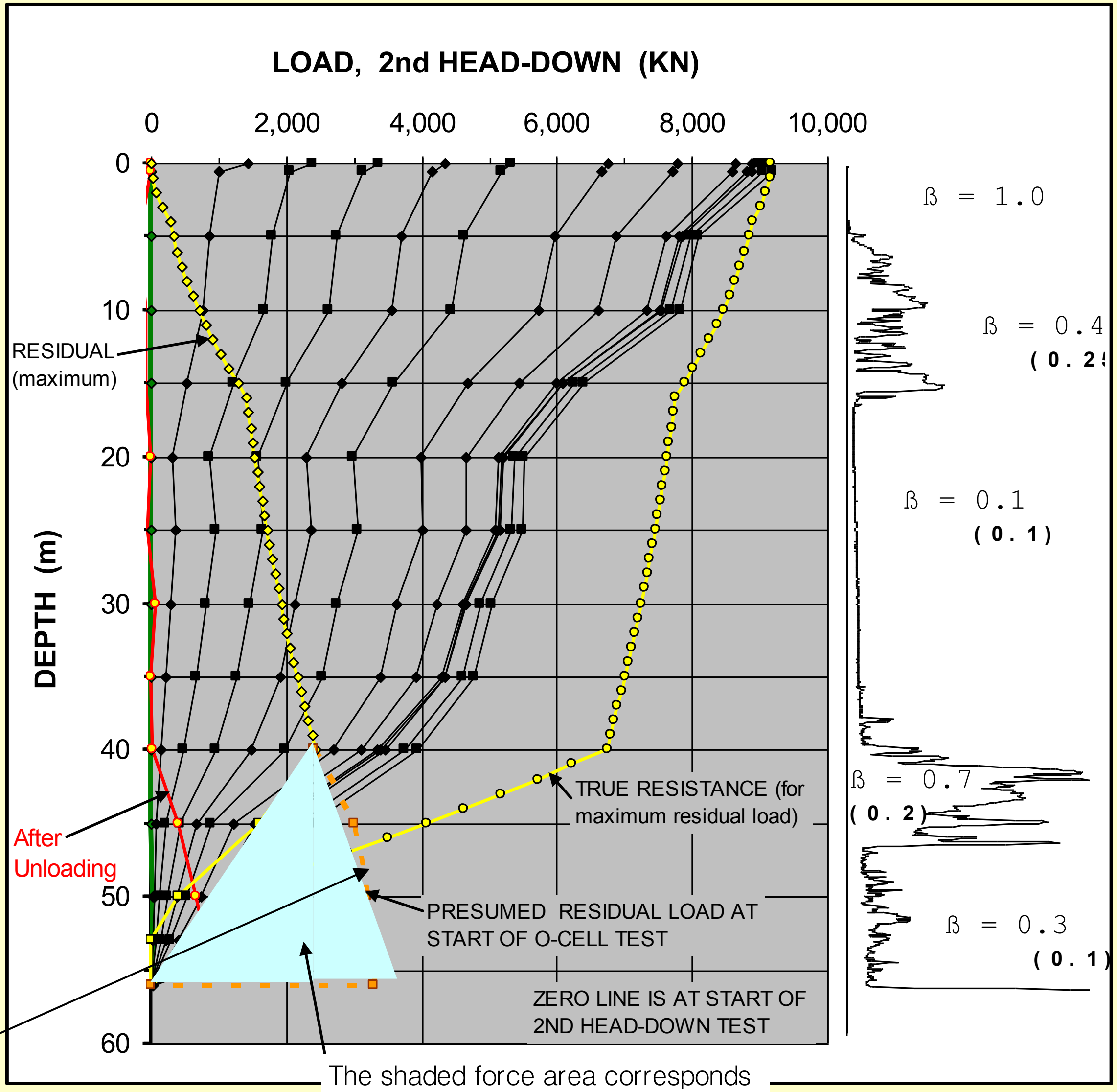
The Shinho test pile — head-down test



The Shinho test pile — head-down test

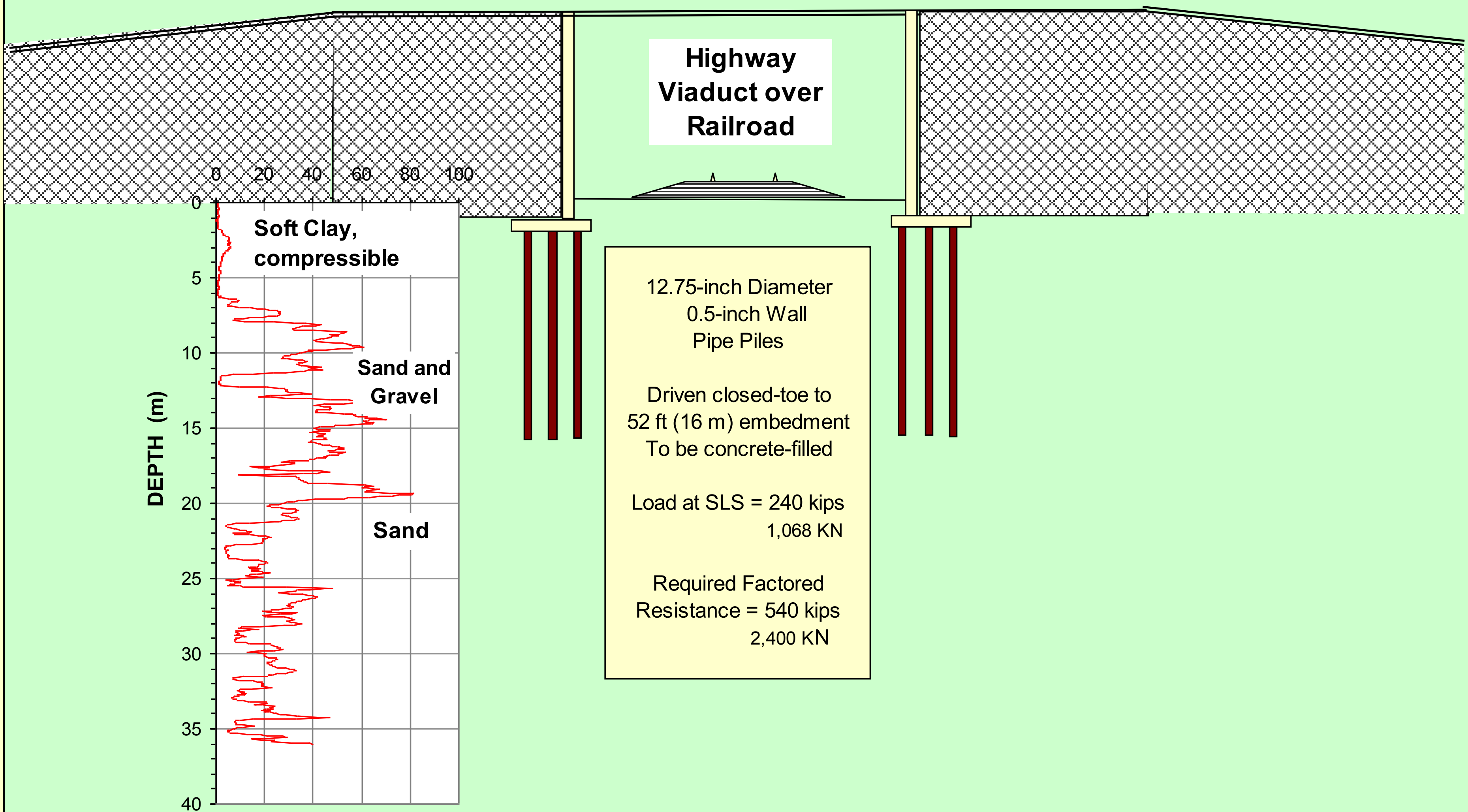


