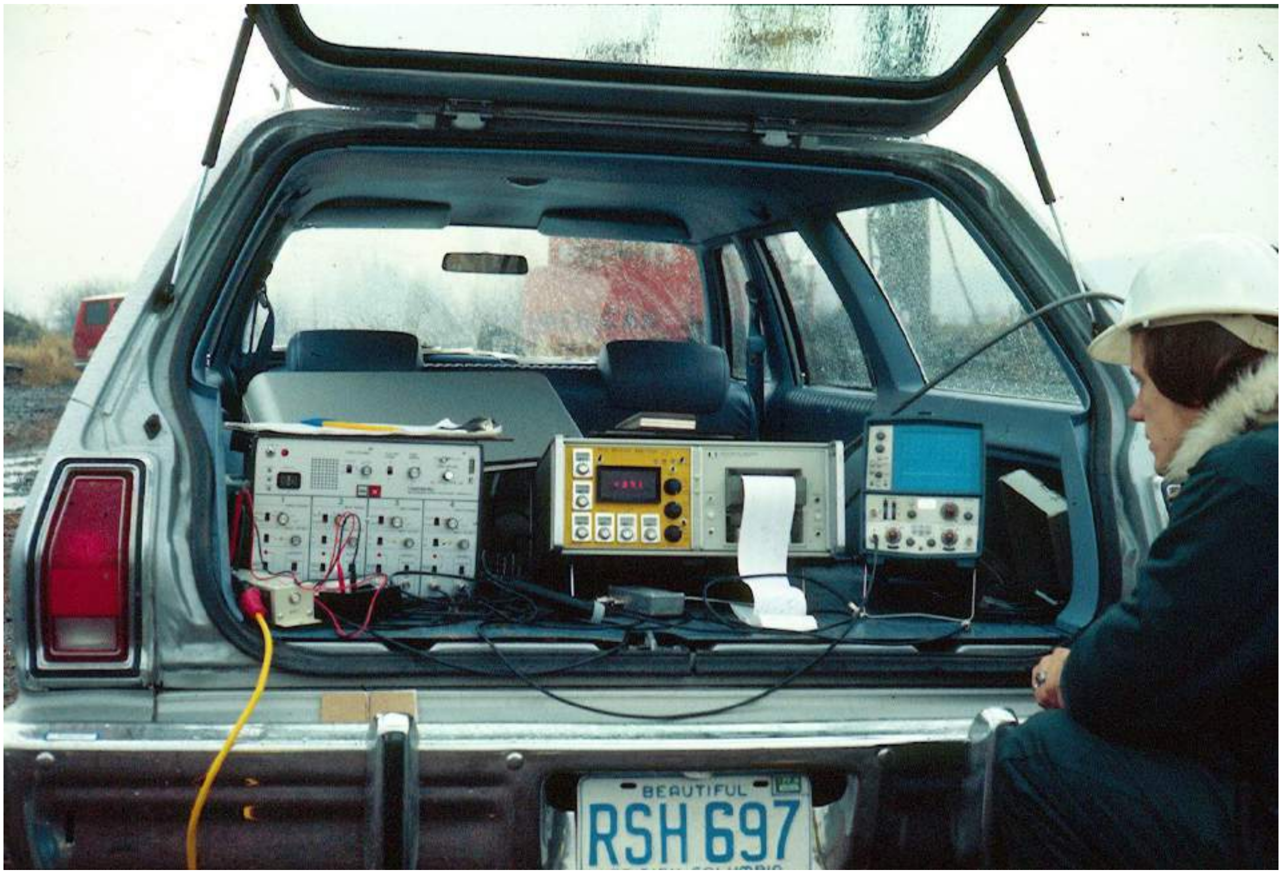


1977

387. Fellenius, B.H. 2018. Sixty Years of dynamic testing and analysis of piles—A retrospective. Keynote Lecture to the 10th International Conference on Stress-wave Theory and Testing. June 27-29, 2018, San Diego, CA, 50 p.

Sixty Years of dynamic testing and analysis of piles

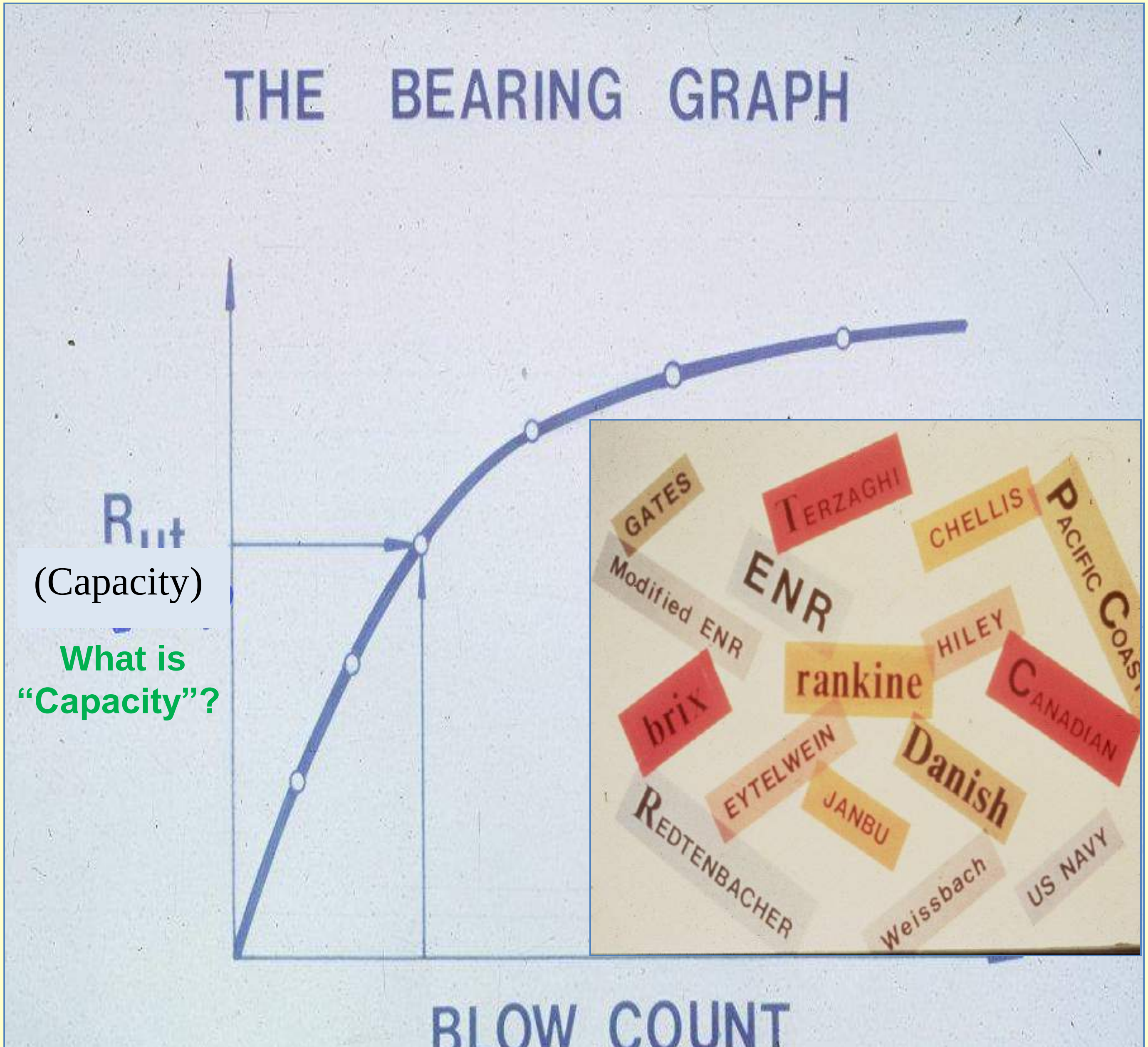
A retrospective

Bengt H. Fellenius

June 29, 2018

10th International Conference on Stress-wave Theory and Testing Methods For Deep Foundations
June 27-29, 2018, San Diego, CA

- *Proudly Sponsored By: The Pile Driving Contractors Association*



(Capacity)

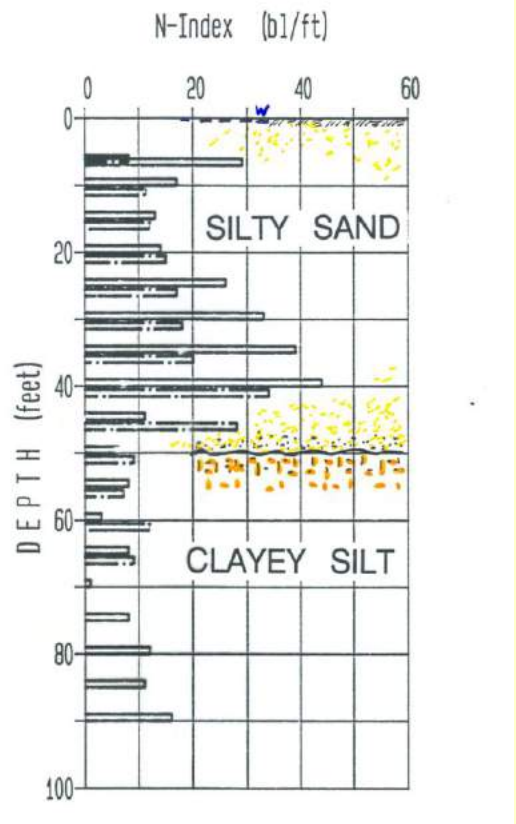
What is “Capacity”?

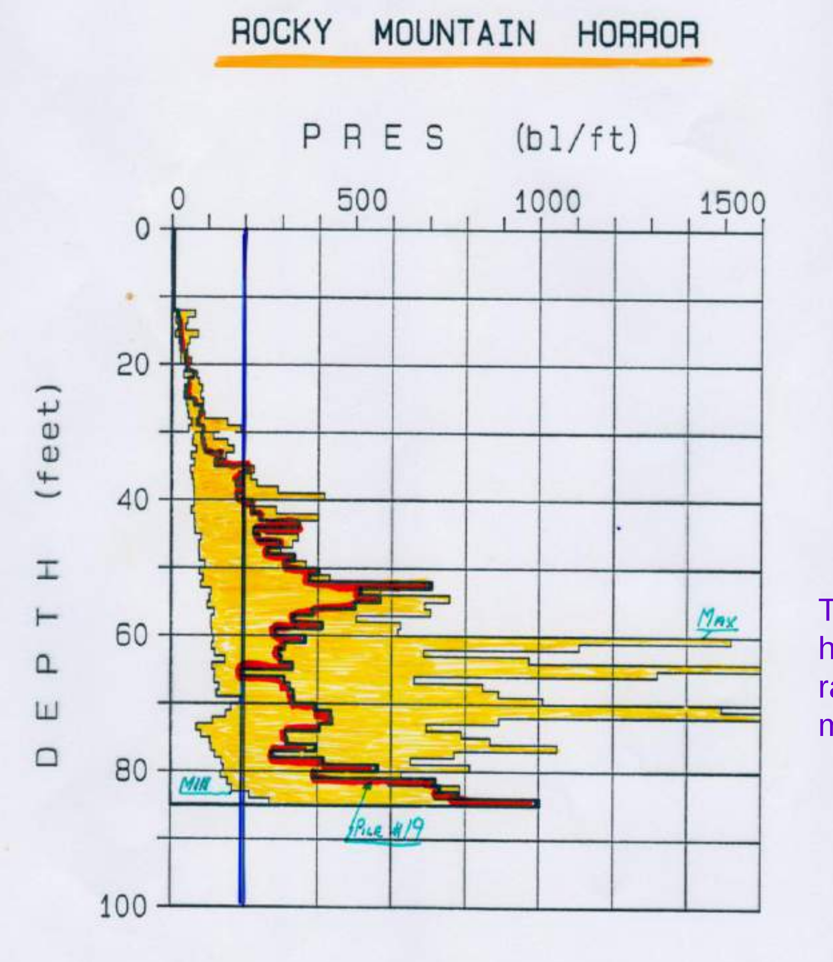
Slide by courtesy of
Mohamad Hussein, GRL

A project in the early 1970s illustrating that, despite the diminishing return of the blow-count demonstrated by the dynamic formulae, then, as now, stupidity prevails.

To support the tower, the design required 23 steel H-piles driven to 85 ft depth.

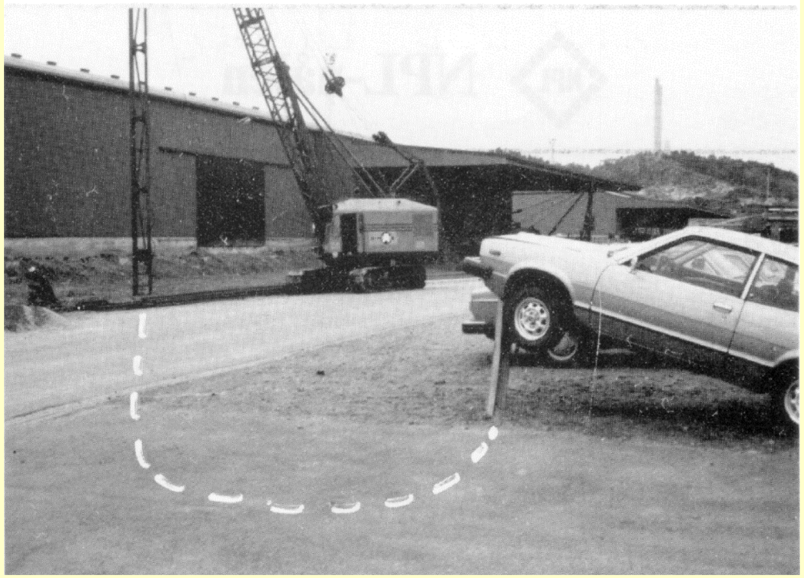
Soil profile and SPT N-diagram at a piled foundation for a power line tower in the middle of Alaska





The drop hammer height-of-fall was raised to more than 10 ft!

Another project at about the same time. Here the contractor had no problem getting the piles down to specified depth. The toe resistance was rather small toward the end, though.



Side View

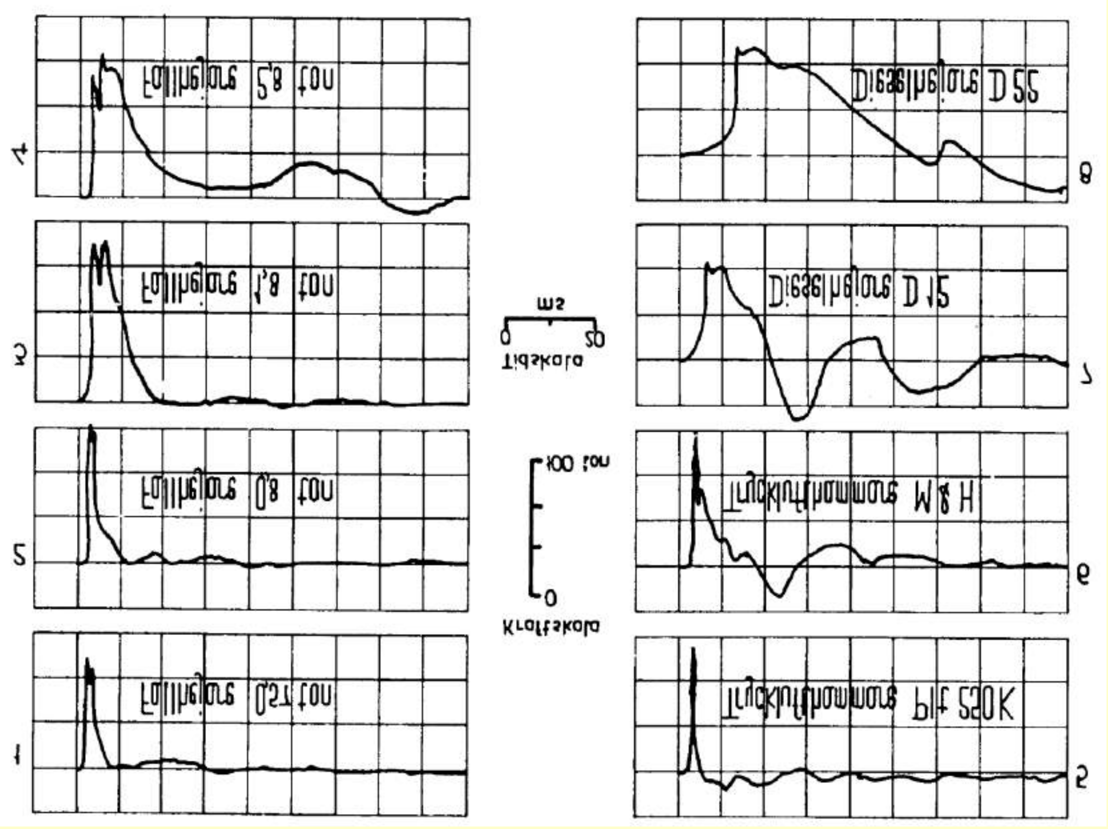
Front View

The 1959 Gubbero Tests, Göteborg, Sweden

Comparison of strain-waves from a pile driven with several different hammers.

Nos. 1 - 4 are drop hammers (0.6, 0.8, 1.8, and 2.8 tonne)
Nos. 5 - 6 are pneumatic hammers (Plt 290 K and M&H)
Nos. 7 - 8 are diesel hammers (D12 and D22)

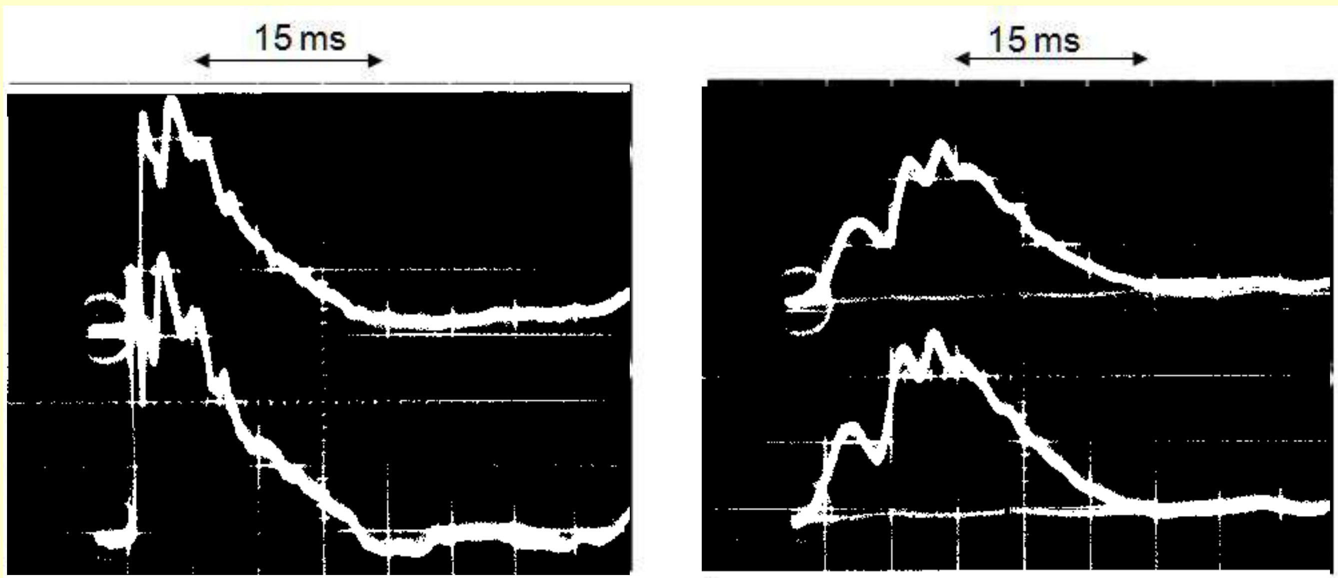
4 3 2 1



8 7 6 5

The 1959 Gubbero Tests, Göteborg, Sweden

Stress-waves (strain) measured at the head of a 260 mm diameter, 75 m long concrete pile before and after cushion change. Two blows recorded from each event.

