A routine static loading test provides the load-movement of the pile head... and the pile capacity?
The Offset Limit Method
Davisson (1972)

\[ Q = \frac{AE}{L} \Delta L \]

OFFSET (inches) = 0.15 + \( b \)/120
OFFSET (mm) = 4 + \( b \)/120
\( b \) = pile diameter

Tom Davisson determined this definition as the one that fitted best to the capacities he intuitively determined from a FWHA data base of loading tests on driven piles. The definition does not mean or prove the pile diameter has anything to do with the interpretation of capacity from a load-movement curve.

The Decourt Extrapolation
Decourt (1999)

\[ \frac{Q}{\delta} = \frac{C_2}{C_1} \]

\( C_1 \) = Slope
\( C_2 \) = Y-intercept

Ult.Res = 474 kips
\( R_0 = 470 \text{ kips} - 230 \text{ tons} \)
Other methods are:

- The Load at Maximum Curvature
- Mazurkiewicz Extrapolation
- Chin-Kondner Extrapolation
- DeBeer double-log intersection
- Fuller-Hoy Curve Slope
- The Creep Method
- Yield limit in a cyclic test

For details, see Fellenius (1975, 1980)
A rational, upper-limit definition to use for "capacity" is the load that caused a 30-mm pile toe movement.

Indeed, bringing the toe movement into the definition is the point. A "capacity" deduced from the movement of the pile head in a static loading test on a single pile has little relevance to the structure to be supported by the pile. The relevance is even less when considering pile group response.

Definition of capacity (ultimate resistance) is only needed when the actual value is not obvious from the load-movement curve. However, the below test result is rare.
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?
Result on a test on a 2.5 m diameter, 80 m long bored pile
Plotting the repeat test in proper sequence

The Testing Schedule

A much superior test schedule. It presents a large number of values (≈20 increments), has no destructive unloading/reload cycles, and has constant load-hold duration. Such tests can be used in analysis for load distribution and settlement and will provide value to a project, as opposed to the long-duration, unloading/reloading, variable load-hold duration, which is a next to useless test.

Plan for 200 %, but make use of the opportunity to go higher if this becomes feasible.

The schedule in blue is typical for many standards. However, it is costly, time-consuming, and, most important, it is diminishes or eliminates reliable analysis of the test results.
Pile Interaction

CASE 1

Load-Movement curves from static loading tests on two "ACTIVE" piles (one at a time) and one "PASSIVE".

Diameter, \( b = 400 \text{ mm} \); Depths = 8.0 m and 8.6 m. The "PASSIVE" piles are 1.2 m and 1.6 m away from the "ACTIVE" (c/c = 3b and 4b).


CASE 2

Load-Movement curves from static loading tests on one pile ("ACTIVE"). Diameter (b) = 500 mm; Depth = 20.6 m. "PASSIVE" pile is 3.5 m away (c/c = 7b).

Group Effect

Comparing tests on single pile, a 4-pile group, and a 9-pile group

O'Neill et al. (1982)
Loading tests on a single pile and a group of 5 piles in loose, clean sand at Gråby, Sweden.

Pile #1 was driven first and tested as a single pile. Piles #2 - #5 were then driven, whereafter the full group was tested with pile cap not in contact with the ground.

First test: Pile #1 as a single pile. Second test (after driving Piles #2 - #5): Testing all five piles with measuring the load on each pile separately.

Data from Phung, D.L (1993)
Pile #1: Effect of compaction caused by driving Piles #2 through #5 and then testing #1 again

Data from Phung, D.L (1993)

4

Instrumentation and Interpretation
**Telltales**

- A telltale measures shortening of a pile and must never be arranged to measure movement.
- Let toe movement be the pile head movement minus the pile shortening.
- For a single telltale, the **shortening** divided by the distance between the pile head and the telltale toe is the **average strain** over that length.
- For two telltales, the distance to use is that between the telltale tips.
- The **strain** times the cross section area of the pile times the pile material E-modulus is the average **load** in the pile.

- To plot a load distribution, where should the load value be plotted? Midway of the length or above or below?