Estimated Residual Load Distribution at Start of the O-cell Test

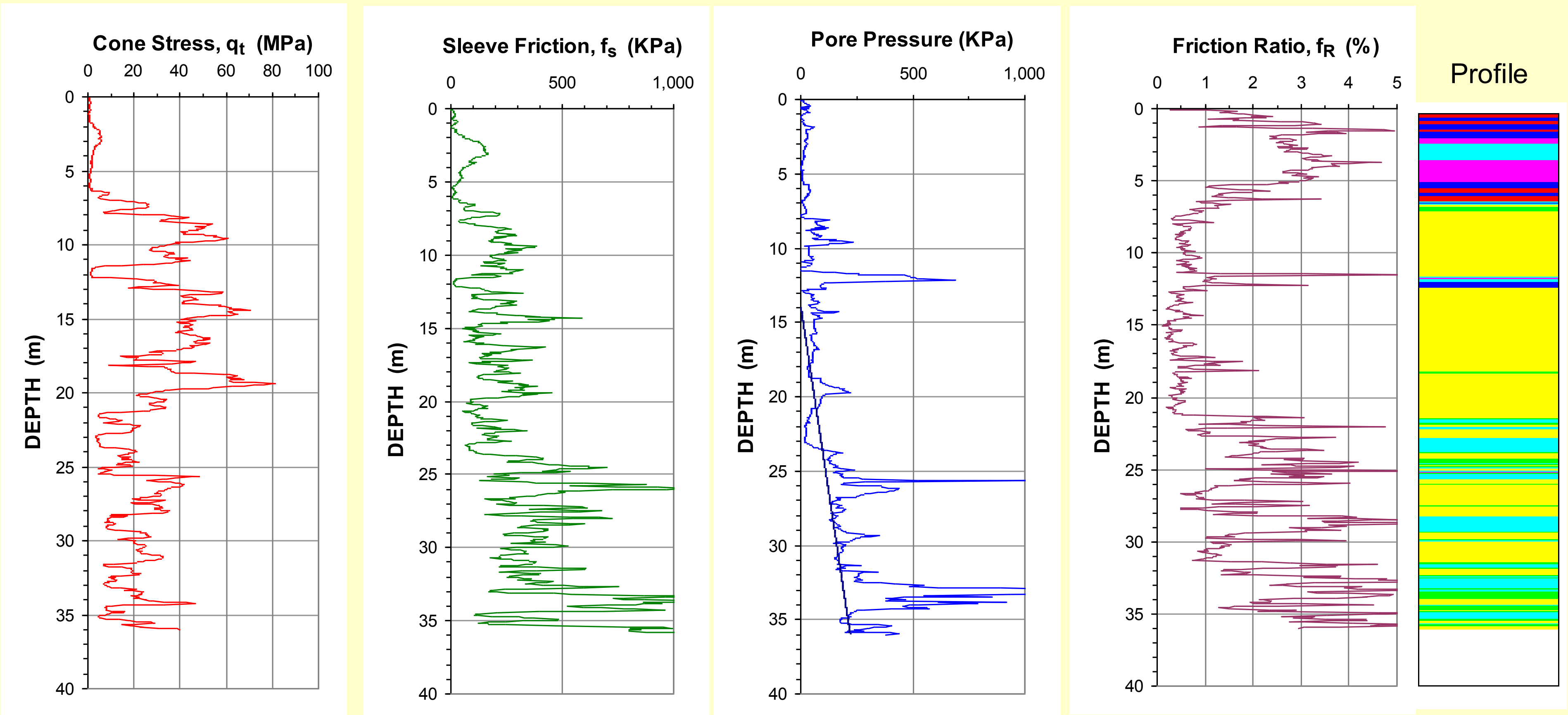


The shaded force area corresponds to a shortening of just about 3 mm