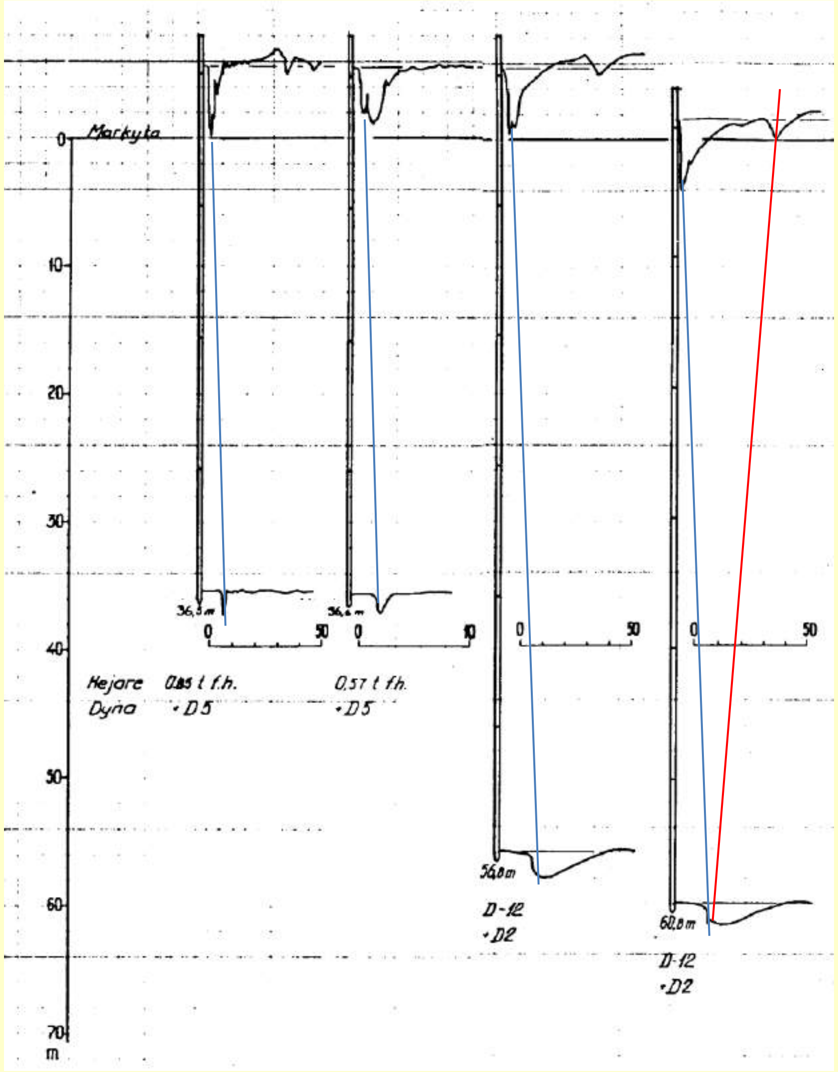


Before changing the pile cushion

After changing the pile cushion.

Stress-waves measured both at the pile head and at the pile toe.

(Different hammers, different pile lengths, and different cushions, but travel time is the same)



Some small steps toward theoretical analysis were indeed made by man, but the main result of the 1959 Gubbero tests was the realization of the complexity of pile driving.

Then, came the means to Analysis.

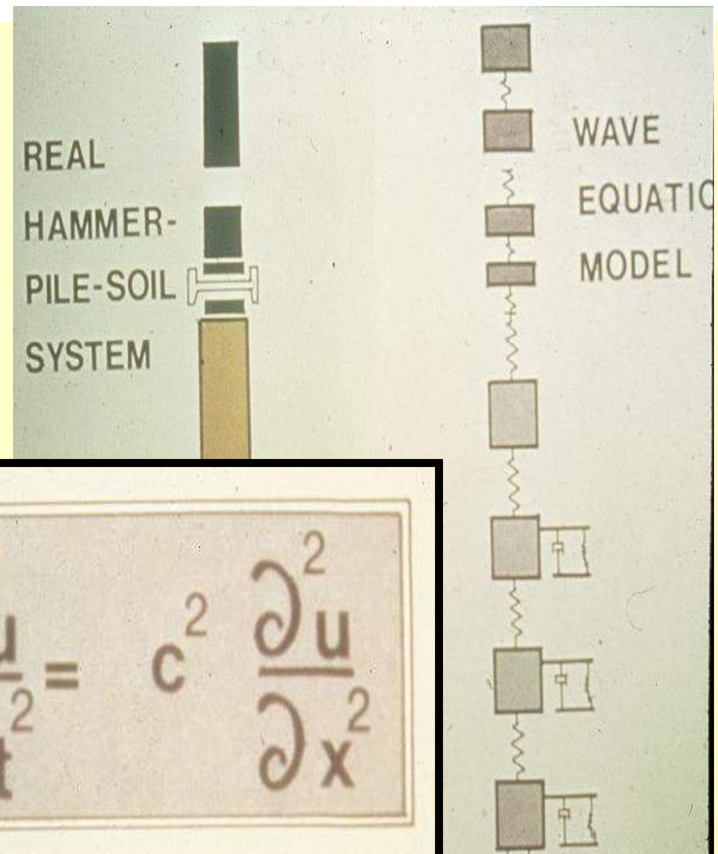
E.A.L. Smith (1960)

2574 August, 1960 SM 4

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

PILE-DRIVING ANALYSIS BY THE WAVE EQUATION

By E. A. L. Smith[1]

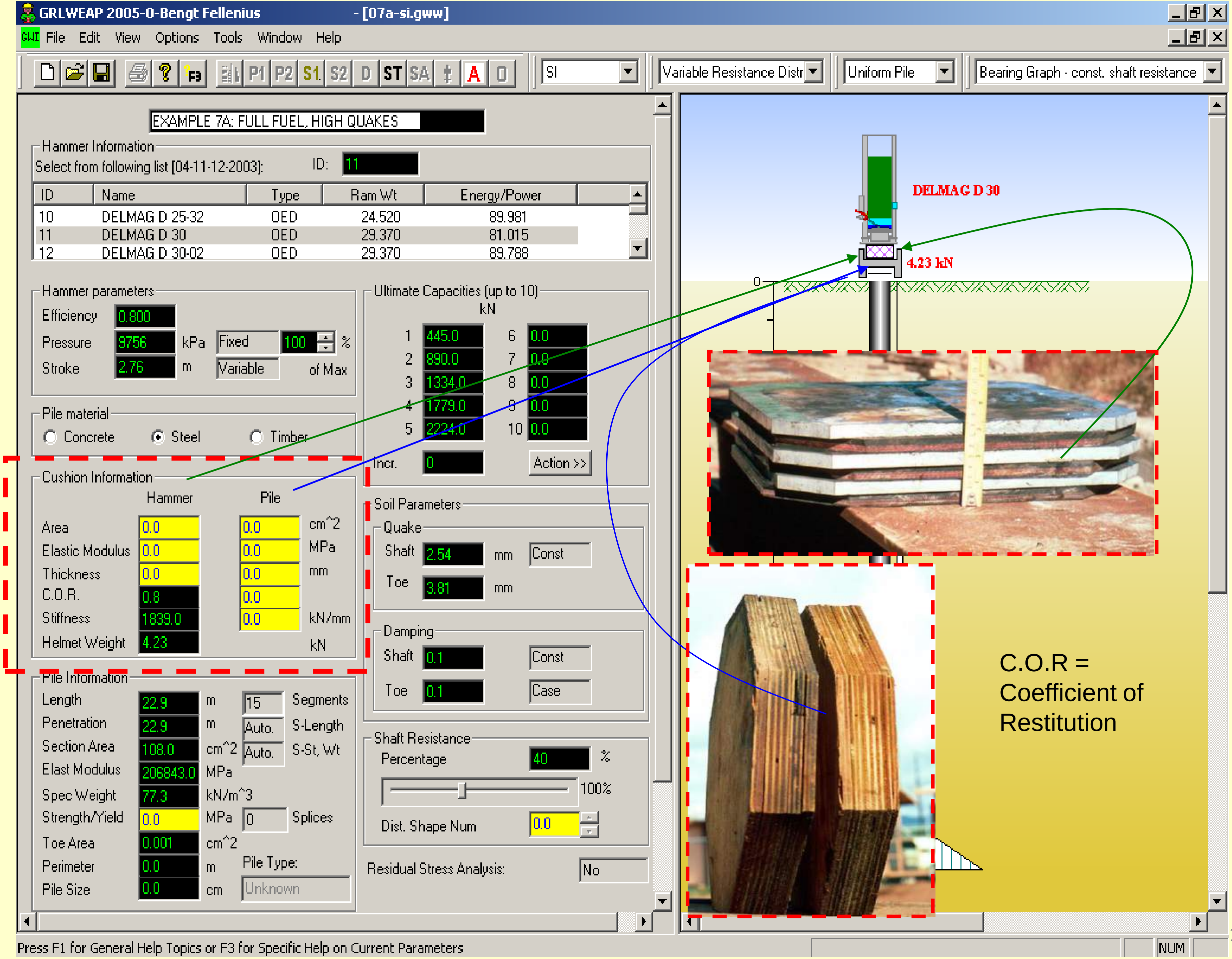


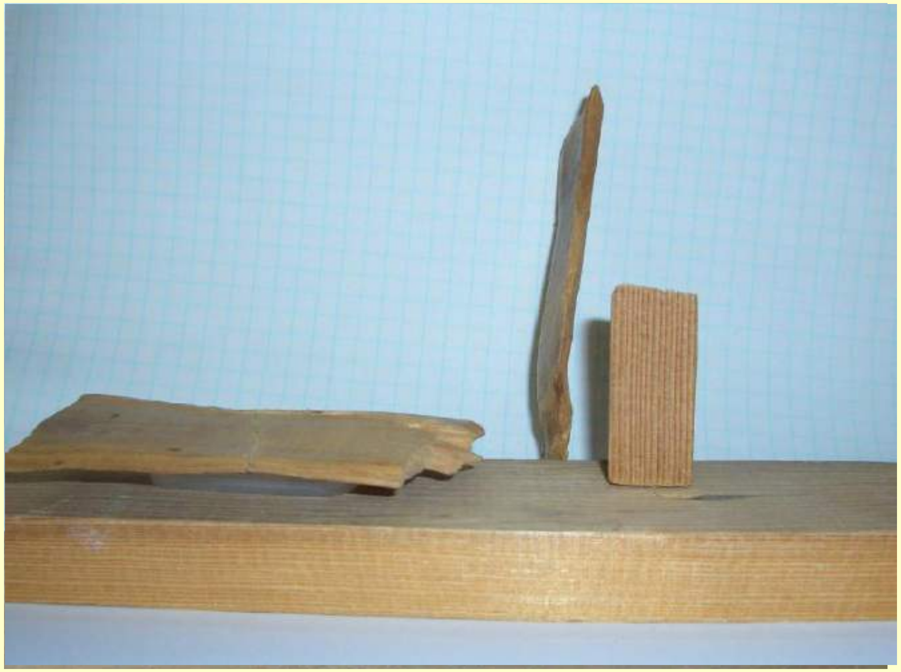
$$\frac{\partial^2 u}{\partial t^2} = c^2 \frac{\partial^2 u}{\partial x^2}$$

Slide by courtesy of
Mohamad Hussein, GRL

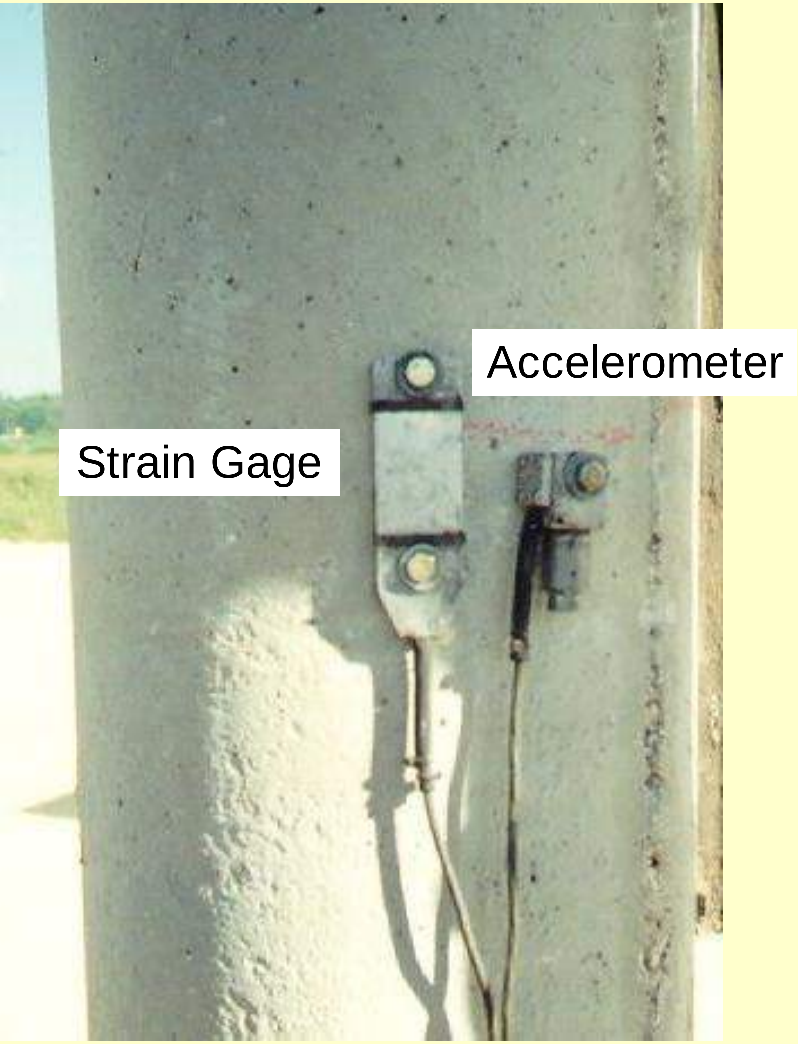
Tip-toeing through, missing the point

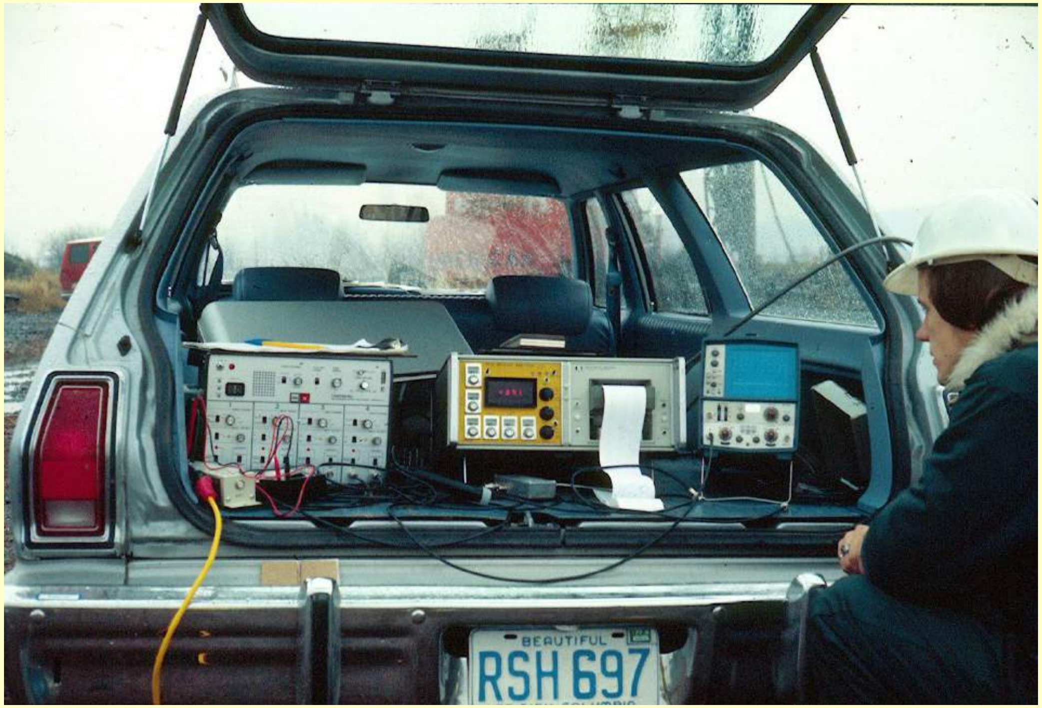
AN INTERESTING ASPECT OF THE TWO DAMPING FORMULATIONS FOR END RESISTANCE DEVELOPS WHEN THE STATIC POINT RESISTANCE IS ZERO: IN THE CASE FORMULATION, THE DYNAMIC TOE RESISTANCE IS FINITE (BECAUSE THE PILE TIP HAS VELOCITY), BUT IN THE SMITH FORMULATION IT IS ZERO.