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**Load distribution for constant unit shaft resistance**

![Diagram showing load distribution for constant unit shaft resistance](image-url)
**Linearly increasing** unit shaft resistance and its load distribution

\[ A_1 = A_2 \implies \frac{ax^2}{2} = 0.5 \frac{ah^2}{2} \implies x = \frac{h}{\sqrt{2}} \]

- Today, telltales are not used for determining strain (load) in a pile because using strain gages is a more assured, more accurate, and cheaper means of instrumentation.
- However, it is good policy to include a toe-telltale to measure toe movement. If arranged to measure shortening of the pile, it can also be used as an approximate back-up for the average load in the pile.
- The use of vibrating-wire strain gages (sometimes, electrical resistance gages) is a well-established, accurate, and reliable means for determining loads imposed in the test pile.
- It is very unwise to cut corners by field-attaching single strain gages to the re-bar cage. Always install factory assembled “sister bar” gages.
Rebar Strain Meter — “Sister Bar”

Three bars?!

Hayes 2002
The curves are well together and no bending is discernible.

Both pair of curves indicate bending; averages are very close; essentially the same for the two pairs.

If one gage “dies”, the data of surviving single gage should be discarded. It must not be combined with the data of another intact pair. Data from two surviving single gages must not be combined.
**Glostrext Retrievable Extensometer (Geokon 1300 & A9)**

Anchor arrangement display


Gage for measuring displacement, i.e., distance change between upper and lower extensometers. Accuracy is about 0.02mm/5m corresponding to about 5 µε.

Schematic diagram of typical instrumented spun pile using Global Strain Extensometer technology

Load transferred ($P_{vo}$) at midpoint of each anchored interval can be calculated as:

$$ P = c (E_c A_c) $$

where,

- $c$ = average change in global strain gauge readings;
- $A_c$ = cross-sectional area of spun pile section;
- $E_c$ = concrete secant modulus in pile section.
That the shape of a pile sometimes can be quite different from the straight-sided cylinder can be noticed in a retaining wall built as a pile-in-pile wall.

Determining actual shape of the bored hole before concreting

Data from Loadtest Inc.
We have got the strain. How do we get the load?

• Load is stress times area

• Stress is Modulus (E) times strain

\[ \sigma = E \varepsilon \]

• The modulus is the key

For a concrete pile or a concrete-filled bored pile, the modulus to use is the combined modulus of concrete, reinforcement, and steel casing

\[ E_{\text{comb}} = \frac{E_s A_s + E_c A_c}{A_s + A_c} \]

- \(E_{\text{comb}}\) = combined modulus
- \(E_s\) = modulus for steel
- \(A_s\) = area of steel
- \(E_c\) = modulus for concrete
- \(A_c\) = area of concrete
• The modulus of steel is 200 GPa (207 GPa for those weak at heart)

• The modulus of concrete is . . . . ?

Hard to answer. There is a sort of relation to the cylinder strength and the modulus usually appears as a value around 30 GPa, or perhaps 20 GPa or so, perhaps more.

This is not good enough answer but being vague is not necessary. The modulus can be determined from the strain measurements.

Calculate first the \textit{change of strain for a change of load} and plot the values against the strain.

\[ E_t = \frac{\Delta \sigma}{\Delta \varepsilon} \]

Values are known

Example of “Tangent Modulus Plot”
In the stress range of the static loading test, modulus of concrete is not constant, but a more or less linear relation to the strain

\[ E_t = \left(\frac{d\sigma}{d\varepsilon}\right) = a\varepsilon + b \]

Which can be integrated to:

\[ \sigma = \left(\frac{a}{2}\right)\varepsilon^2 + b\varepsilon \]

But stress is also a function of secant modulus and strain:

\[ \sigma = E_s \varepsilon \]

Combined, we get a useful relation:

\[ E_s = 0.5a\varepsilon + b \]

and \[ Q = A E_s \varepsilon \]
Note, just because a strain-gage has registered some strain values during a test does not guarantee that the data are useful. Strains unrelated to force can develop due to variations in the pile material and temperature and amount to as much as about $50\pm$ microstrain. Therefore, the test must be designed to achieve strains due to imposed force of ideally about 500 microstrain and beyond. If the imposed strains are smaller, the relative errors and imprecision will be large, and interpretation of the test data becomes uncertain, causing the investment in instrumentation to be less than meaningful. The test should engage the pile material up to at least half the strength. Preferably, aim for reaching close to the strength.