Milford, Beaver County, Utah

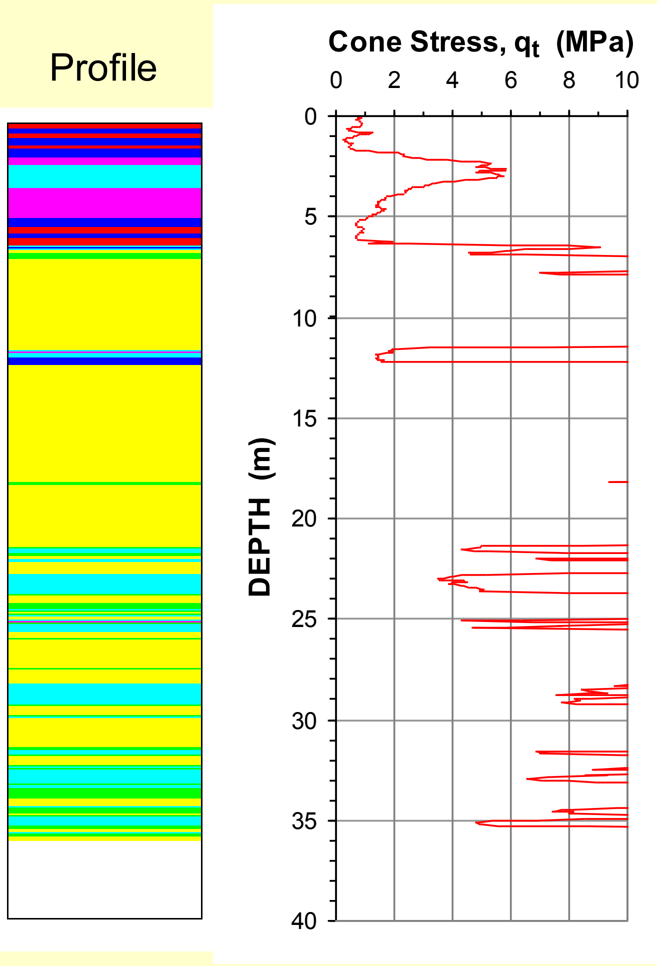


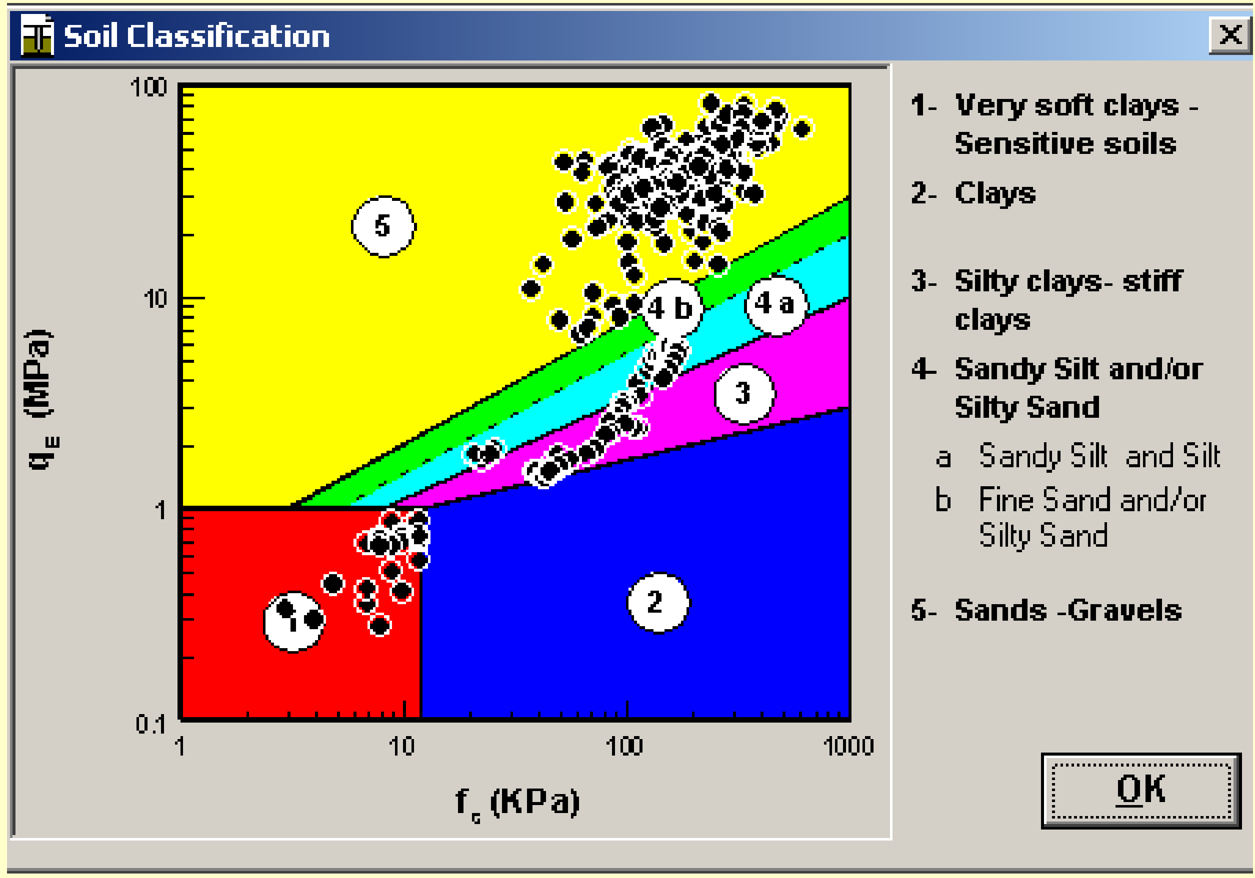
CPTU Sounding Results



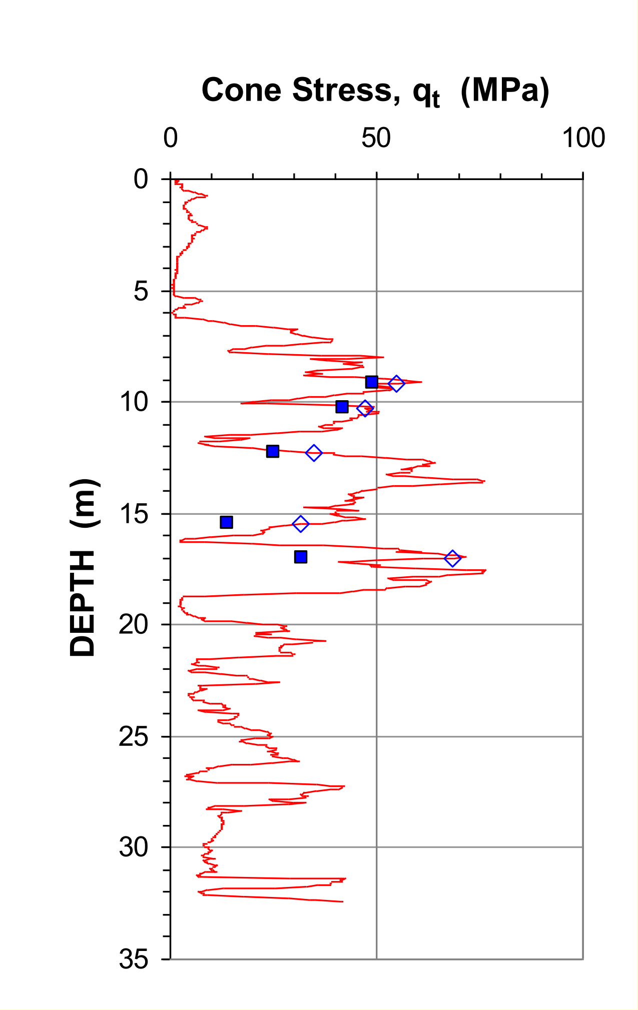