GRLWEAP 2005-0-Bengt Fellenius - [07a-si.gww]
File Edit View Options Tools Window Help
SI
Variable Resistance Distr
Uniform Pile
Bearing Graph - const. shaft resistance
EXAMPLE 7A: FULL FUEL, HIGH QUAKES
Hammer Information
Select from following list [04-11-12-2003]: ID: 11
ID Name Type Ram Wt Energy/Power
10 DELMAG D 25-32 OED 24.520 89.981
11 DELMAG D 30 OED 29.370 81.015
12 DELMAG D 30-02 OED 29.370 89.788
Hammer parameters
Efficiency 0.800
Pressure 9756 kPa Fixed 100 %
Stroke 2.76 m Variable of Max
Pile material
Concrete Steel Timber
Cushion Information
Hammer Pile
Area 0.0 0.0 cm^2
Elastic Modulus 0.0 0.0 MPa
Thickness 0.0 0.0 mm
C.O.R. 0.8 0.0
Stiffness 1839.0 0.0 kN/mm
Helmet Weight 4.23 kN
Pile Information
Length 22.9 m 15 Segments
Penetration 22.9 m Auto. S-Length
Section Area 108.0 cm^2 Auto. S-St. Wt
Elast Modulus 206843.0 MPa
Spec Weight 77.3 kN/m^3
Strength/Yield 0.0 MPa 0 Splices
Toe Area 0.001 cm^2
Perimeter 0.0 m Pile Type:
Pile Size 0.0 cm Unknown
Ultimate Capacities (up to 10)
kN
1 445.0 6 0.0
2 890.0 7 0.0
3 1334.0 8 0.0
4 1779.0 9 0.0
5 2224.0 10 0.0
Incr. 0 Action >>
Soil Parameters
Quake
Shaft 2.54 mm Const
Toe 3.81 mm
Damping
Shaft 0.1 Const
Toe 0.1 Case
Shaft Resistance
Percentage 40 %
100%
Dist. Shape Num 0.0
Residual Stress Analysis: No
Press F1 for General Help Topics or F3 for Specific Help on Current Parameters
NUM
DELMAG D 30
4.23 kN
0
C.O.R =
Coefficient of
Restitution



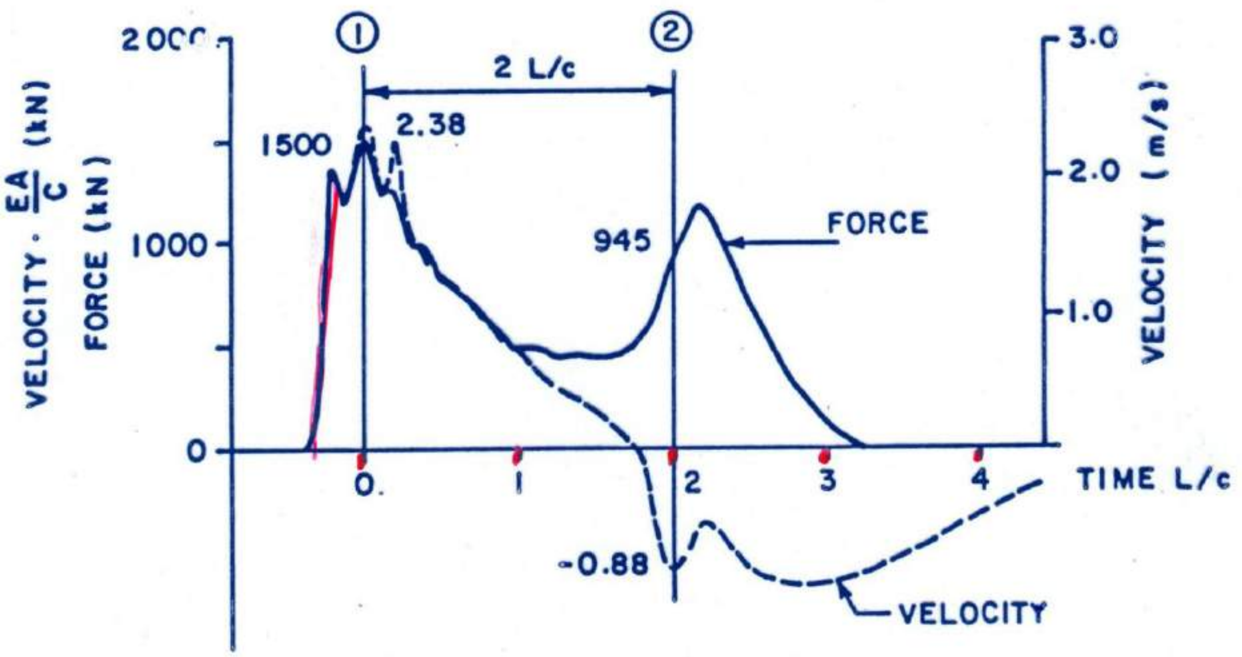
Along with WEAP came the Pile Driving Analyzer, the PDA!





PDA set-up in 1977

With the break-through use of both strain-gages and accelerometers.



$$\text{MOBILIZED RESISTANCE} = \frac{F_1 + F_2}{2} + \frac{EA}{C} \cdot \frac{1}{2}(V_1 - V_2)$$

Dynamic and Static

Static Resistance includes adjustment by a damping factor

$$= \frac{1500 + 945}{2} + 638 \cdot \frac{1}{2}(2.38 + 0.88)$$

$$= 2260 \text{ kN}$$

The Case Method Estimate — CMES-RSP

THE PIONEERS

George Goble 1975

Photo courtesy of Pete Bentley

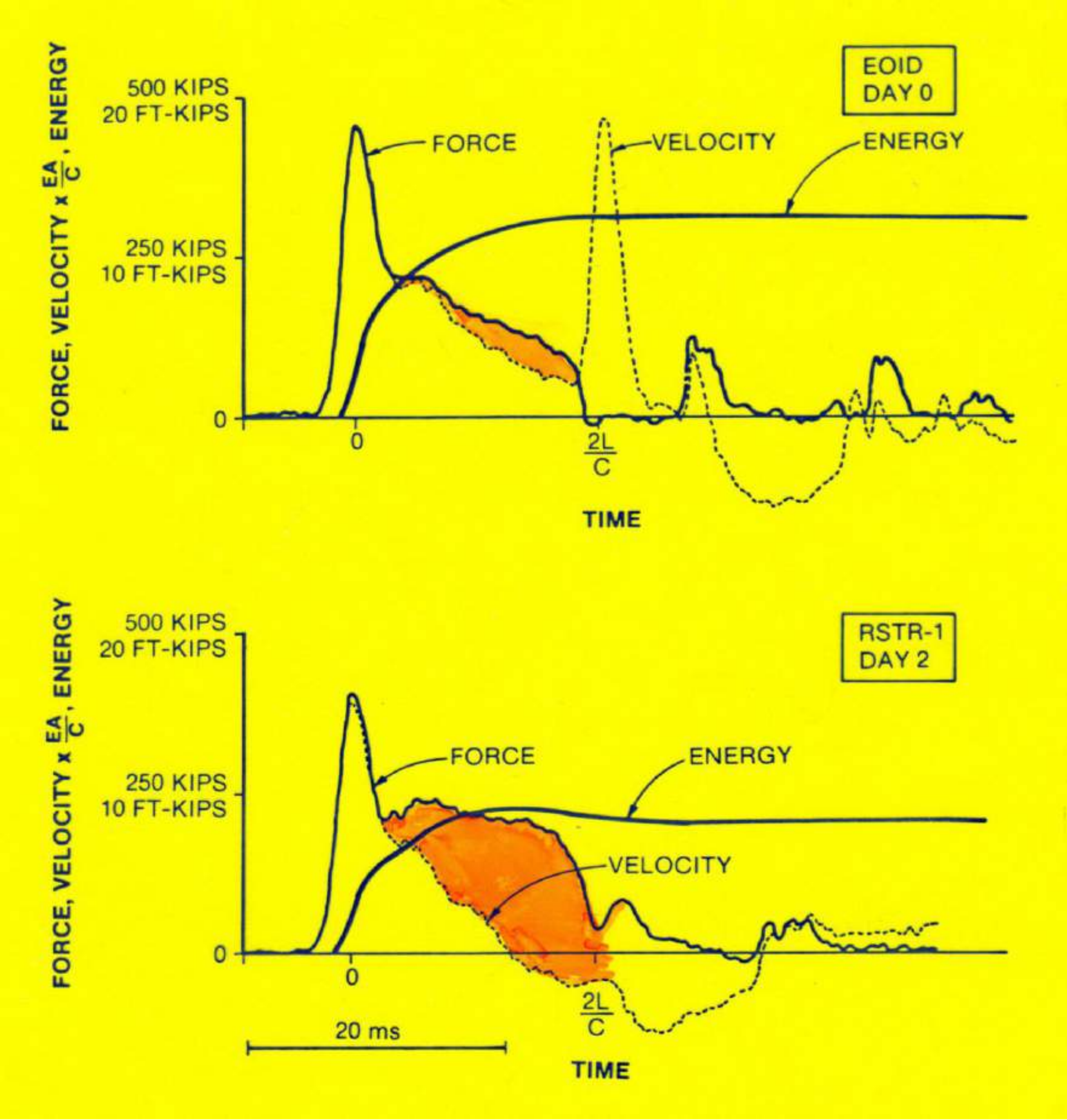
Frank Rausche 1975

Photo courtesy of Pete Bentley

Frank Rausche and Garland Likins 1975

Photo courtesy of Pete Bentley

A couple of wave-trace graphs from mid 1970s



Initial Driving,

EOID

PRES

1 bl/in

Restrike Driving,

RSTR-1

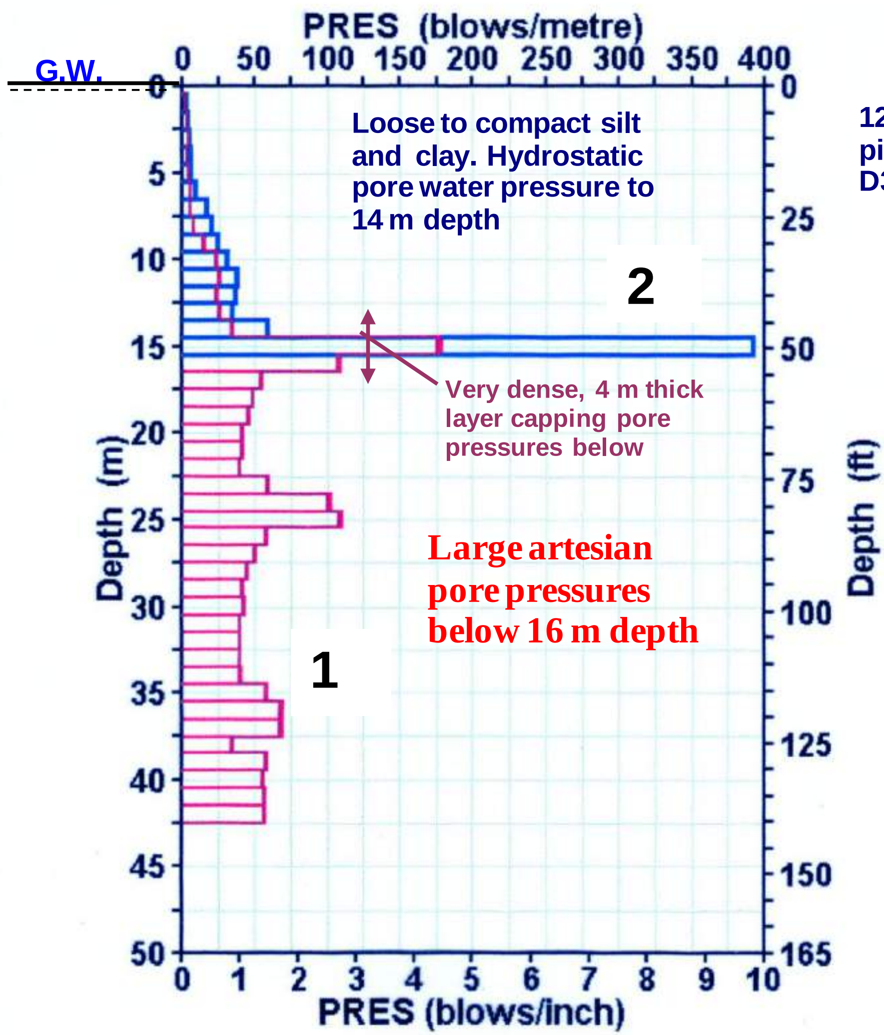
BOR-1

PRES

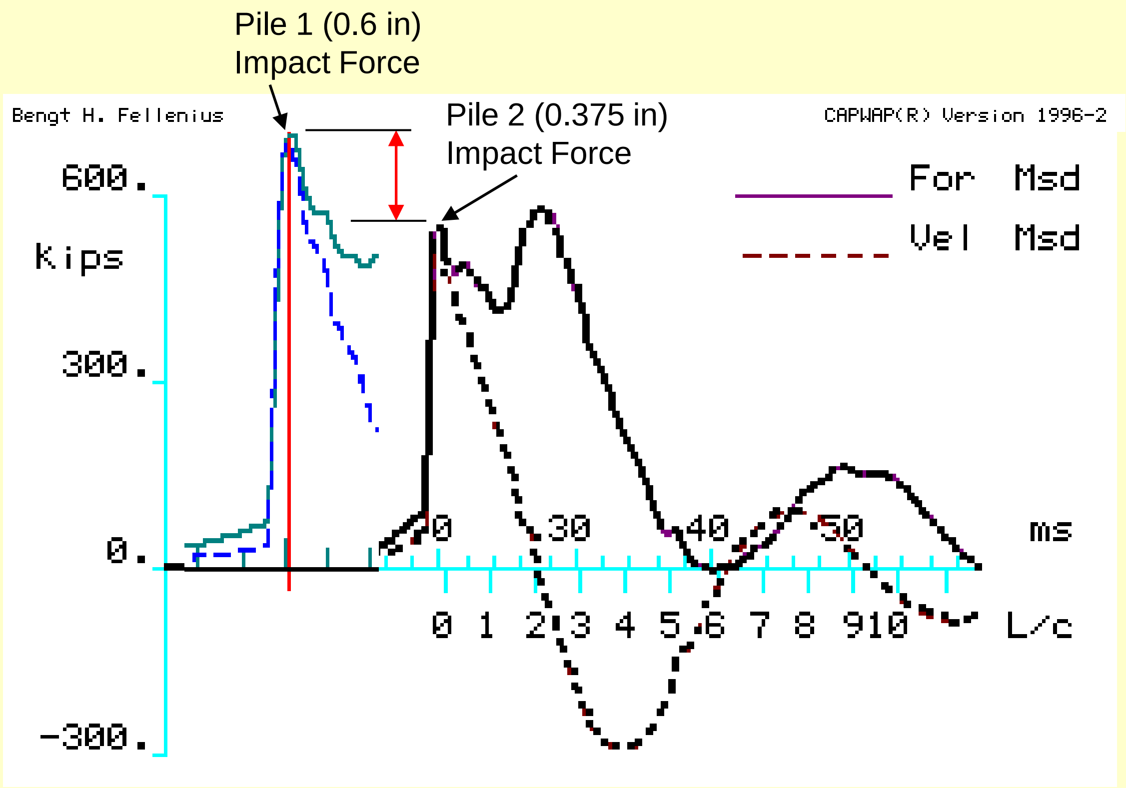
3 bl/in

A project in Salt Lake City in late 1980

Piles (similar to Pile 1, below) were driven well, but then, suddenly, they could not be driven deeper than about 15 m (e.g. Pile 2, below).



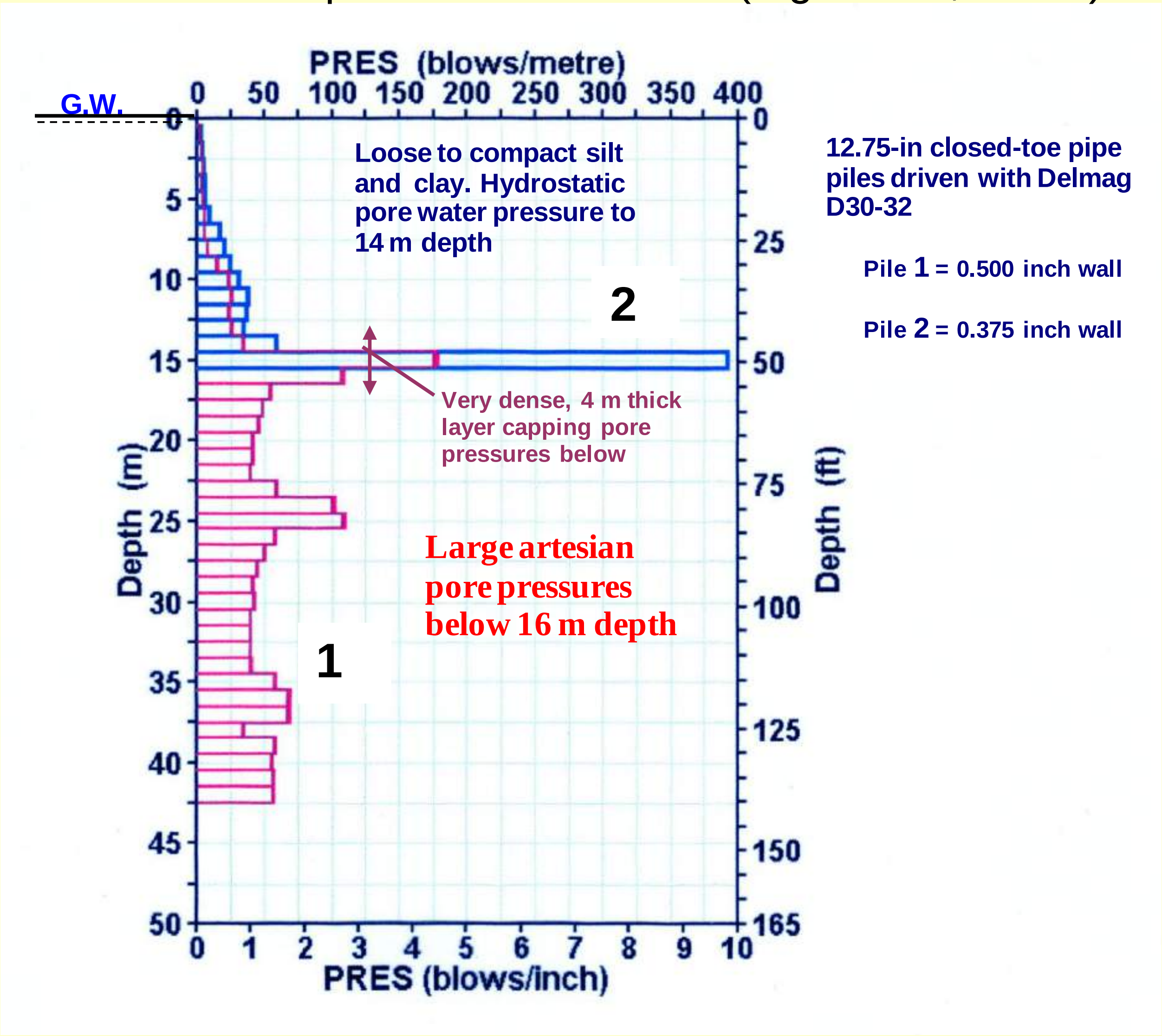
Did the pile driving hammer cease to work properly for the No. 2 piles? Or, was the difference in driving response between Piles 1 and 2 due to "changed conditions"? If the latter, the Contractor could recoup his costs.



The impact stress and stress-wave length were about the same for the piles, but the impact force is stress times area and the area was larger for Pile 1. Force is what moves a pile against the soil.

