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The analysis of results from routine pile load tests

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THIS PAPER deals with the analysis of results from axial testing of vertical single piles, i.e. the most common field test performed. Despite the numerous tests which have been carried out and the many Papers which have reported on such tests and the analysis thereof, the understanding of pile test loading in current engineering practice leaves much to be desired. The reason for this is that the engineers have concerned themselves with mainly only one question: "Does the pile have a certain least capacity?", finding little of practical value in analysing the actual capacity and the pile-soil interaction. This Paper aims to show that engineering value can be gained from elaborating on a pile test — during the actual testing in the field, as well as in the analysis of the results.

The first portion of the notes make use of an earlier Paper by the author (Fellenius, 1975). However, additional views and recent literature have been incorporated.

Testing methods

The most common test procedure is the slow maintained load method referred to as the "standard loading procedure" in the ASTM Designation D-1143 (ASTM 1974) in which the pile is loaded in eight equal increments up to a maximum load, usually twice the predetermined allowable load. Each increment is maintained until zero settlement is reached, defined as 0.01in/h (= 0.002in/10 min.). The final load, the 200% load, is maintained for 24 hours. The "standard method" is very time consuming requiring from 30 to 70 hours to complete. It should be realised that the words "zero settlement" are very misleading, as the settlement rate of 0.01in (0.25mm)/hr is equal to a settlement of 7ft (2.1m)/yr.

The "standard method" can be speeded up by using the method of equilibrium proposed by Mohan *et al* (1967), where the load (jack pressure) is allowed to drop rather than being maintained by pumping. The equilibrium load value is taken as the load applied on the pile.

Housel (1966) proposes that each of the eight increments be maintained exactly one hour regardless of having reached "zero" settlement or not. The Housel method of applying the load at equal time intervals allows an analysis of movement with time, which is not possible with the "standard method". By plotting the magnitude of movement obtained during the last 30 minutes of each one-hour load duration versus the applied load, two approximately straight lines are obtained. Provided the test has approached failure, that is, the intersection of the two lines is termed yield value.

A maintained-load test according to Housel's method takes a full day to perform. The points on the curve are still very few, but Housel's method is a definite improvement of the "standard method" and it has been incorporated as an optional method in the ASTM Designation D-1143. However, it is the author's opinion that a test consisting of 16 equal increments of say 30 tons applied every 30 minutes would provide a better test than 8 increments of 60 tons applied every 1.0 hour, because it would provide a better defined load-movement curve. Also, a similar yield value, and one not much different, can be evaluated from the movement during the last 15 minutes, provided that readings are taken often enough and that they are accurate. But why stop at 16 increments, when 32 every 15 minutes determine the load deformation curve even better? The load is still applied at a constant rate in terms of tons per hour and no principal change is made.

Actually, the duration of each load is less important, be it 1.0 hour or 15 minutes; it is the fact that the duration of each load is the same which is important. From this realisation, we can progress to the one that even shorter time intervals, and an increase of the rate of loading in tons/hour, are possible without impairing the test. Actually, by using as short time intervals as practically possible, the time-dependent influences are reduced and a more truly undrained test is obtained. In those cases where a study of the time-dependent, or drained conditions, creep, etc. is desirable, the test duration should be measured in weeks,