Enlarged Cone Stress Scale

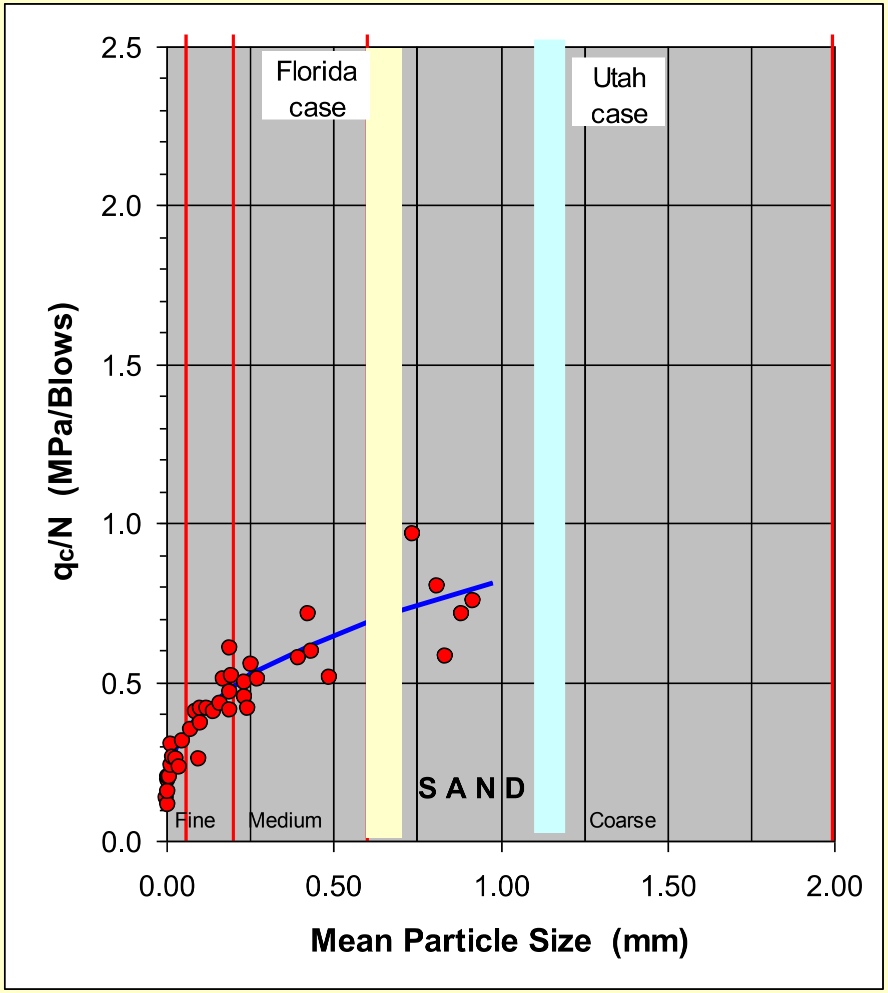
Soil Profiling Chart

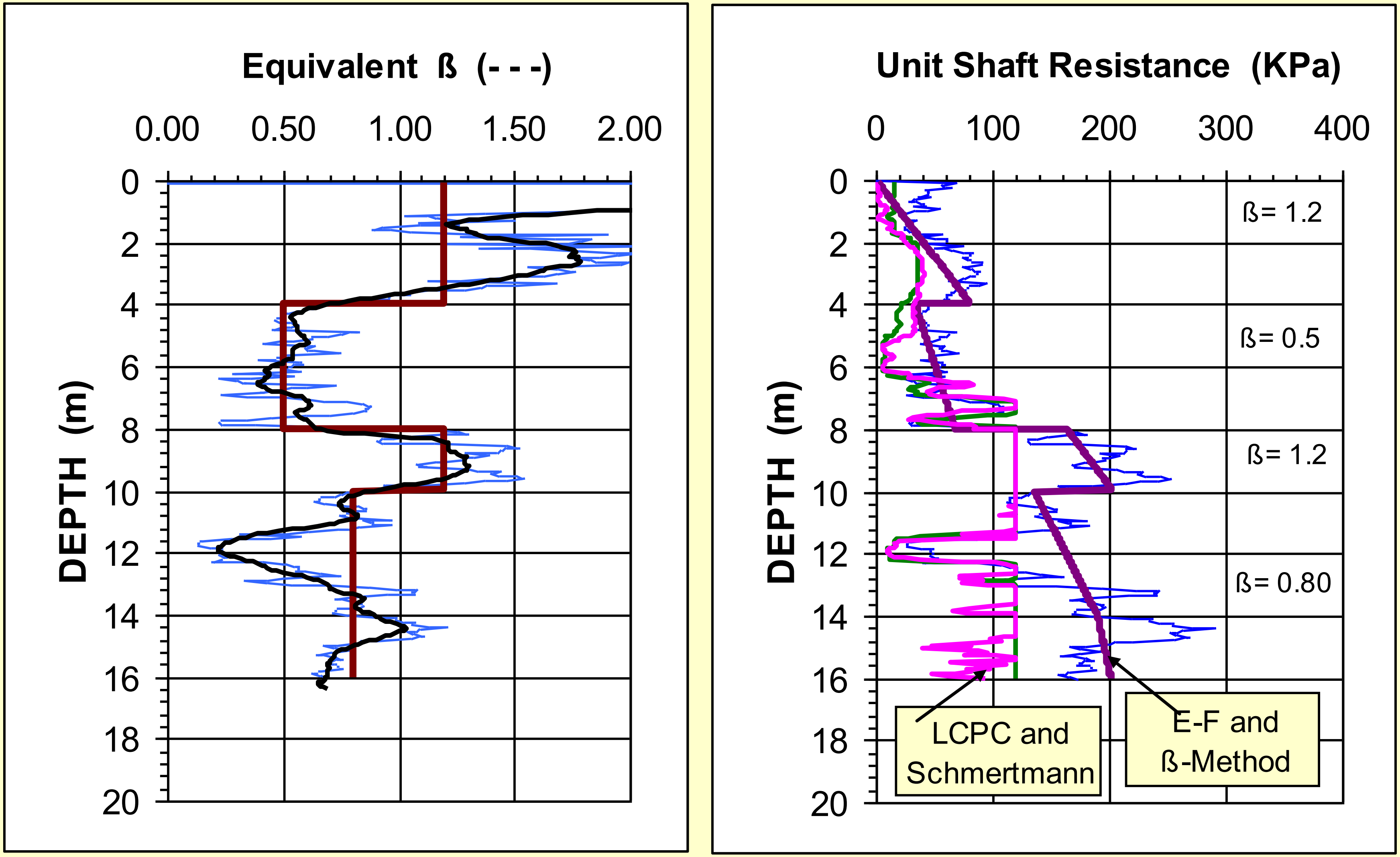




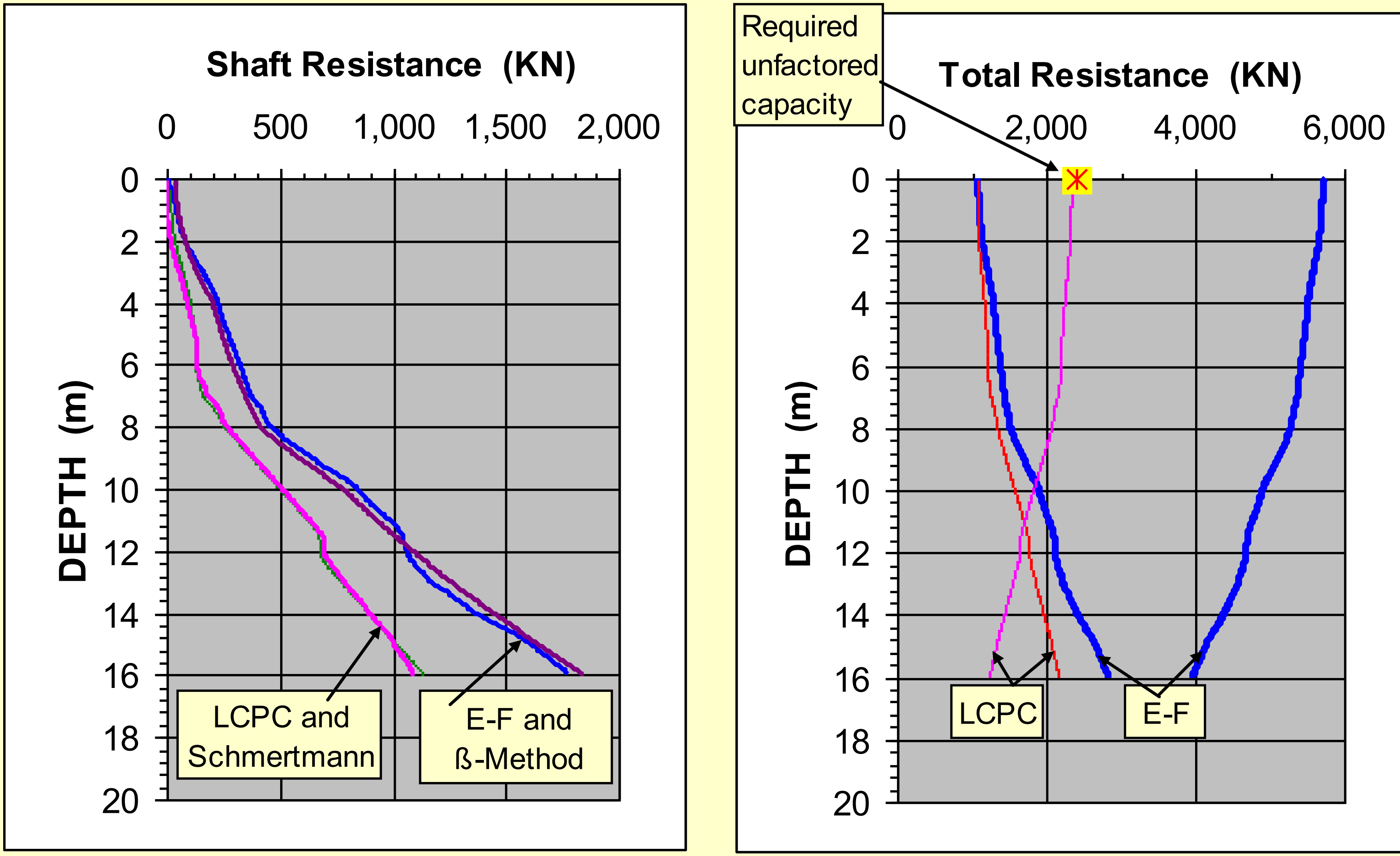