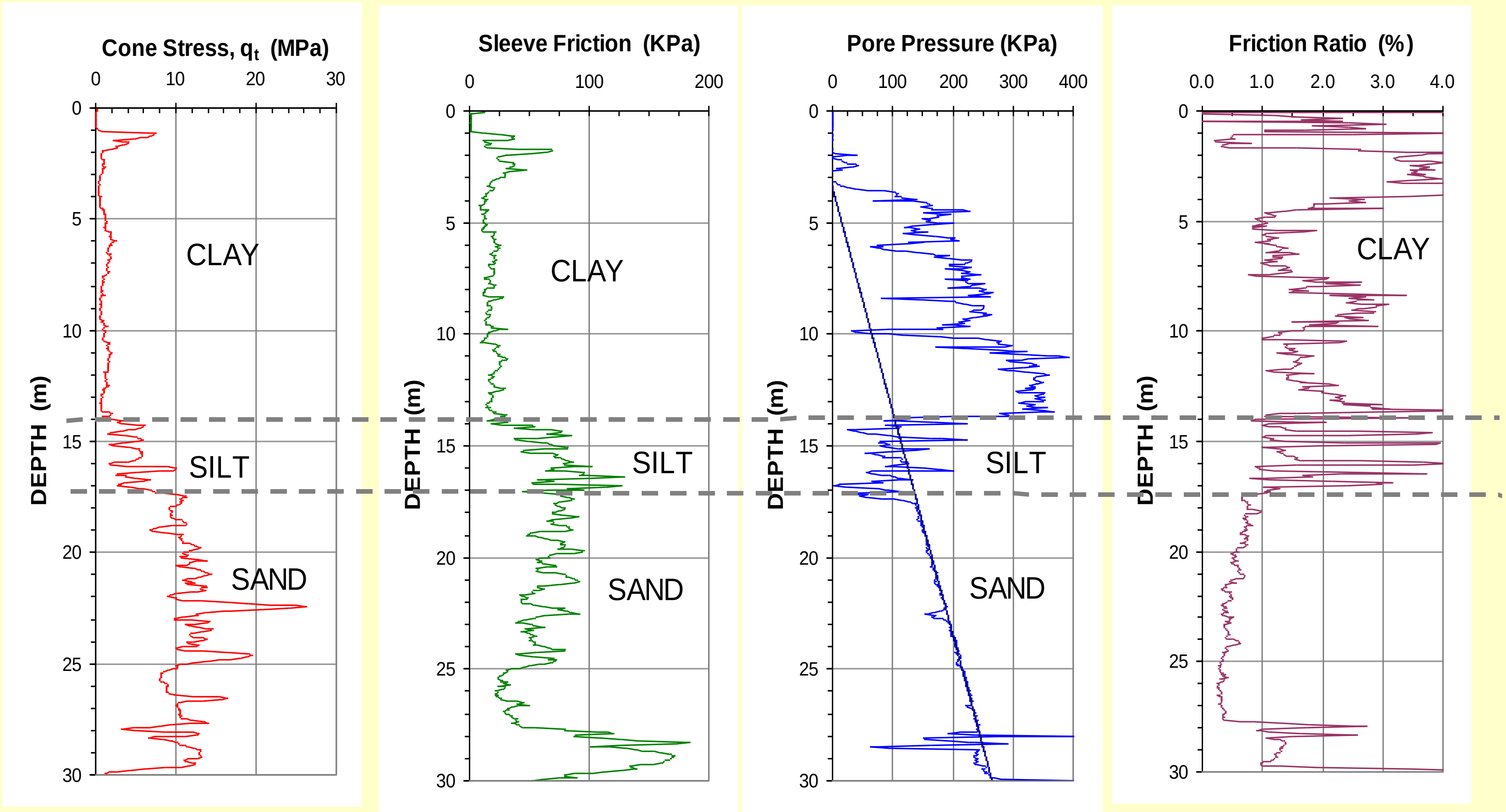
A project in Salt Lake City in late 1980

Piles (similar to Pile 1, below) were driven well, but then, suddenly, they could not be driven deeper than about 15 m (e.g. Pile 2, below).



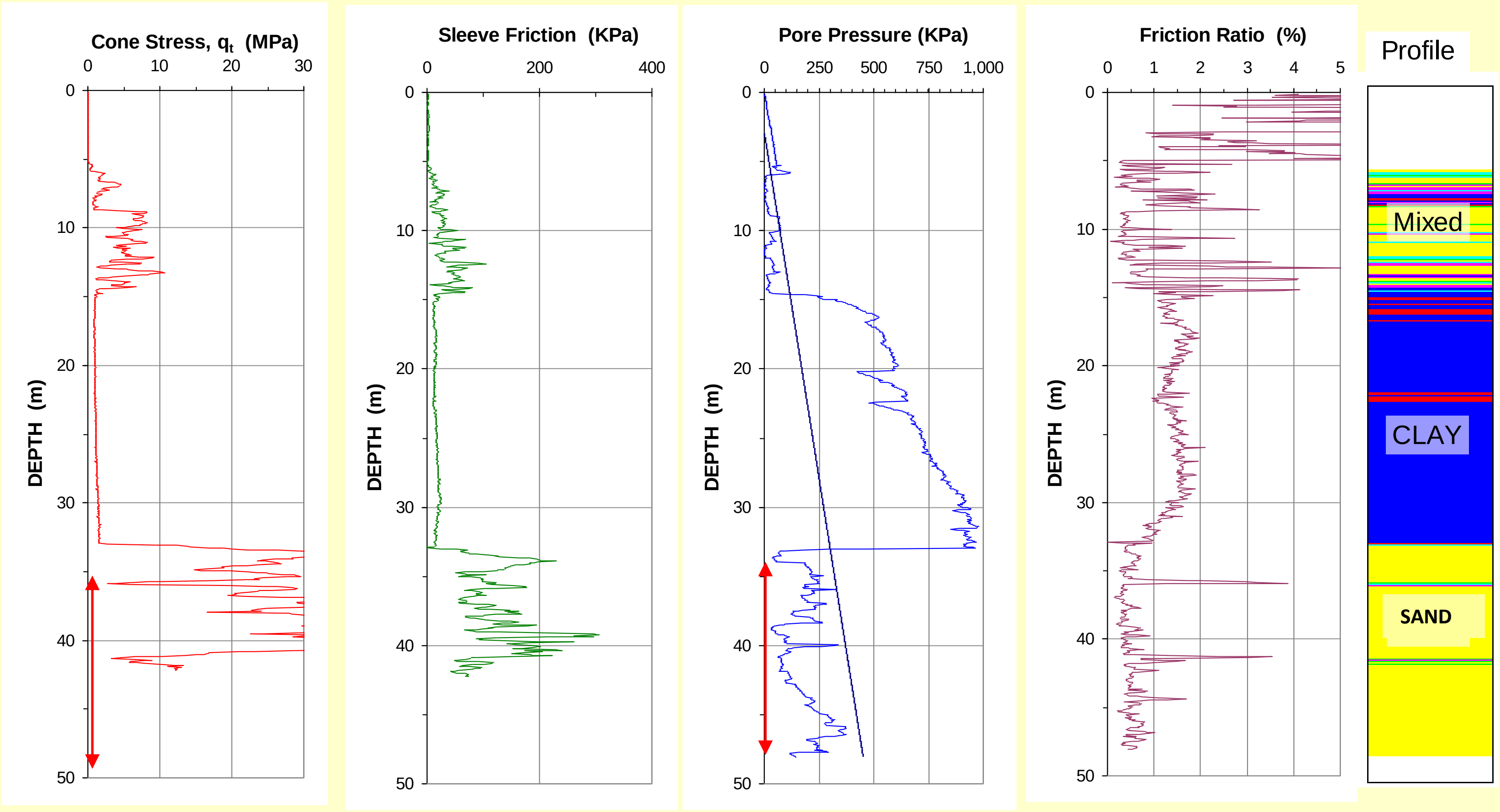
Repeated slide

Example 1 of a CPTU sounding from a river estuary delta (Nakdong River, Pusan, Korea)



CPTU diagrams from a sounding in non-dilatant sand

Example 2 of a CPTU sounding from a river estuary delta (Nakdong River, Pusan, Korea)



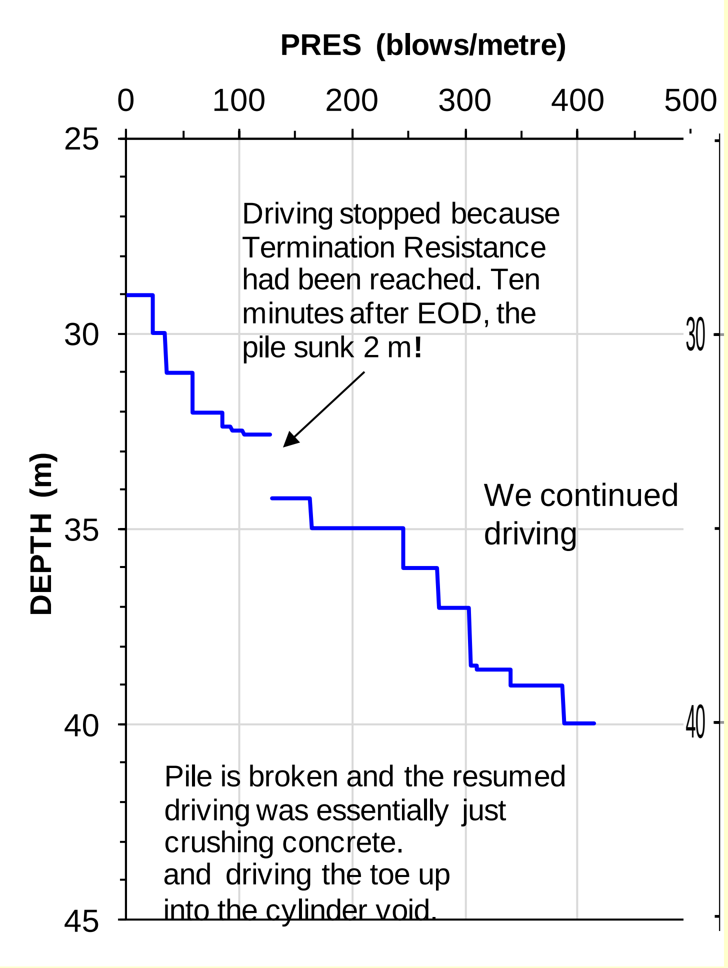
The sand layer between 6 m and 8 m depth is potentially liquefiable.

The clay layer is very soft.

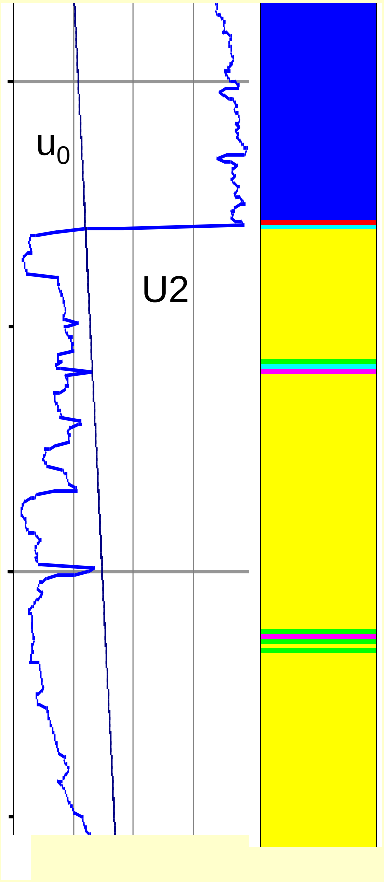
The sand below 34 m depth is very dense and dilatant, i.e., overconsolidated and providing sudden large penetration resistance to driven piles and relaxation problems.

Driving a 600 mm diameter, 45 m long, closed-toe, cylinder pile at the site

"The Pile that Ate Its Toe!"