Strain-gage instrumented, 16.5-inch octagonal prestressed concrete pile driven to 60 m depth through coral clay and sand. Modulus relations as obtained from uppermost gage (1.5 m below head, i.e., 3.6b).

The tangent stiffness approach can be applied to all gage levels. The differentiation eliminates influence of past shear forces and residual load. Non-equal load increment duration will adversely affect the results.

The secant stiffness approach can only be applied to the gage level immediately below the pile head (must be uninfluenced by shaft resistance), provided the strains are uninfluenced by residual load.
Unlike steel, the modulus of concrete varies and depends on curing, proportioning, mineral, etc. and it is strain dependent. However, the cross sectional area of steel in an instrumented steel pile is sometimes not that well known.

Pile stiffness for a 1.83 m diameter steel pile; open-toe pipe pile. Strain-gage pair placed 1.8 m below the pile head.

(Data from Bradshaw et al. 2012)

For "calibrating" uppermost gage level, the secant method appears to be the better one to use, right?

Pile stiffness (Q/ε and ΔQ/Δε versus ε) for a 600 mm diameter concreted pipe pile. The gage level was 1.6 m (3.2b) below the pile head.

Data from Fellenius et al. 2003
Or this case? Here, that initial "hyperbolic" trend can be removed by adding a mere 20 µε to the strain data, "correcting the zero" reading, it seems.

Pile stiffness for a 600-mm diameter prestressed pile. The gage level was 1.5 m (2.5b) below pile the head.

Data from CH2M Hill 1995

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Determining load from strain-gage measurements in the pile

The strain-gage measurement is supposed to be the change of strain due to the applied load relative the “no-load” situation (i.e., when no external load acts at the gage location).

But, is the “no-load” situation really the reading taken at the beginning of the test? What is the true “zero-reading” to use?
• We often assume – somewhat optimistically or naively – that the reading before the start of the test represents the “no-load” condition.

• However, at the time of the start of the loading test, loads do exist in the pile and they are often large.

• For a grouted pipe pile or a concrete cylinder pile, these loads are to a part the effect of the temperature generated during the curing of the grout.

• Then, the re-consolidation (set-up) of the soil after the driving or construction of the pile will impose additional loads on the pile.

Example from Gregersen et al., 1973

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

B. Load and resistance in DA for the maximum test load
Residual load affecting shaft resistance determined from a static loading test

Hysteresis loop for shaft resistance mobilized in a static loading test

ABOVE THE NEUTRAL PLANE When no residual load is present in the pile at the start of the test, the starting point of the load-movement response is the origin, O, and in a subsequent static loading test, the shaft shear is mobilized along Path O-B. Residual load develops as negative skin friction along Path D-A (plotting from "D" instead of from "O"). Then, in a static loading test, the shaft shear is mobilized along Path A-O-B. However, if presence of residual load is not recognized, the path will be thought to be along A-B -- practically the same. If Point B represents fully mobilized shaft resistance, then, the assumption of no residual load in the pile will indicate a "false" resistance that is twice as large as the "true" resistance.

"Continuation" loop for shaft resistance mobilized in a static loading test

BELOW THE NEUTRAL PLANE When no residual load is present in the pile at the start of the test, the starting point of the load-movement response is the origin, O, and in a subsequent static loading test, the shaft shear is mobilized along Path O-Z-X. Residual load develops as positive shaft resistance along Path O-Z. Then, in a static loading test, additional shaft resistance is mobilized along Path Z-X. However, if presence of residual load is not recognized, the origin will be thought to be O, not Z. If Point X represents fully mobilized shaft resistance, then, the assumption of no residual load in the pile will indicate a "false" resistance that could be only half the "true" resistance.

Pile toe response in unloading and reloading in a static loading test

Similar to the below the N.P. for the shaft, if residual load develops, it will be along Path O-Z. Then, in a static loading test, the toe resistance (additional) is mobilized along Path Z-X and on to Y and beyond.

When the pile construction has involved an unloading, say, at Point X, per the Path X-I-II, the reloading in the static loading test will be along Path I-II-X and on to Y. Unloading of toe load can occur for driven piles and jacked-in piles, but is not usually expected to occur for bored piles (drilled shafts). However, it has been observed in such piles, in particular for test piles which have had additional piles constructed around them and for full displacement piles. The toe resistance be underestimated and the break in the reloading curve at Point X can easily be mistaken for a failure load and be so stated.
Method for evaluating the residual load distribution

Immediately before the test, all gages must be checked and "Zero Readings" must be taken.