"Correlation" CPT - SPT

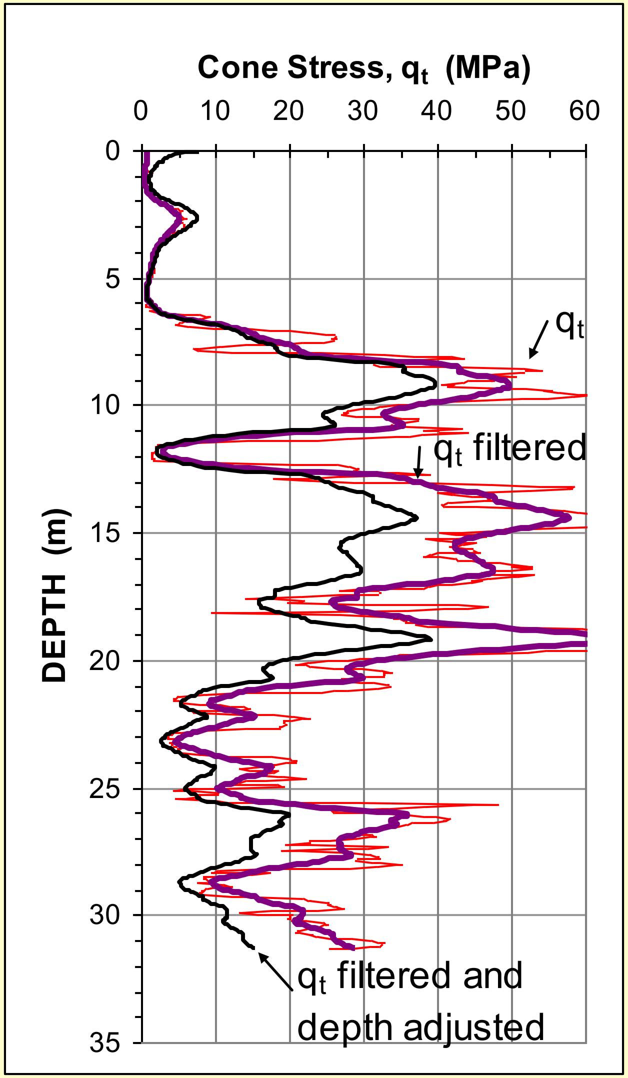


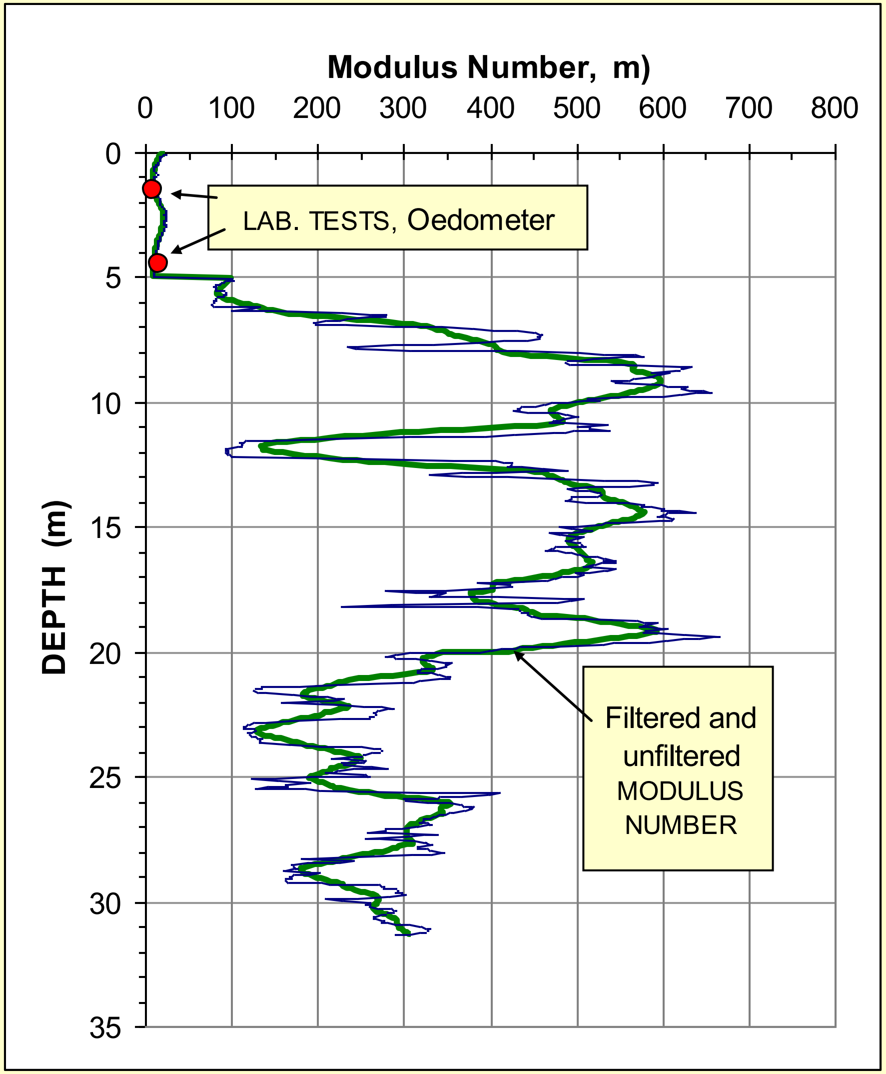


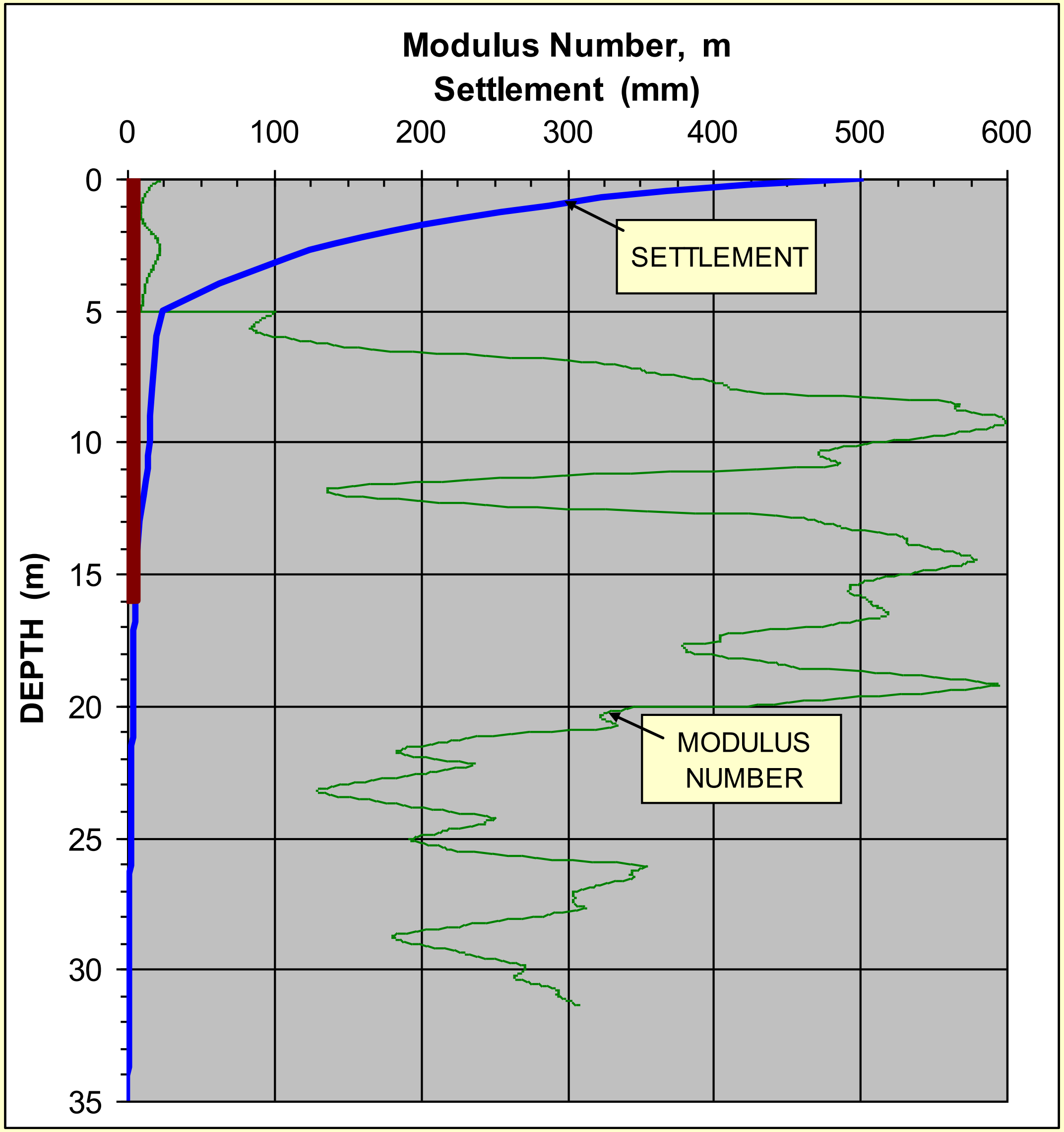


Equivalent ß (- - -)
0.00
0.50
1.00
1.50
2.00
0
2
4
6
8
10
12
14
16
18
20
DEPTH (m)
Unit Shaft Resistance (KPa)
0
100
200
300
400
ß= 1.2
ß= 0.5
ß= 1.2
ß= 0.80
LCPC and
Schmertmann
E-F and
ß-Method