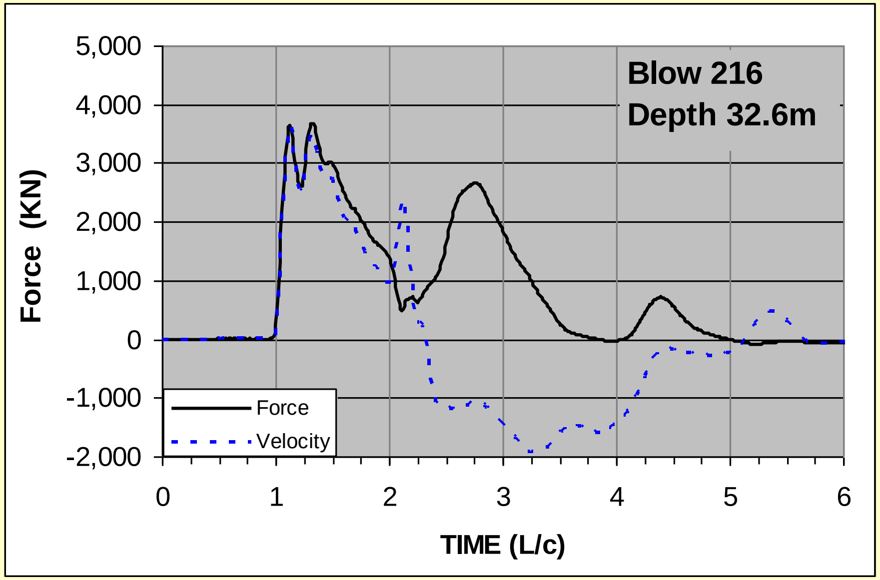


Portion of the CPTU U2-diagram

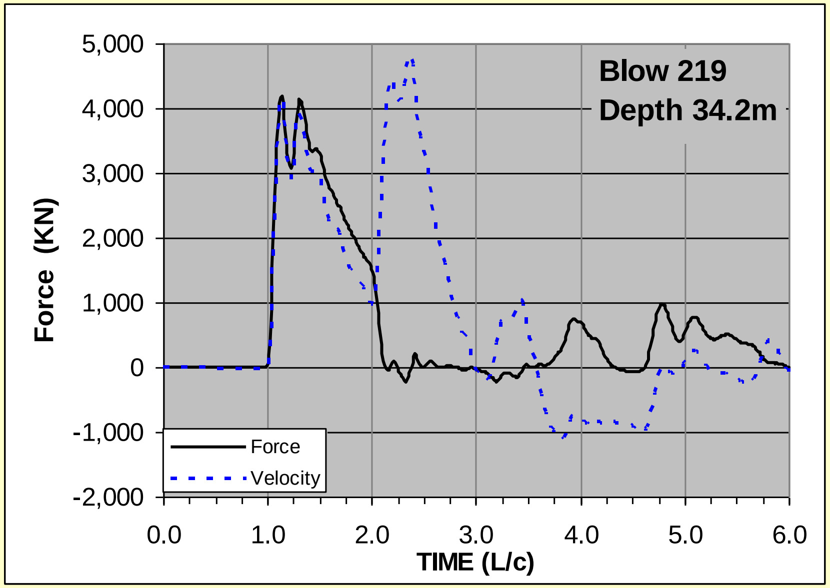


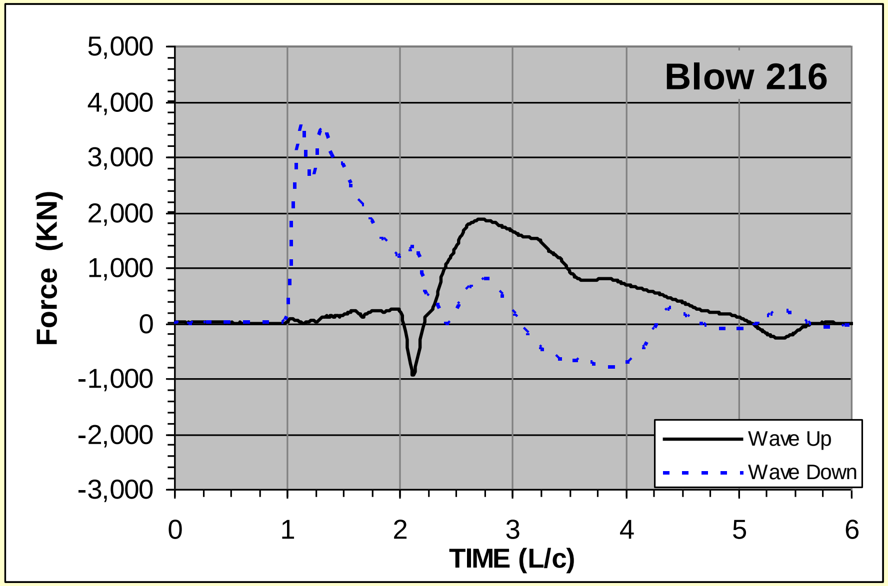
Wave Traces from the event

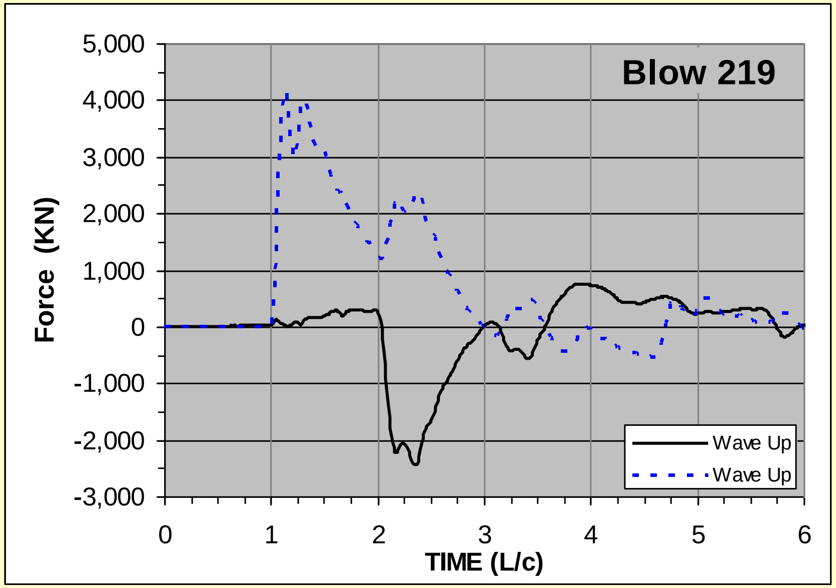
Three blows before the event occurred



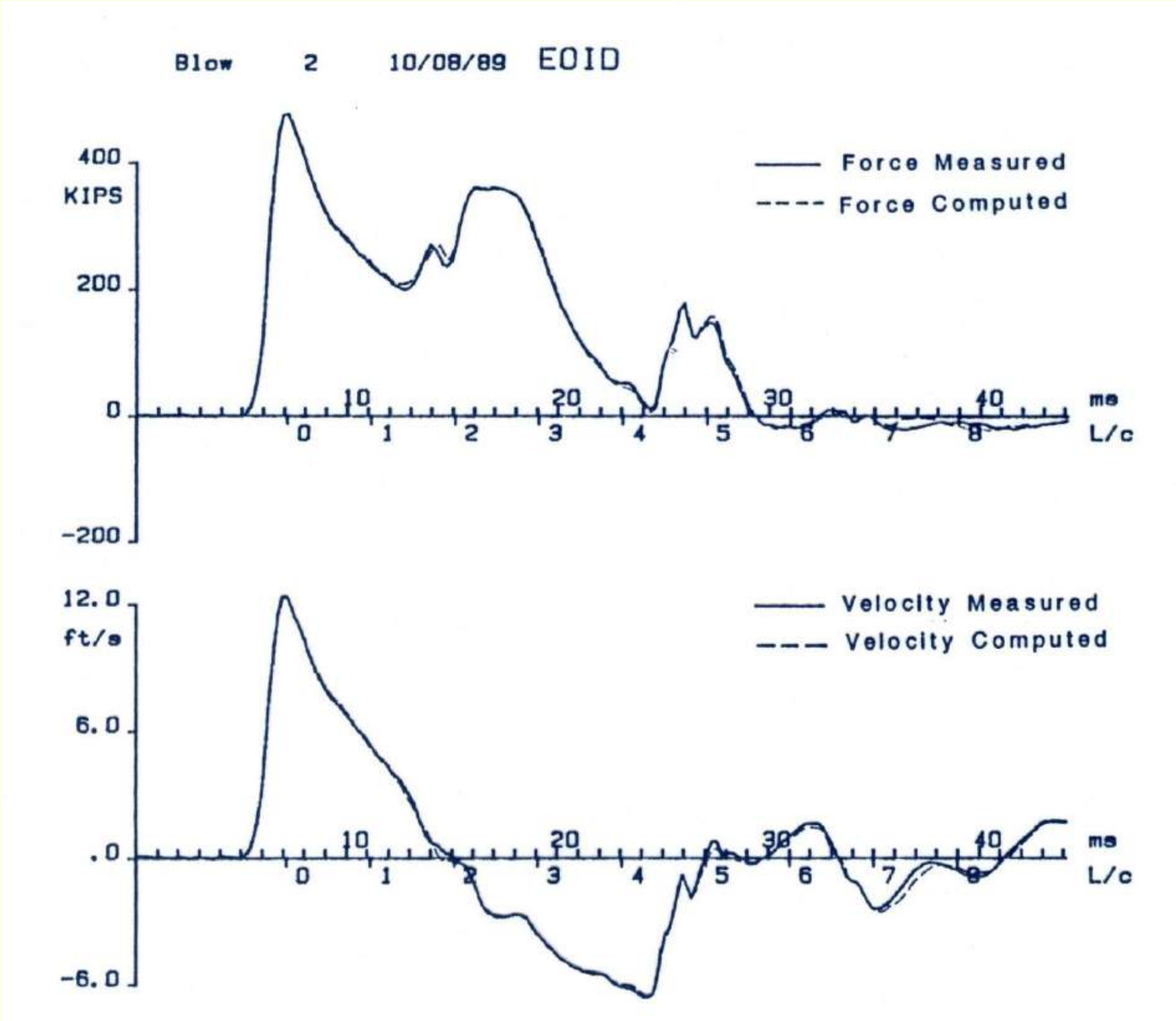
First blow after the event







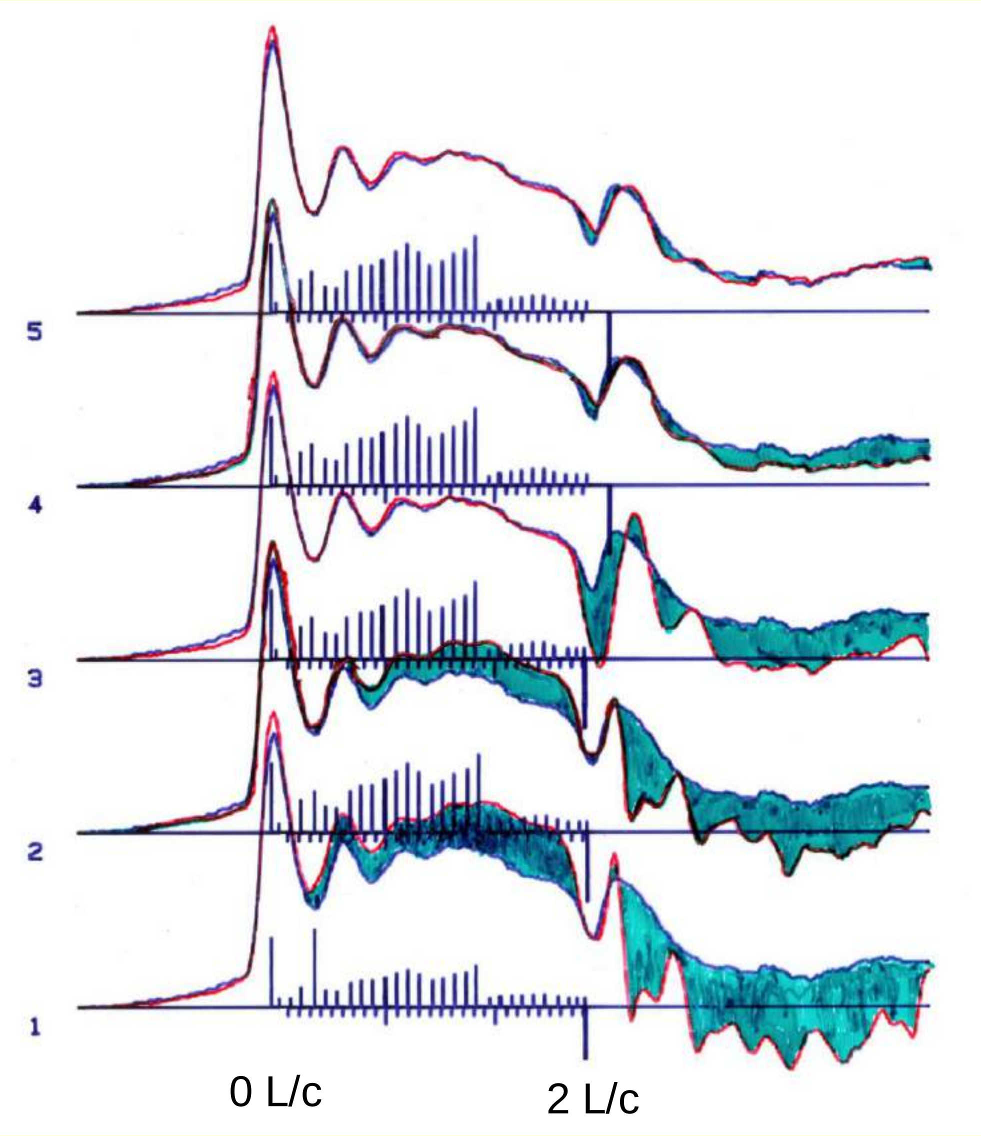
C A P W A P   Analysis



Force-match and Velocity match

(Hannigan 1990)

C A P W A P   Analysis Process

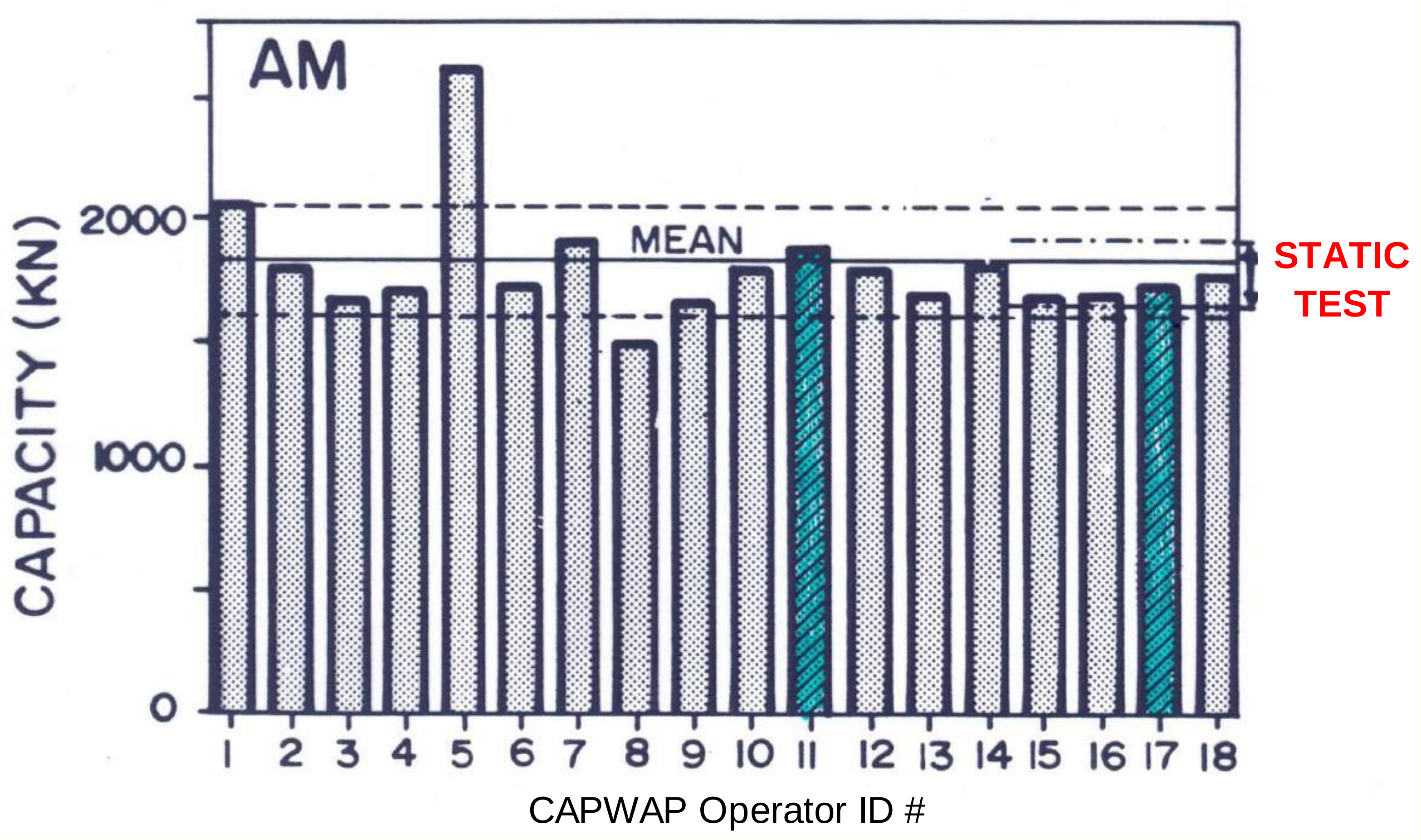


Example of force-match iterations

(Hannigan 1990)

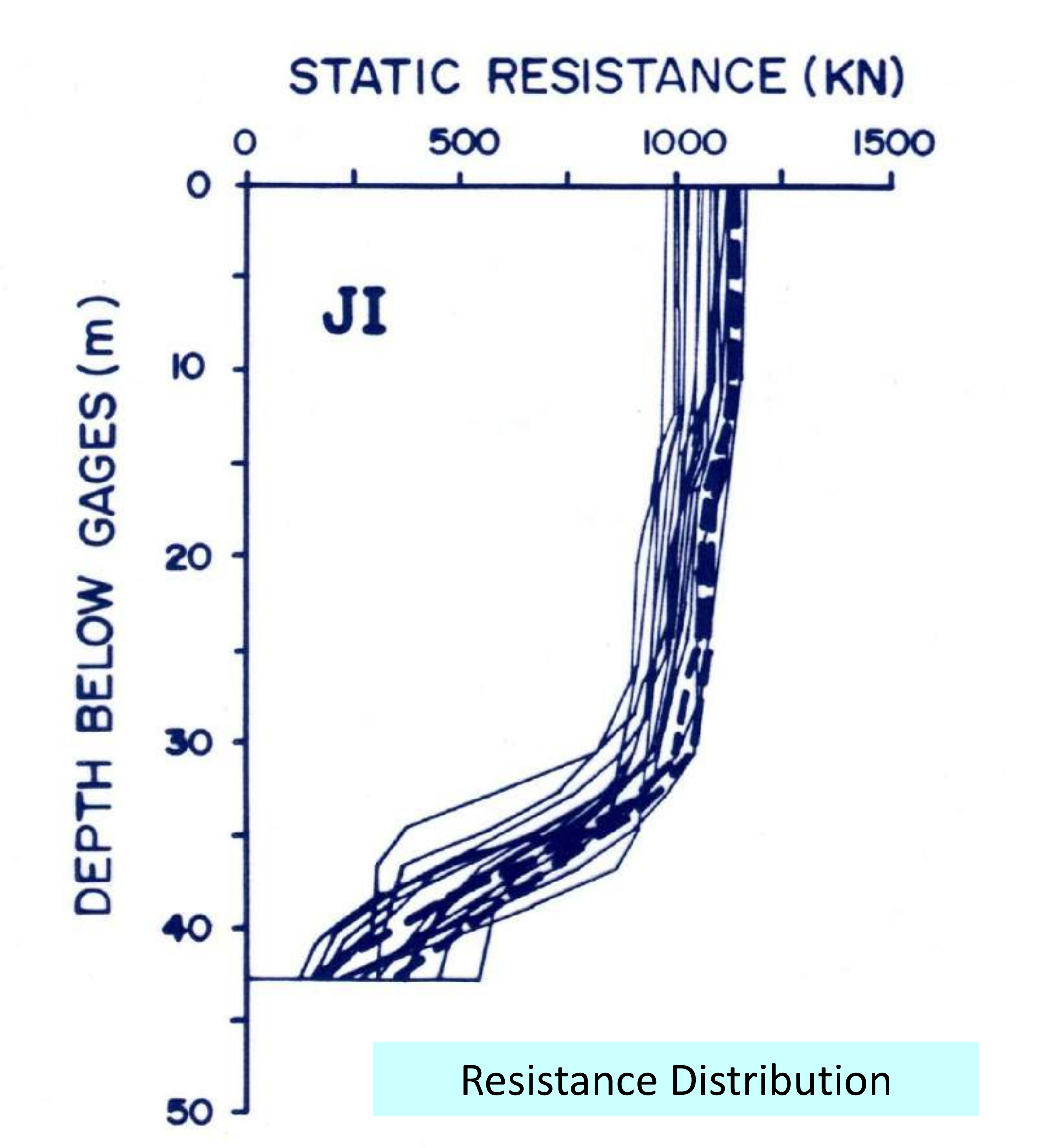
Back in the early days, we all wondered

(1) how true was the CAPWAP-determined capacity to that determined from a static loading test and
(2) how consistent would the capacities be between analyses performed by different operators?



Compilation of CAPWAPs by different operators — AM site
(Fellenius 1988)

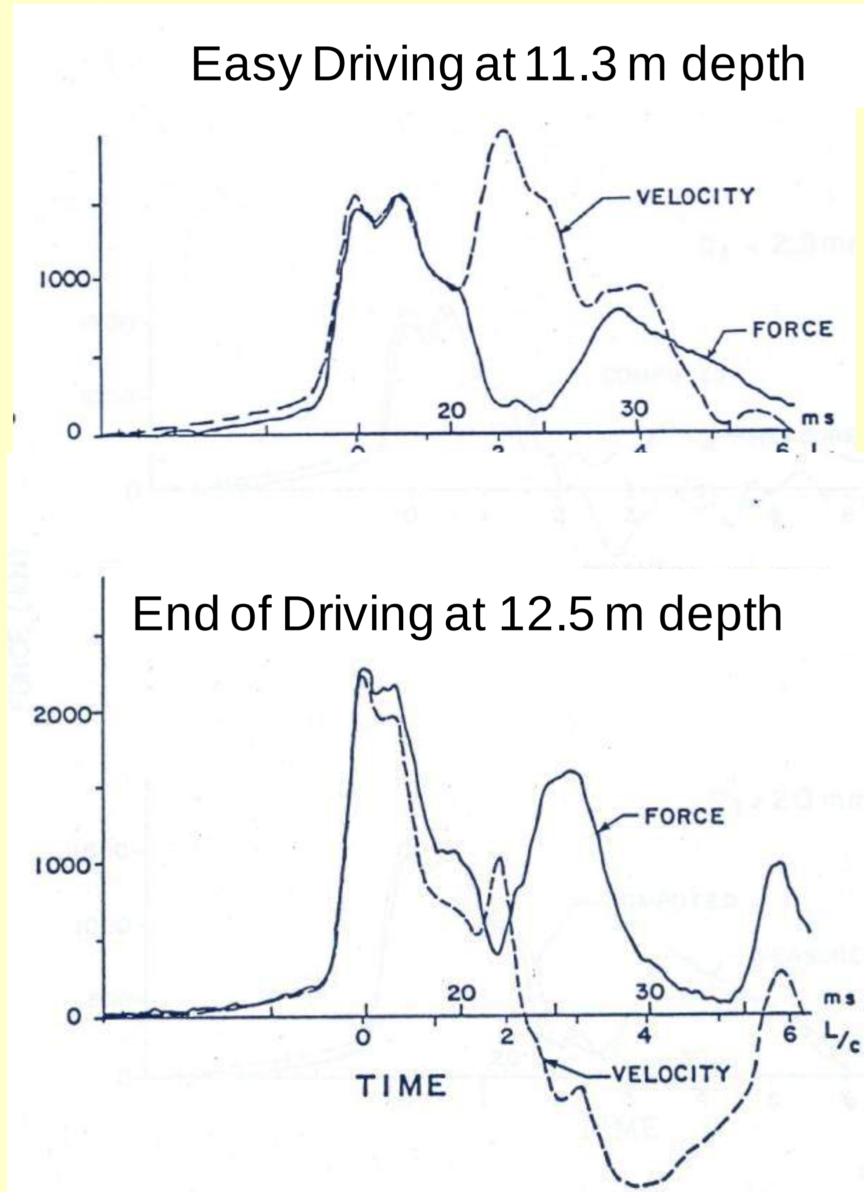
Compilation of CAPWAPs by different operators — JI site
(Fellenius 1988)



Compilation of CAPWAPs by different operators — JI site
(Fellenius 1988)

QUAKE!

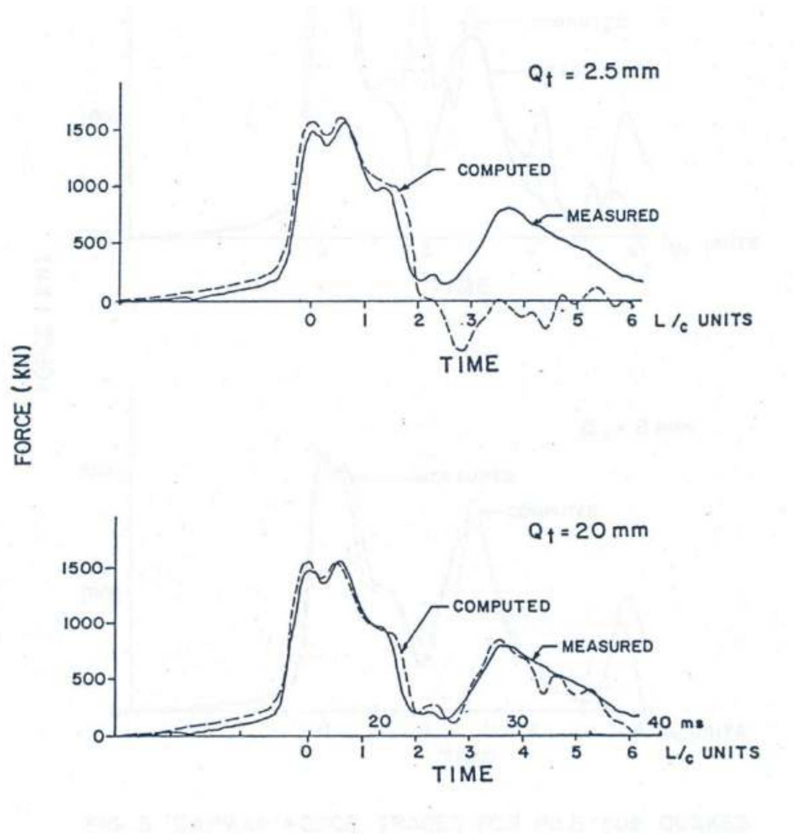
Wave traces from one of twenty-four 305 mm square, precast concrete piles were driven through about 11 m of clay deposit into **<u>dense clayey silty</u>** glacial till.