Answer to the question in the graph:

No, there's always residual load 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.
Concrete hydration **temperature** measured in a grouted concrete cylinder pile

Temperature at various depths in the grout of a 0.4 m center hole in a 56 m long, 0.6 m diameter, cylinder pile.

Pusan Case

Change of strain measured in a 74.5 m long, 2.6 m diameter bored pile

Change of strain during the hydration of the grout in the Golden Ears Bridge test pile
Temperature During Curing of a 2m Lab Specimen

Strain History During Curing of a 2 m Long Full-width Lab Specimen
First 5 days after placing concrete
The strain gages themselves are not are temperature sensitive, but the records may be!

The vibrating wire and the rebar have almost the same temperature coefficient. However, the coefficients of steel and concrete are slightly different. This will influence the strains during the cooling of the grout. More important, the rise of temperature in the grout could affect the zero reading of the wire and its strain calibration. It is necessary to "heat-cycle" (anneal) the gage before calibration. (Not done by all, but annealing is a routine measure of Geokon, US manufacturer of vibrating wire gages).
• Readings should be taken immediately before (and after) every event of the piling work and not just during the actual loading test

• The No-Load Readings will tell what happened to the gage before the start of the test and will be helpful in assessing the possibility of a shift in the reading value representing the no-load condition

• If the importance of the No-Load Readings is recognized, and if those readings are reviewed and evaluated, then, we are ready to consider the actual readings during the test

Results of static loading tests on a 40 m long, jacked-in, instrumented steel pile in a saprolite soil
A more thoughtful analysis of the data

A static loading test with a toe telltale to measure toe movement—typical records

Now with instrumentation to separate shaft and toe resistances

Shaft resistance load-movement curve (t-z function) — typical

300 mm diameter, 30 m long concrete pile
- $E = 35$ GPa
- $\beta = 0.30$; Strain-softening

SHORTENING

300 mm diameter, 30 m long concrete pile
$E = 35$ GPa; $\beta = 0.30$ at $\delta = 5$ mm; Strain-softening: Zhang with $\delta = 5$ mm and $a = 0.0125$

SEGMENT MOVEMENT AT MID-POINT (mm)
SEGMENT SHAFT RESISTANCE AT MID-POINT (kN)
lower
middle
upper

LOAD at PILE HEAD (kN)
PILE HEAD MOVEMENT (mm)
HEAD TOE

LOAD (KN)
DEPTH (m)
$\beta = 0.3$
$\beta = 0.4$

UNIT SHAFT SHEAR (KPa)
DEPTH (m)
$\beta = 0.3$
$\beta = 0.4$

A more thoughtful analysis of the data

The butler. The differentiation did it.
The simulations are made using UniPile Version 5.
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 de Solos 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.
Three Cells inside the reinforcing cage
(My Thuan Bridge, Vietnam)
The bidirectional cell can also be installed in a driven pile (after the driving). Here in a 600 mm cylinder pile with a 400 mm central void.

Bidirectional cell attached to an H-beam inserted in a augercast pile after grouting.
Test on a 1,250 mm diameter, 40 m long, bored pile at US82 Bridge across Mississippi River installed into dense sand

The Equivalent Head-down Load-movement Curve

Measured upward and downward curves

With correction for the increased pile compression in the head-down test

Construction of the “Direct Equivalent Curve”

Reference: Appendix to regular reports by Fugro Loadtest Inc.
From the upward and downward results, one can produce the equivalent head-down load-movement curve that one would have obtained in a routine “Head-Down Test”

Example 1
Example 2

Upward and downward curves fitted to measured curves

UniPile5 analysis using the t-z and q-z curves fitted to the load-movement curves at the gage levels in an effective-stress simulation of the test
Example 2: The upward shaft response extracted and compared to the response of the shaft to an "Equivalent Head-down Test"

The difference shown above between the upward BD cell-plate and the head-down load-movement curves 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 effect of residual load on a bidirectional test

The cell load includes the residual load whereas the load evaluated from the strain-gages does not.
Similar to combining a compression test with a tension test, combining a bidirectional test and a head-down compression test will help in determining the true resistance.

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.