Shaft Resistance (KN)
0
500
1,000
1,500
2,000
DEPTH (m)
0
2
4
6
8
10
12
14
16
18
20
LCPC and Schmertmann
E-F and ß-Method
Required unfactored capacity
Total Resistance (KN)
0
2,000
4,000
6,000
DEPTH (m)
0
2
4
6
8
10
12
14
16
18
20
LCPC
E-F


Cone Stress, q_t (MPa)
0
10
20
30
40
50
60
DEPTH (m)
0
5
10
15
20
25
30
35
q_t
q_t filtered
q_t filtered and depth adjusted


Modulus Number, m)
0
100
200
300
400
500
600
700
800
DEPTH (m)
0
5
10
15
20
25
30
35
LAB. TESTS, Oedometer
Filtered and unfiltered MODULUS NUMBER


Modulus Number, m
Settlement (mm)
0
100
200
300
400
500
600
0
5
10
15
20
25
30
35
DEPTH (m)
SETTLEMENT
MODULUS
NUMBER

Vaughani Shores, Port Vila, Vanuatu [www.DiveVanuatu.org

Comments for the handouts on Contact Stress, Piled Raft, and Piled Pad

At the level of the pile cap, there is no contact stress between the underside of the pile cap and the soil, because the soil will always settle more than the pile cap. Therefore, **it is incorrect to allow any contribution from contact stress.**

The exception to this is in the case of a **piled raft**, which is a term referring to a piled foundation designed with a factor of safety for the piles of close to unity, or better expressed: The neutral plane is designed to be located close to or at the underside of the raft. Only if the external loads on the pile cap are equal to or larger than the combined pile capacities will there be a contact stress.

The emphasis of the design for a piled raft is on ensuring that the contact stress is uniformly distributed across the raft. The piled-raft design intends for the piles to serve both as soil reinforcing (stiffening) elements reducing settlements **and** as units for receiving unavoidable concentrated loads on the raft. This condition governs the distribution across the raft of the number and spacing of the piles.

The design first decides on the depth and number of piles (average spacing and lower boundary number of piles) necessary for reinforcing the soil so that the settlement for the raft is at or below the acceptable level. This analysis includes all loads to be supported by the raft. Thereafter, the magnitude of the uniform contact stress is decided, and finally, the spacing and number of piles to carry load concentrations (the portion of the load exceeding that determining the contact stress) are designed as to depth and locations assigning them a factor of safety of unity. An iterative procedure of these steps may be required.

A further development of the Piled Raft is the ”**Piled Pad**”, also called “Disconnected Footing/Piled Foundation” (*!*), which really is a soil improvement method that lately has met with considerable interest after its use for the Rion-Antirion Bridge. The Piled Pad combines stiffening up the soil with piles and placing a compacted backfill between the piles and a footing slab. The piles are calculated to carry only a portion of the load (The factor of safety may be smaller than unity) and the design is for settlement.

There will be conditions that warrant reducing the negative skin friction (e.g., in order to lower the location of the neutral plane and/or reducing the maximum load in the pile). As the case histories have shown, bitumen coating will be very efficient in this regard. However, it comes at a price — $$$ and frustrations — and it should only be contemplated as a last resort.

Comments for the handouts on the Case with the 25 m long pile

First, the drag load does not affect the pile bearing capacity — the ultimate resistance. The only forces and loads to include are those present were the factors-of-safety equal to unity — which is **analysis of the ultimate state**. For piles, this is the plunging state and no drag load is present at that stage, as the pile is moving down against the soil along its entire length. Therefore, only the first approach, that with the allowable load of 700 KN, is correct.

Second, the drag load is only of importance with regard to the pile structural strength. The critical location lies at the location of the maximum load, i.e., at the neutral plane. If the structural integrity of the pile is safe considering the sum of dead load and drag load, i.e., 900 KN, the design for drag load is complete.

Third, the settlement of the soil might drag the pile down. If the soil is settling at the depth of the neutral plane, downdrag will occur.

There are two different definitions of the neutral plane. The two give the same result, or location, rather. One defines it as located at <u>the force equilibrium</u> in the pile, which is where the shaft resistance changes from negative to positive direction and dead load and drag load are in equilibrium with the positive forces in the pile. (Note, the toe resistance is always only as large as is needed to establish an equilibrium between forces and movements). The other defines the location to where the pile and the soil move equally—"<u>the settlement equilibrium</u>". (Sometimes it is overlooked that however large the pile capacity and however large the factor of safety, if the soil is settling at the neutral plane, the pile will settle too and by that amount). The two definitions are illustrated in previous figure. The diagram to the left shows the load distributions and locations of the neutral plane for the dead loads associated with the different allowable loads on the pile (the 100 KN case is excluded). The diagram to the right shows the distribution of soil settlement and location of neutral planes for the three approaches.

Note, reduction of the allowable load (dead load) results in a lowering of the neutral plane and a smaller settlement. The latter may be very desirable. However, it is usually more economically achieved by installing the pile deeper.

Summary opinion

When faced with a design of a pile foundation, the first step is to look at available pile types and installation methods and consider required capacity in selection the pile type(s) suitable for the foundation. When getting closer to the actual pile design, an analysis is performed to determine the shaft and toe resistances and the load-transfer conditions (the load distribution). That is, the capacity requirement governs at this stage. If the soils are not expected to settle (low-compressibility soils, no fills, other loads, or groundwater lowering that can result in increase of effective stress), then, the design effort is limited to ensuring that the piles will indeed provide the desired capacity.

In finalizing the design, the capacity analysis should be fine-tuned by using all the available information to establish the load-movement behavior of the pile toe so that the expected movement of the pile cap can be determined. It is not good enough to state that the piled foundation has a factor-of-safety of a certain at-least value. To satisfy serviceability requirement, the anticipated settlement (deformation) needs to be provided to the structural engineer. This effort involves estimating the height of the "transition zone" (the zone within which the negative direction of the shear forces changes to positive direction).

Some designers would subtract the 300 KN drag load from the 1,400 KN pile capacity before applying the 2.0 factor-of-safety and arrive at an amended allowable load of **550 KN** (which is a violation of principles as it reduces the drag load by a factor of 2.0)?