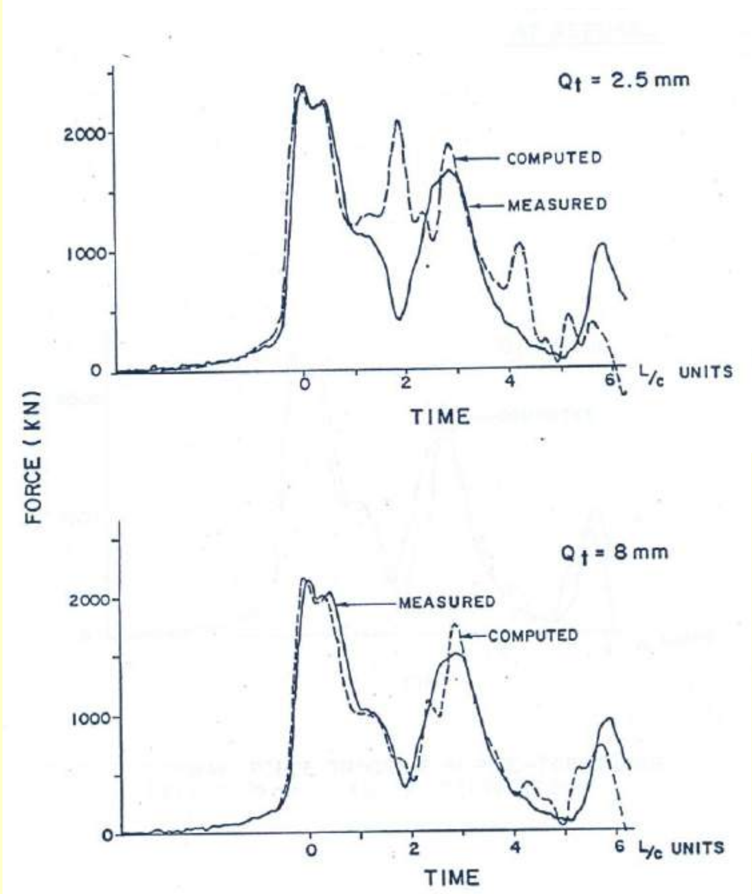
1st Stress-wave Conference; Authier and Fellenius (1980), reporting analysis produced by Frank Rausche, GRL.

CAPWAP Matches

Easy Driving at 11.3 m depth

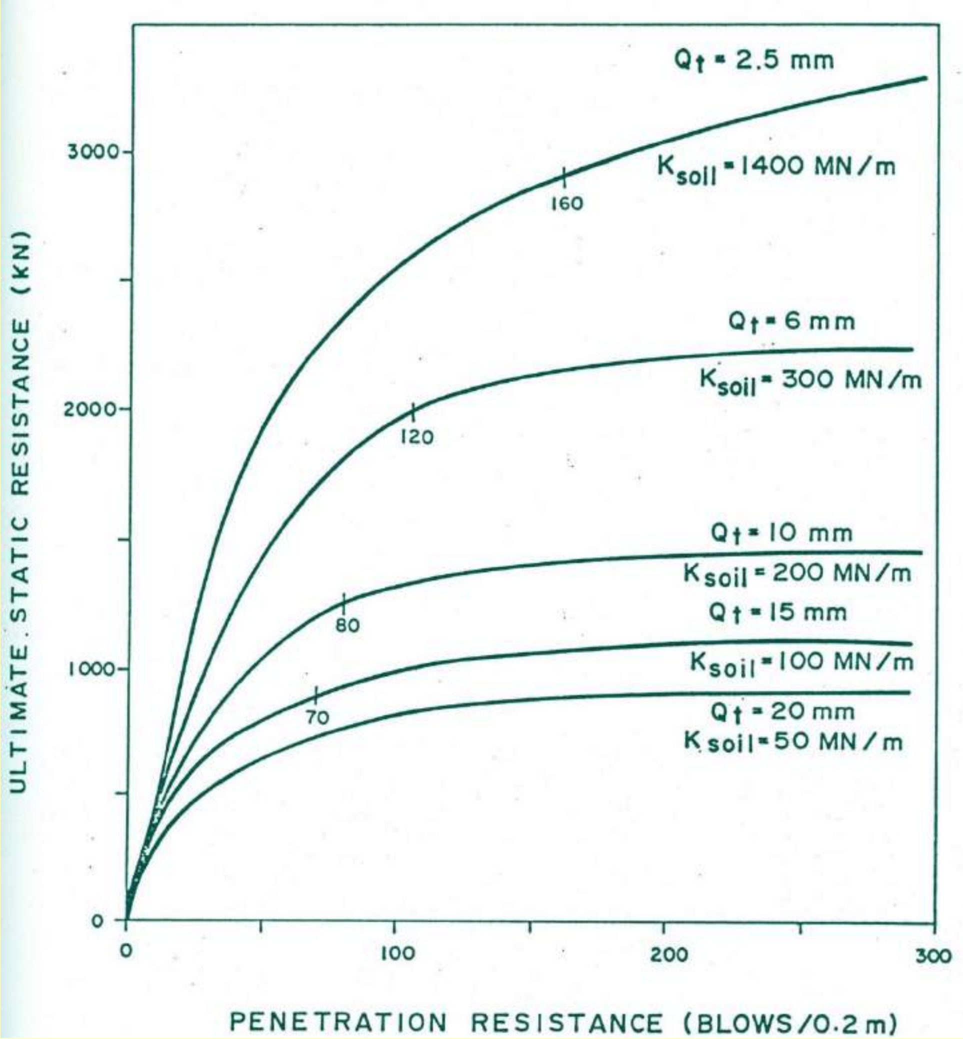


End of Driving at 12.5 m depth



1st Stress-wave Conference; Authier and Fellenius (1980), reporting analysis produced by Frank Rausche, GRL.

Bearing Graphs from WEAP Simulations assuming different quake magnitudes



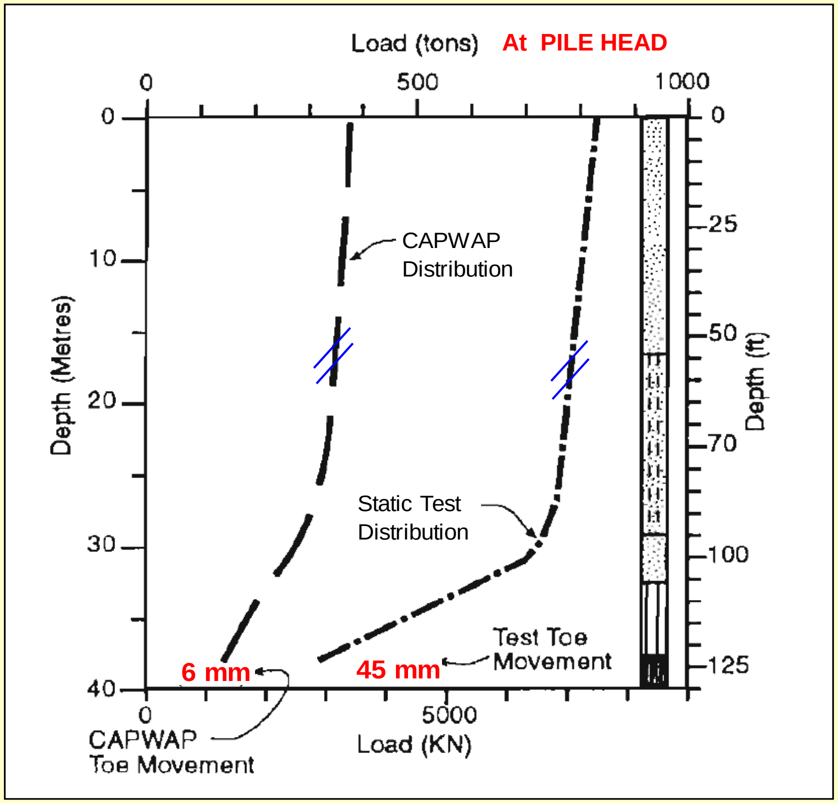
1st Stress-wave Conference; Fellenius and Authier (1980)

Stress-wave Conferences 1980 - 2008

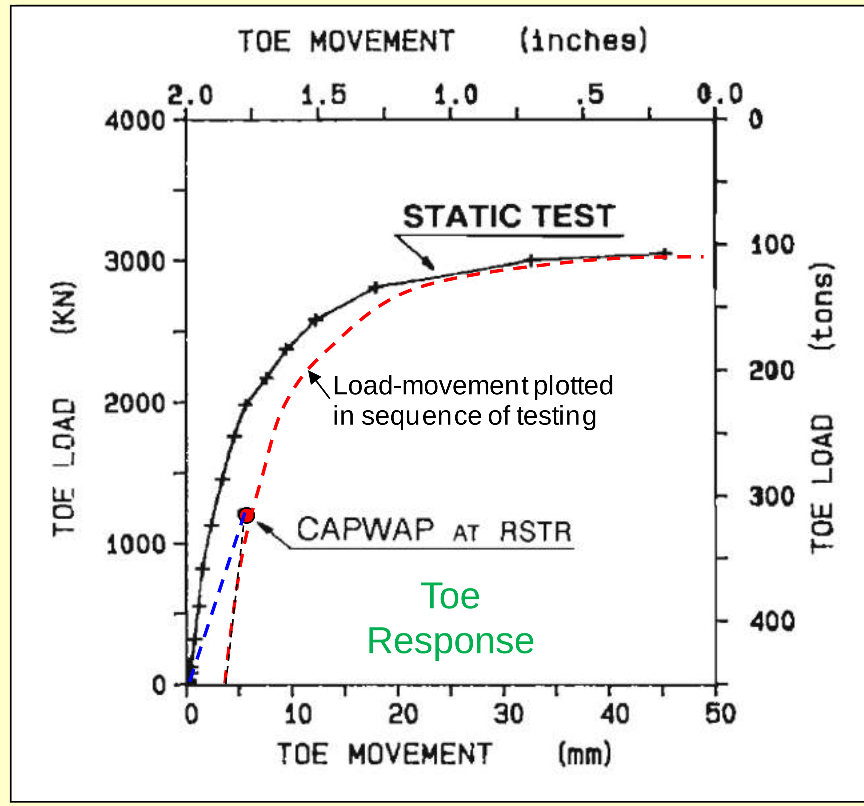
Not shown: the 9th Kanazawa 2012.

Dynamic and static tests on a 20-inch diameter, 41 m long prestressed pile driven for Alesea Bay Bridge foundations

CAPWAP-determined capacity was 3,600 kN, but static loading test gave 8,000+ kN.
Yet, I consider the two tests to agree perfectly.



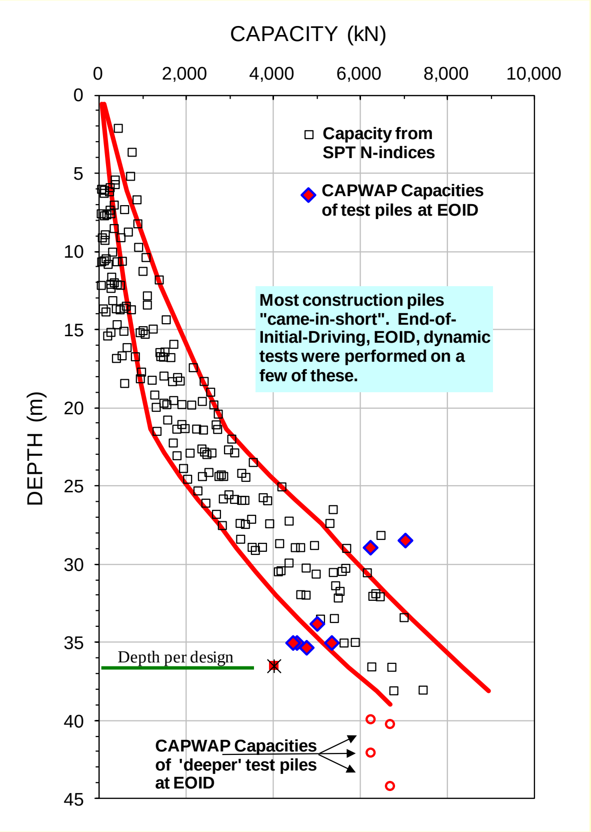
Load distribution



Toe load-movement response

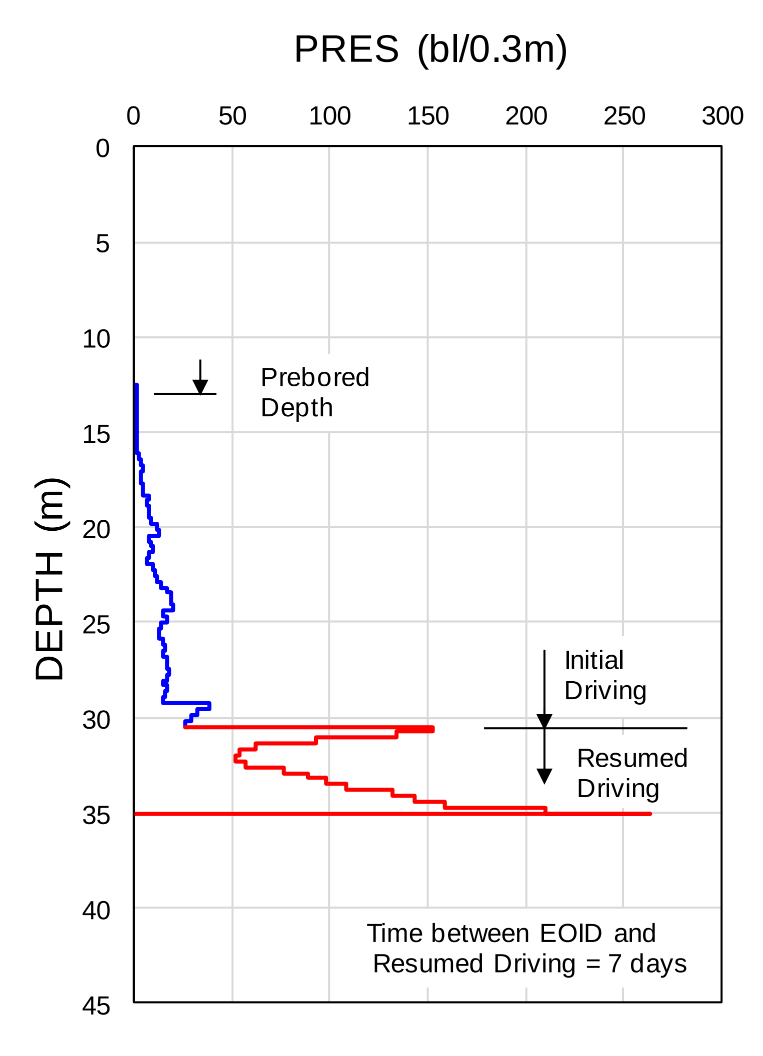
Fellenius and Riker, 1992

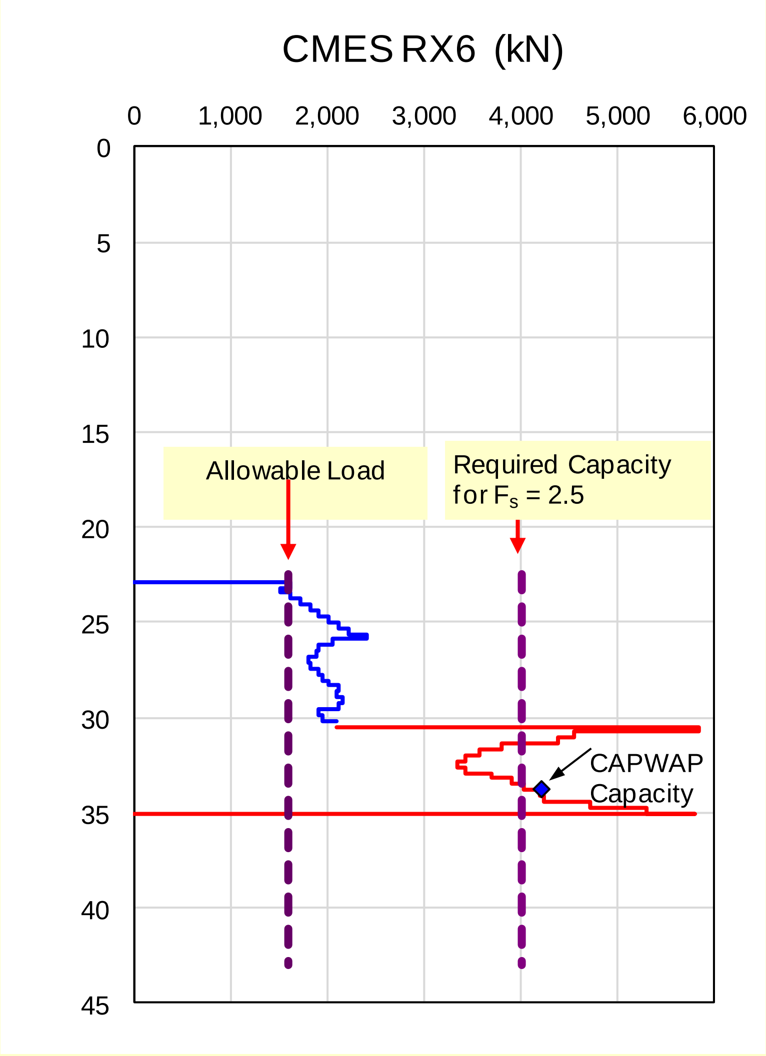
Design of a piled foundation LNG facility involving ≈1,000, 120 ft long, 24-inch prestressed piles



At End-of-Driving, EOD, the construction piles had been 'hammered' in excess of 100 bl/ft for several feet!