Acknowledgment: The bidirectional test are courtesy of Arcos Egenharia de Solos Ltda., Belo Horizonte, Brazil.
After data reduction and processing

Equivalent Head-down Load-movements

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

Equivalent Head-down Load-distributions

Pensacola, Florida

410 mm diameter, 22 m long, precast concrete pile driven into silty sand
After the push test, the pile toe is located higher up than when the test started!

Bidirectional-cell test on a 16-inch, 72-ft, prestressed pile driven into sand.

Los Angeles Coliseum, 1994

The Northridge earthquake in Los Angeles, California, in January 1994 was a “strong” moment magnitude of 6.7 with one of the highest ground accelerations ever recorded in an urban area in North America.

The earthquake caused an estimated $20 billion in property damage. Amongst the severely damaged buildings was the Los Angeles Memorial Coliseum, which repairs and reconstruction cost about $93 million. The remediation work included construction of twenty-eight, 1,300 mm diameter, about 30 m long, bored piles, each with a working load of almost 9,000 KN (2,000 kips), founded in a sand and gravel deposit.

The piles had been designed using the usual design approach with adequate factors of safety to guard against the unknowns. Moreover, the acceptable maximum movement was more stringent than usual.

It was imperative that all construction work was finished in six months (September 1994, the start of the football season). However, after constructing the first two piles, which took six weeks, it became obvious that constructing the remaining twenty-six piles would take much longer than six months. Drilling deeper than 20 m was particularly time-consuming. The design was therefore changed to about 18 m length, combined with equipping every pile with a bidirectional cell at the pile toe. Note, the cell was now used as a construction tool.
The first stage loading is of interest in the context of general evaluation of test results and applying them to design. The second is where the bidirectional cell was used as a preloading tool, resulting in a significant increase of toe stiffness, much in the way of the Expander base.
Bidirectional Tests on a 1.4 m diameter bored pile in North-West Calgary constructed in silty glacial clay till

A study of Toe and Shaft Resistance Response to Loading and correlation to CPTU calculation of capacity
The upper 8 m will be removed for basement.

14 m net pile length

Cone Stress, $q_t$ (MPa)

Sleeve Friction, $f_s$ (KPa)

Pore Pressure (KPa)

Friction Ratio, $f_R$ (%)
Load Distribution

Load Measured Distribution Compared to Distributions Calculated from the CPTU Soundings
Analysis of the results of a bidirectional test on a 21 m long bored pile

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 pile will be one a group of 16 piles (4 rows by 4 columns) installed at a 4-diameter center-to-center distance. Each pile is assigned a working load of 1,000 kN.

The soil profile determined by CPTU and SPT
The results of the bidirectional test

To fit a simulation of the test to the results, first input is the effective stress parameter (β) that returns the maximum measured upward load (840 kN), which was measured at the maximum upward movement (35 mm). Then, “promising” t-z curves are tried until one is obtained that, for a specific coefficient returns a fit to the measured upward curve. Then, for the downward fit, t-z and q-z curves have to be tried until a fit of the downward load (840 kN) and the downward movement (40 mm) is obtained.

Usually for large movements, as in the example case, the t-z functions show a elastic-plastic response. However, for the example case, no such assumption fitted the results. In fact, the best fit was obtained with the Ratio Function for the entire length of the pile shaft.
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 results appear to suggest that the pile is affected by a filter cake along the shaft and 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. The first author prefers 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.

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 addressed a few slides down.

The final fit establishes also the equivalent head-down distributions of shaft resistance and equivalent head-down load distribution for the maximum load (and of any load in-between, for that matter). Load distributions have also been calculated from the SPT-indices using the Decourt, Meyerhof, and O'Neil-Reese methods, as well that from the Eslami-Fellenius CPTU-method.

By fitting a UniPile simulation to the measured curves, we can determine all pertinent soil parameters, the applicable t-z and q-z functions, and the distribution of the equivalent head-down load-distribution. The results also enable making a comparison of the measured pile response to that calculated from the in-situ test methods.

However, capacity of the single pile is just one aspect of a piled foundation design. As mentioned, the key aspect is the foundation settlement.

Note, the analysis results suggest that the pile was more than usually affected by presence of a filter cake along the pile shaft and by some debris being present at the bottom of the shaft when the concrete was placed in the hole. An additional benefit of a UniPile analysis.