Others realize that the foregoing approach means that the drag load is applied without a factor-of-safety, preferring to increase the drag load by multiplying it with a factor-of-safety? This results in an allowable load of (1,400/2 - 2x300) = 100 KN — don't laugh, I have seen it done!

Suppose the structure resting on the piled foundation was built applying the 700 KN load before the drag load conditions were recognized. Then, what factor-of-safety would the piles be considered to have? Would it be 1,400/700 = 2.0, or (1,400 - 300)/700 = 1.6, or 1,400/(700+300) = 1.4? [I wonder how a fellow preferring the laughable approach (the 100 KN allowable load) would react when realizing that the piles are supporting 7 times more load than the maximum load their approach would allow as safe].

Before answering, consider that the magnitude of the drag load depends on the magnitude of the dead load on the piles. For the case of an allowable load of 400 KN (made up of a dead load of, say, 325 KN and a live load of 75 KN), the drag load is no longer 300 KN, it is 400 KN! For an allowable load of 550 KN (made up of a dead load of, say, 475 KN and a live load of 75 KN), the drag load is 500 KN. "***But wait, there is more!***", if the dead load is reduced from 600 KN to 325 KN or to 475 KN, the neutral plane location changes from 17.0 m to 19.5 m or 18.0 m, respectively. "***But wait, there is more!***", the deeper down the neutral plane lies, the smaller the enforced penetration of the pile toe into the sand and the smaller the mobilized toe resistance. And, when the toe resistance reduces, the location of the neutral planes moves upward and the drag load increases. ***Nature always wins! Stupidity and ignorance never does — in the long run, that is.***

7.5 Piled Raft and Piled Pad Foundations

Every design of a piled foundation postulates a stable long term situation. "Stable" means that the foundation has reached an equilibrium state with the location of the neutral plane established and when more or less all settlement has developed. For a conventional piled foundation design, i.e., a pile cap cast on the piles, the neutral plane lies well down in the soil. This means that there is no physical contact between the underside of the pile cap and the soil immediately below the pile cap, or, at least, there is no load transfer to the soil from the pile cap (i.e., no contact stress). Therefore, the design for service conditions must not include any benefit from the pile cap transferring loads directly onto the soil through contact stress. A design considering contact stress is not a conventional design, it is a design for a piled raft.

A piled raft is a foundation supported on piles that have a factor of safety of unity or smaller, which places the neutral plane at the underside of the pile cap—the raft. Such designs emphasize the settlement behavior of the foundation (discussed below). Note, the neutral plane is the location of the force equilibrium and of the settlement equilibrium. Both are affected by the magnitude of the toe resistance, which is a function of the load-movement response of the pile toe with the movement governed by the soil settlement at the neutral plane, and both are located at the same depth.

The emphasis of the design for a piled raft lies on ensuring that the contact stress is uniformly distributed across the raft. The contact stress is the effect of the load on the raft that is not supported by the piles. This means that contact tress only develops if the piles support less than the full load (Fs □ 1.0) .

The piled-raft design intends for the piles to serve both as soil reinforcing (stiffening) elements reducing settlements and as units for receiving unavoidable concentrated loads on the raft. This condition governs the distribution across the raft of the number and spacing of the piles.

The design of a piled raft first decides on the depth of the piles and stiffness of the piles plus soil (governs the average spacing and lower boundary number of piles) necessary for reinforcing the soil so that the settlement of the raft is at or below the acceptable level. This analysis includes all loads to be supported by the raft and includes a check that the number of piles assumed involved will be assigned an average load larger than the capacity of the average pile, i.e., the average Fs is equal to or smaller than unity. Thereafter, a uniform, lower-bound magnitude, design contact stress is chosen, and the design verifies that the piles do not have an average factor of safety larger than unity for that lower-bound contact stress. Unavoidably, the raft will have concentrations of load, however. Wherever this occurs, the portion of the load that causes a stress larger than the chosen design contact stress is supported on additional piles at number, spacing, and depth governed by the surplus (or "overload") portion. An iterative procedure of these steps may be required. The design of the raft itself needs to include margins for the possibility that the contact stress is larger than estimated and, also, that the pile loads will be larger than estimated. Where the loading conditions include large and unevenly distributed live loads, a piled raft foundation may be less suitable

A **piled pad** foundation is similar to a piled raft foundation). However, the piles are not connected to the raft, as a pad of compacted coarse-grained fill is placed around the pile heads and above. The foundation is then a conventional footing cast on the compacted fill above the pile-reinforced soil.

With regard to the soil response to vertical loads of the foundation, the difference between the types is small (though the structural design of the concrete footing and the concrete cap will be different). For both the piled raft and the piled pad foundations, the piles are designed to a factor of safety of unity or smaller. For in particular the piled raft foundation, a factor of safety larger than unity on the pile capacity may result in undesirable stress concentrations. The main difference between the raft and the pad approaches lies with regard to the response of the foundations to horizontal loading and seismic events.

The piles for a piled raft foundation are connected through the raft and this will minimize the effect of any lateral spreading due to the contact stress. Resistance to horizontal loading by a piled raft foundation is obtained by means of pile response to horizontal load. A piled pad foundation provides little resistance to horizontal loads and is less sensitive to lateral spreading. The design of a piled pad foundation needs to consider the potential of lateral soil-spreading under the foundation. This is offset by having the pile group area larger than the area (footprint) of the footing on the pad, incorporating horizontal soil reinforcement in the pad, minimizing the lateral spreading by incorporating vertical drains (wick drains, see Chapter 4) to suitable depths, etc. In case of a piled raft or a conventional piled foundation, the connection of the piles to the concrete footing will restrain lateral soil spreading.

A main advantage for the piled pad foundation is claimed to lie in that the pad can provide a beneficial cushioning effect during a seismic event.

Perhaps the largest difference between the piled raft and piled pad foundation, as opposed to a conventional piled foundation lies in that the former are soil improvement methods to be analyzed from the view of deformation (vertical and horizontal), whereas the conventional foundation also needs to be analyzed from a bearing capacity view with due application of factor-of-safety to the pile capacity.

For settlement response, both foundations can be analyzed as a block (within the pile depths) having a compressibility obtained from proportioning the modulus of the soil and the pile to the respective cross section areas.

A conventional piled foundation is used to support all kinds of structures, whereas piled raft foundations are thought best for supporting structures with large footprint (large floor area), such as buildings as opposed to pile bents, and bridge piers, for example. However, a piled raft or a piled pad can equally well be used to support small footprint structures.