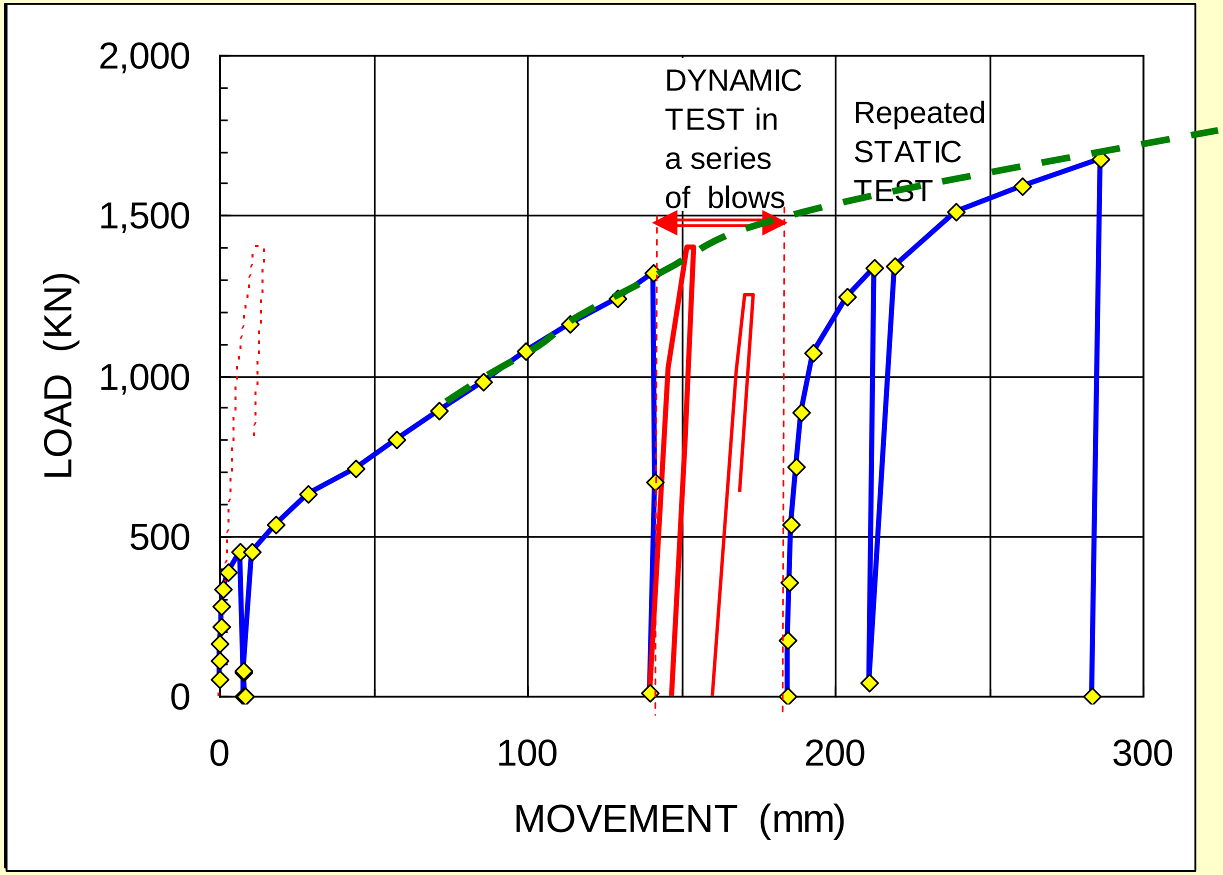
Of course, "set-up" was considered to be just an additional "conservative benefit".





You can lead the horse to water … !

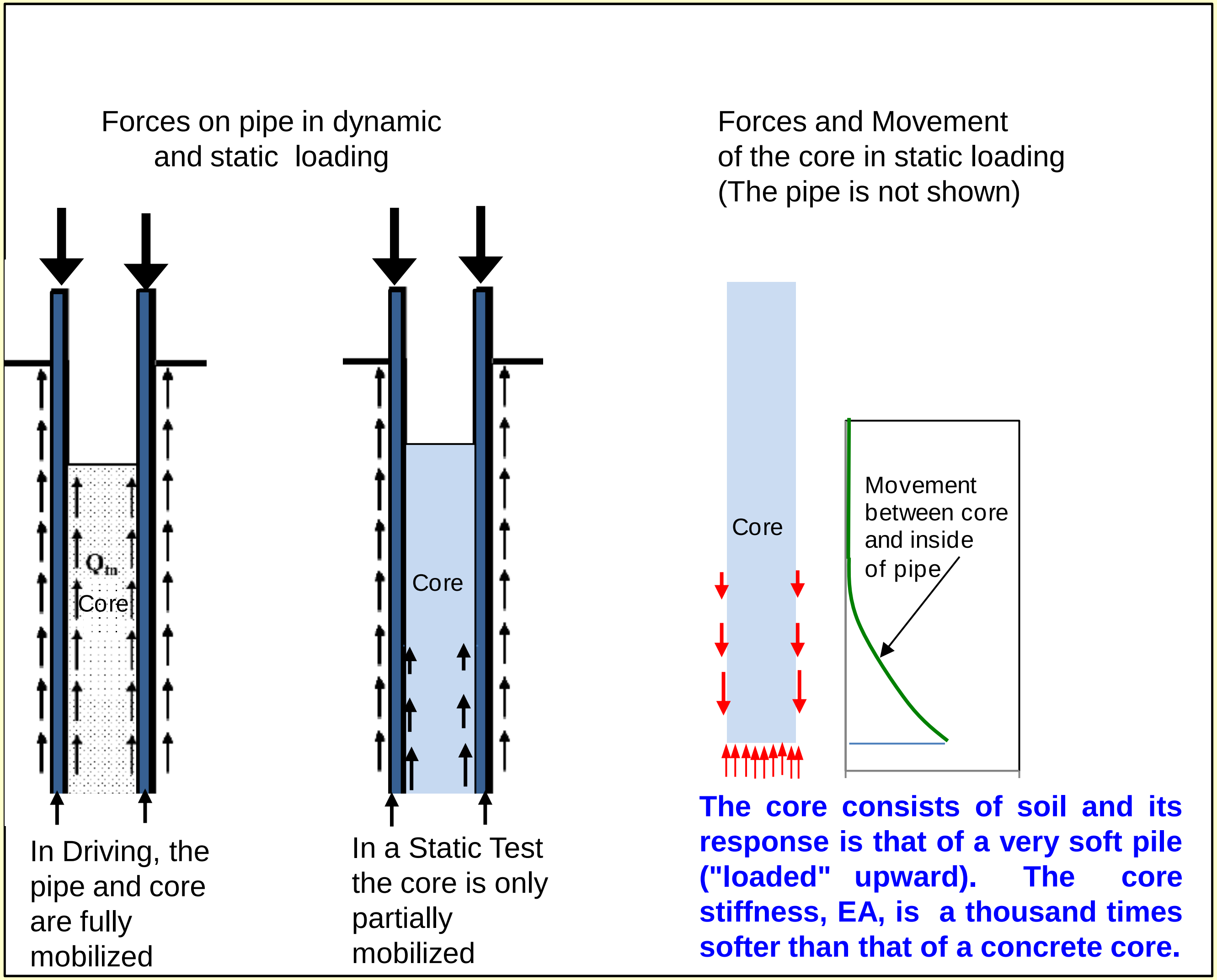
Also the best field work can get messed up if the analysis and conclusion effort loses sight of the history of the data



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

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

“Plugging” of an Open-toe Pipe Pile

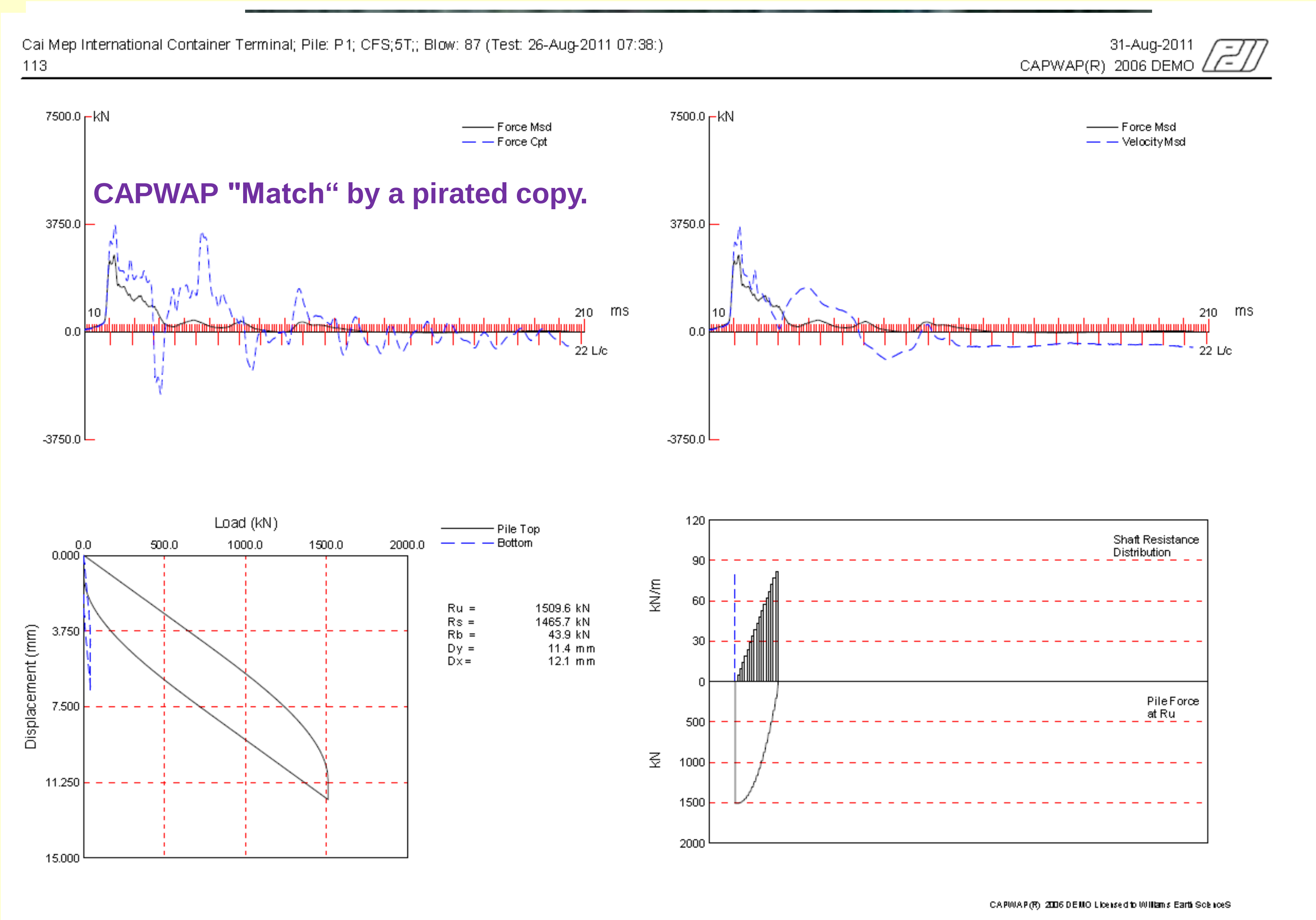


Therefore, CAPWAP-determined capacity is not likely the same as the capacity evaluated from the static loading test.

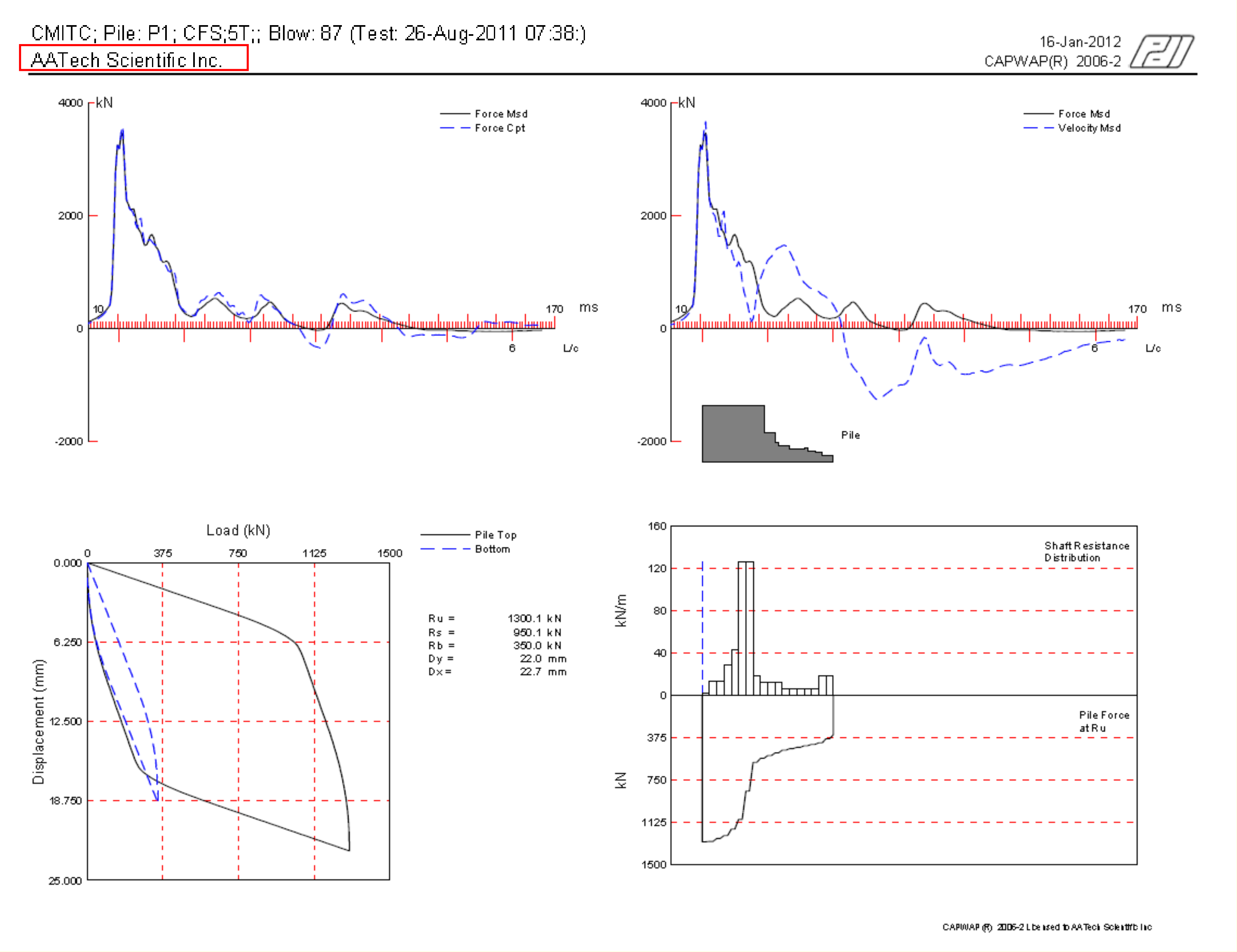
View on October 4, 2011, taken from the south-east end of CFS building showing some of the about 1,680 piles driven for the CFS.

Photos from the driving of piles with extension

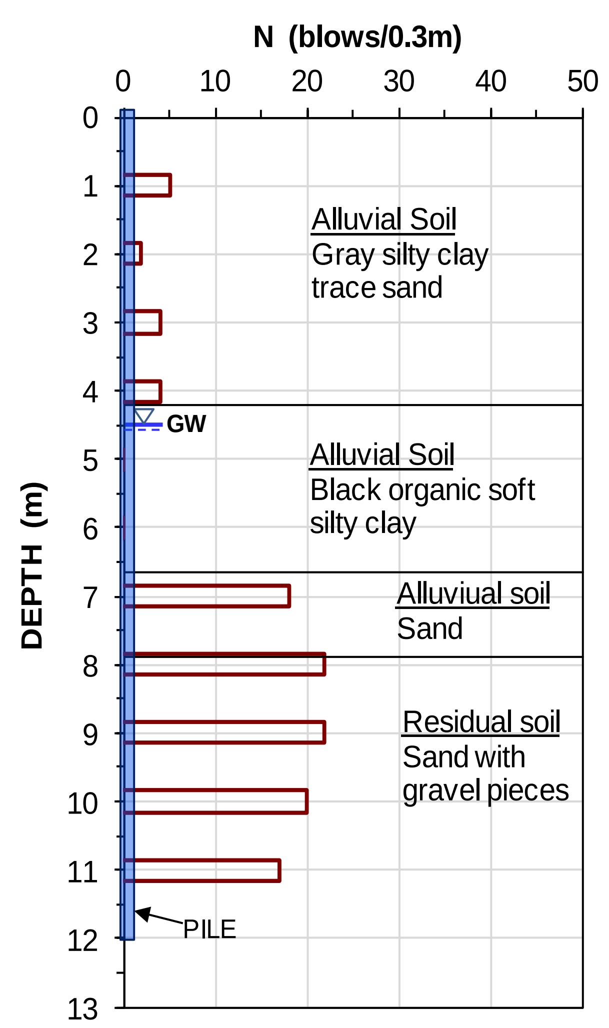
CAPWAP "Match“ by a pirated copy.



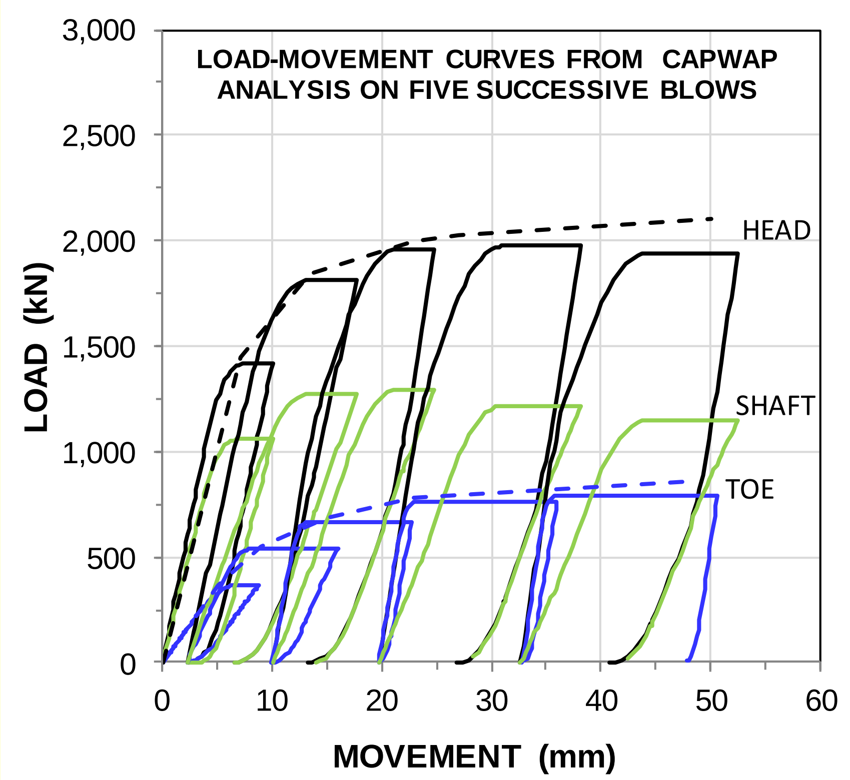
Here, a properly performed CAPWAP



Oliveira et al. (2008) reported a case history from Sao Paolo, Brazil, where dynamic tests were combined with a static loading test performed on a 700-mm diameter, 12 m long, CFA pile. The dynamic test and static loading tests were carried out 66 days and 97 days, respectively, after constructing the pile.

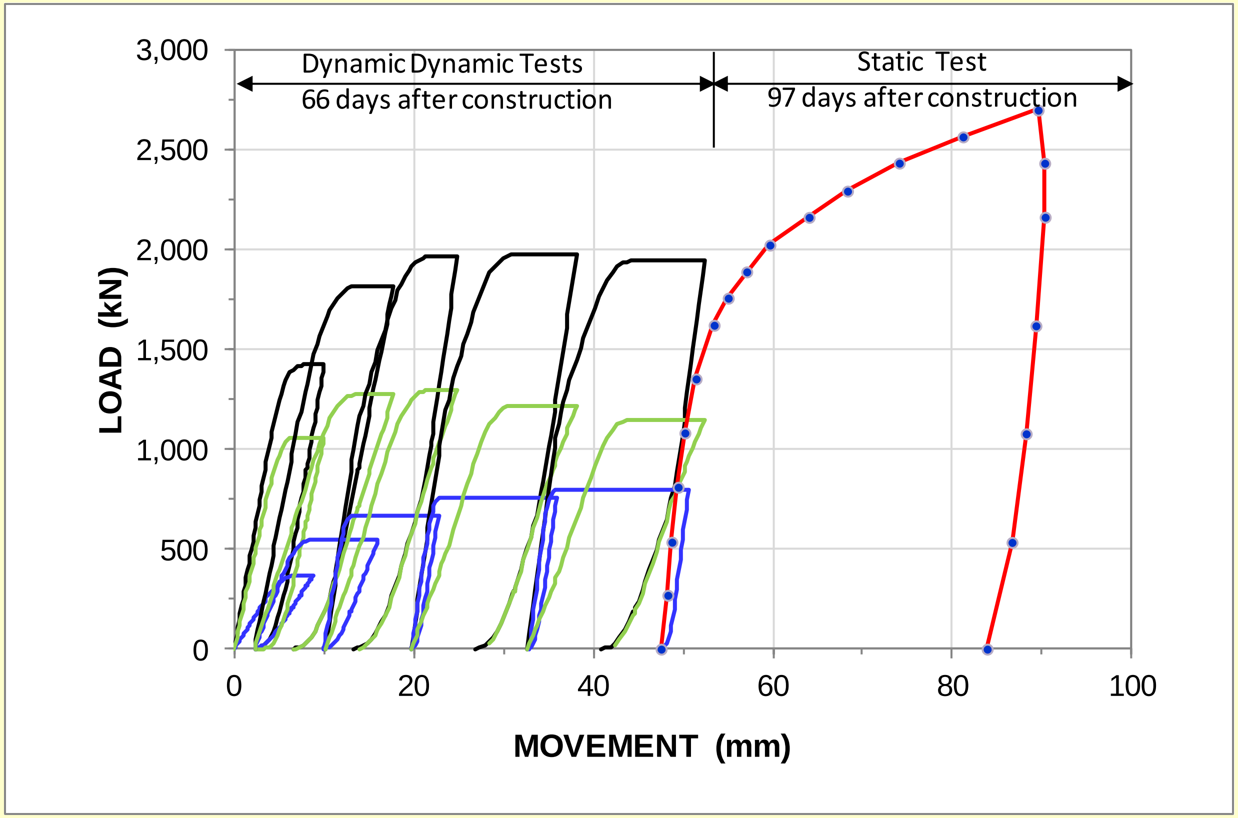


The dynamic tests followed the procedure of Aoki (2000) called "Dynamic Increasing Energy Test, DIET", consisting of a succession of blows from a special free-falling drop hammer, while monitoring the induced acceleration and strain with the Pile Driving Analyzer. Five blows were given with an 8,000-kg hammer and heights-of-fall of 200, 400, 600, 800, and 1,000 mm, respectively. Each blow was analyzed by means of the CAPWAP program.



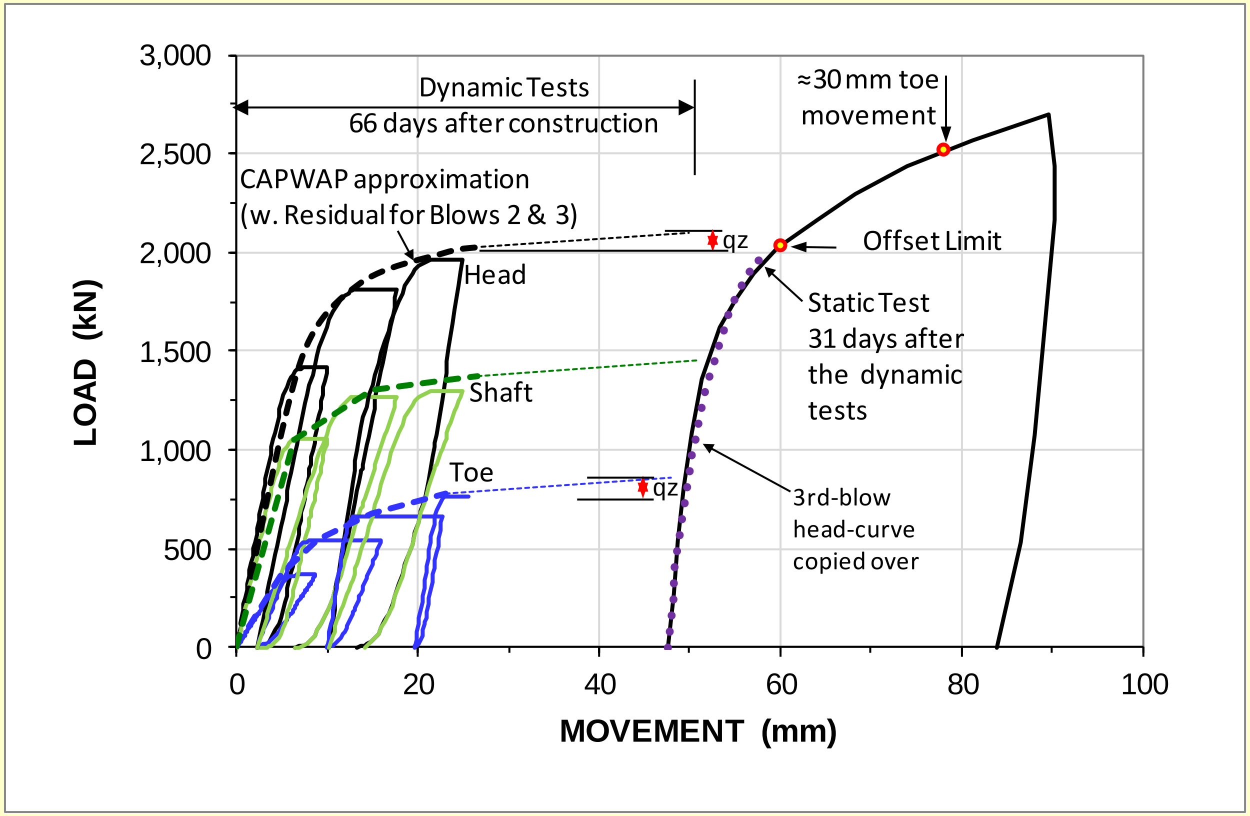
342. Fellenius, B.H., 2014. Analysis of results from routine static loading tests with emphasis on the bidirectional test. Proceedings of the 17th Congress of the Brasiliero de Mecanica dos Solos e Egenharia, Comramseg, Goiania, Brazil, September 10 - 13, 22 p.

Now, with the load-movement curve from the static tests



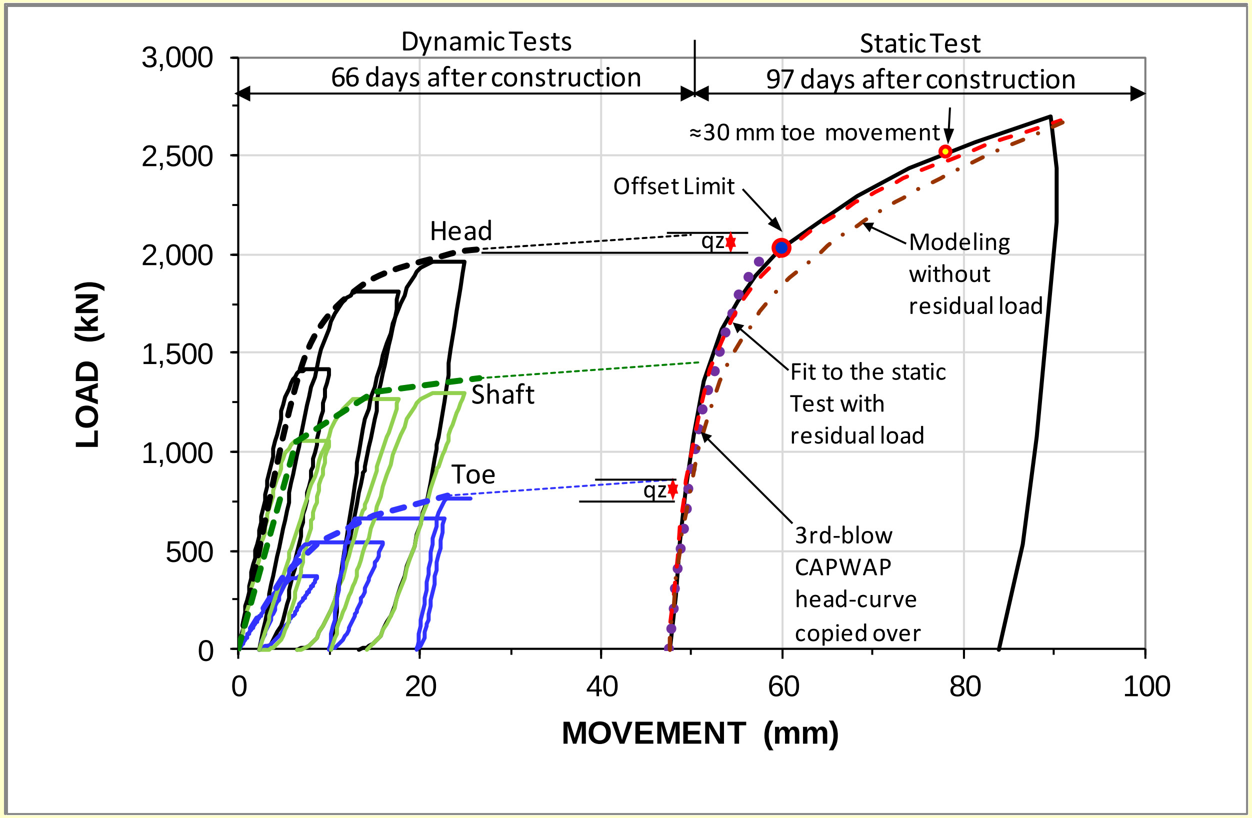
These results were used to state that the capacity determined in the dynamic test did not agree with that from the static test!

Now, with the load-movement curve from the static tests



On closer examination, the records do agree and the quality of the agreement is unusually good.

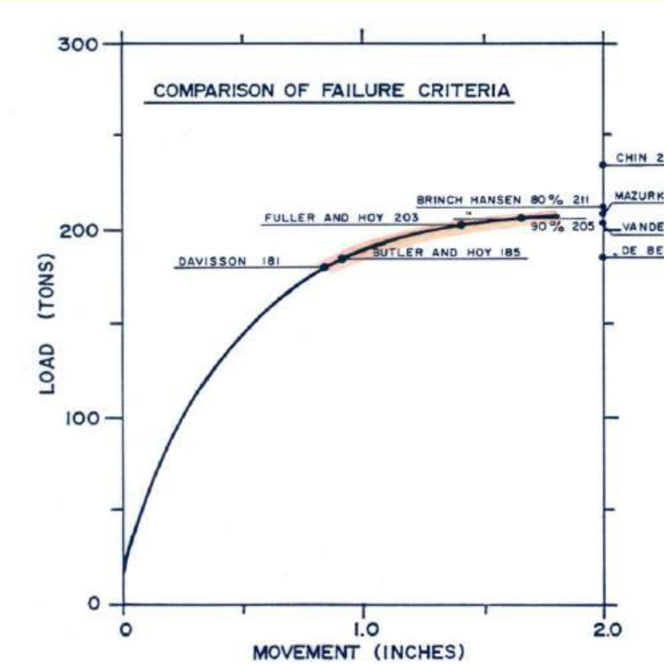
Now, with the load-movement curve from the static tests

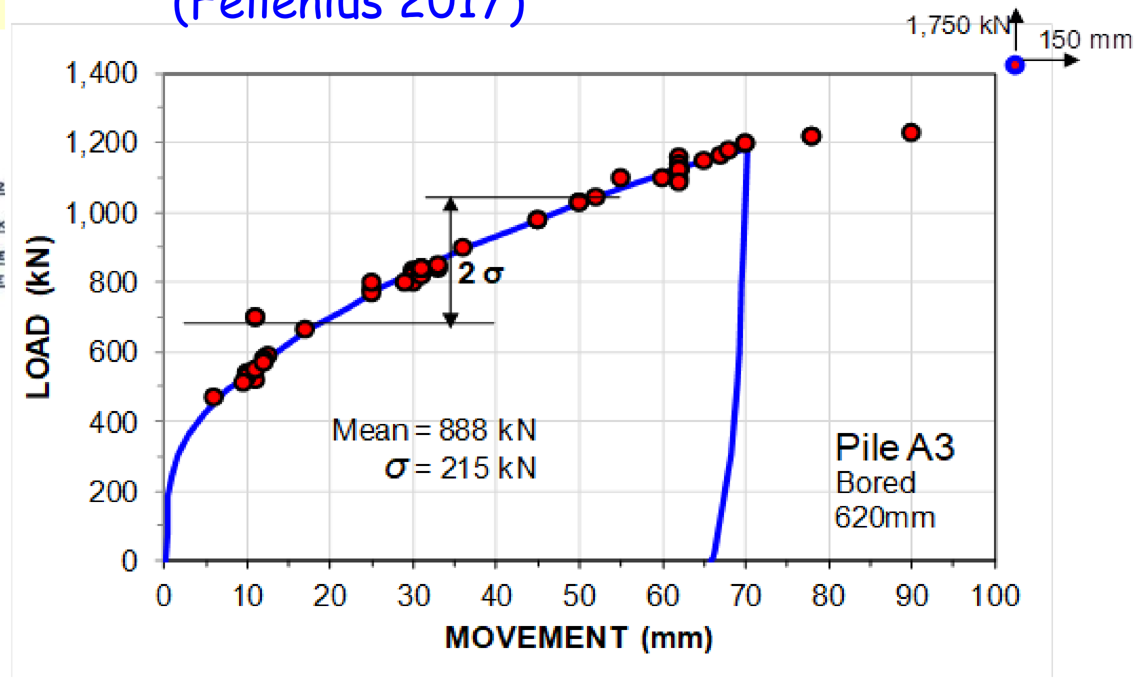


On closer examination, the records do agree and the quality of the agreement is unusually good.

As no surprise at all, the dynamic testing introduced residual load in the pile which made the pile response in the static test a little stiffer than would have been the case in the absence of a prior dynamic test (as shown by the curve "Modeling without residual load).

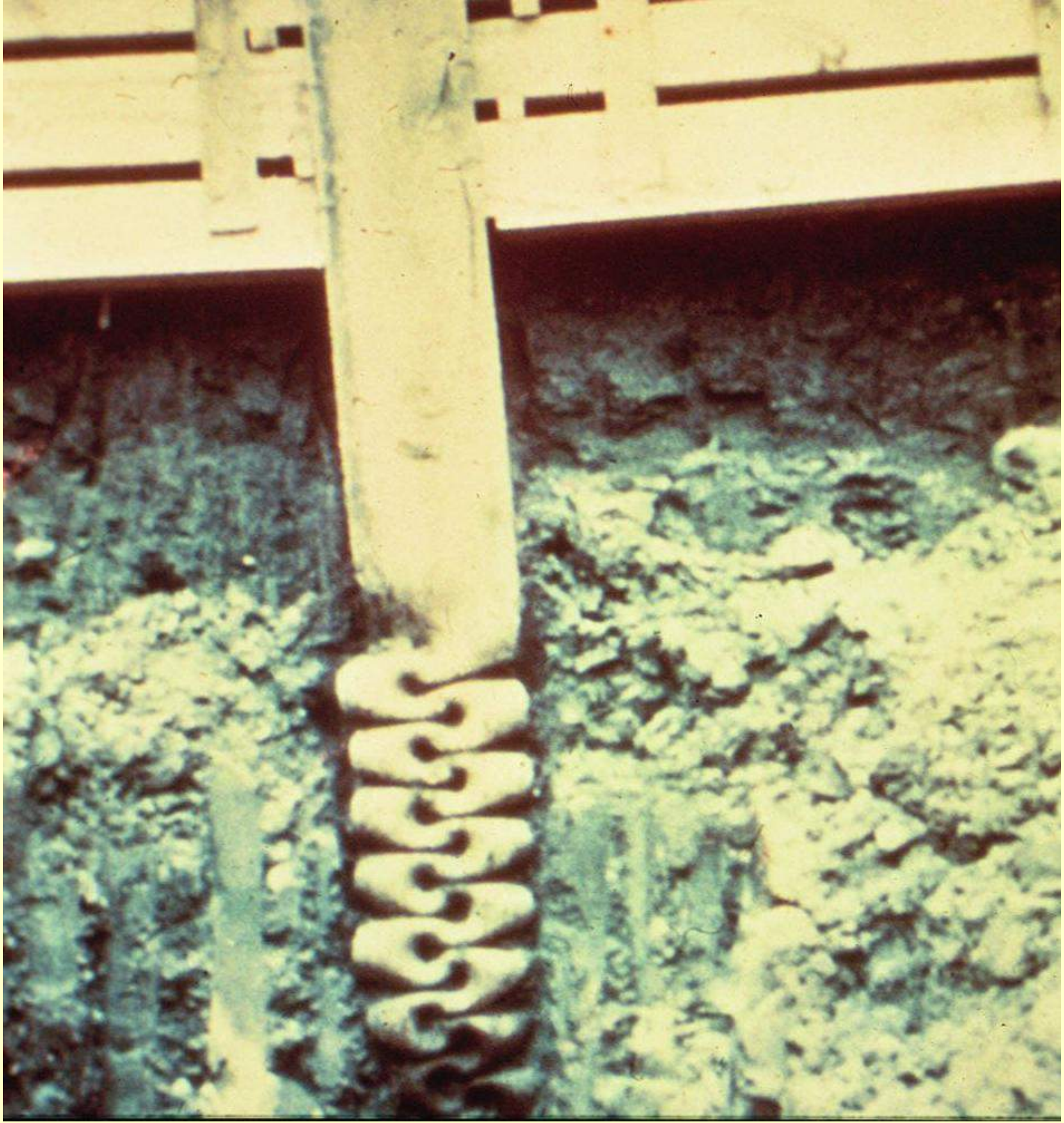
Capacities assessed in a survey of 94 professionals and specialists (Fellenius 2017)





Range of definitions of "Capacity" (Fellenius 1975!)

You can always define a "capacity" and then determine it from the pile-head load-movement curve. So, what pile "capacity" would you assess from this static test?



Fred Kulhawy collection

Thank you for your attention

Hal Hunt's "Pointless" Collection