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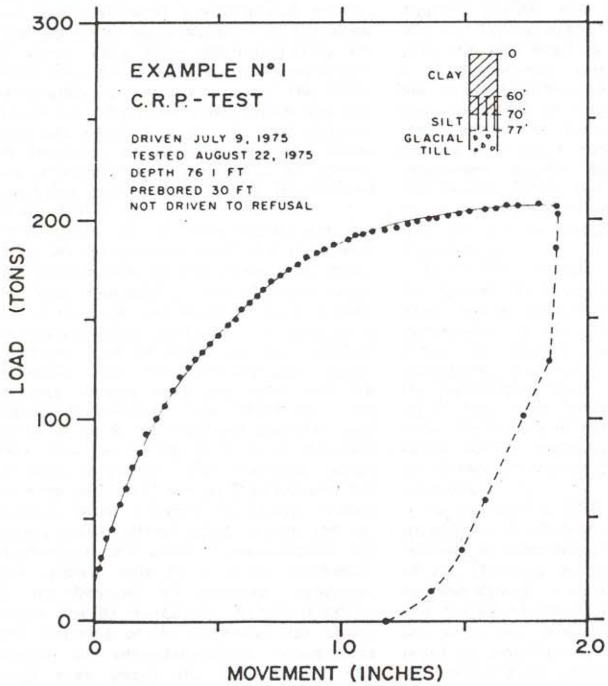


Fig. 1. Load-movement diagram from CRP test

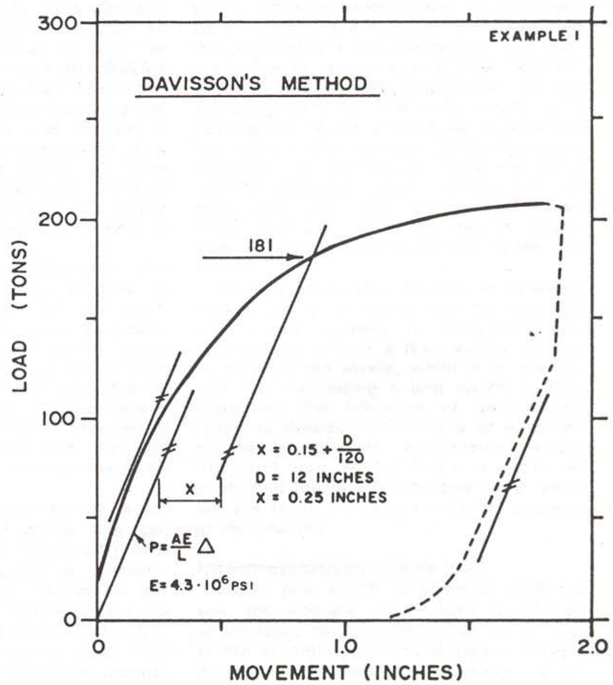


Fig. 2. Construction of Davisson's limit

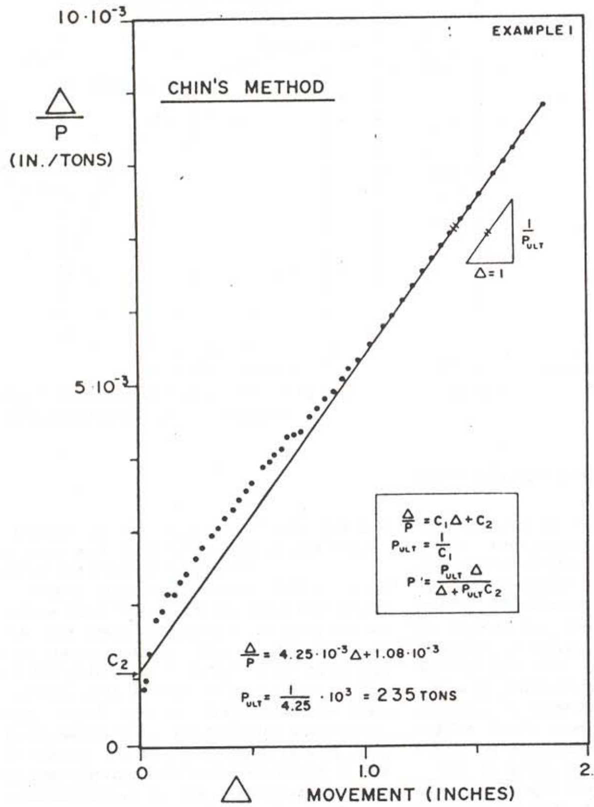
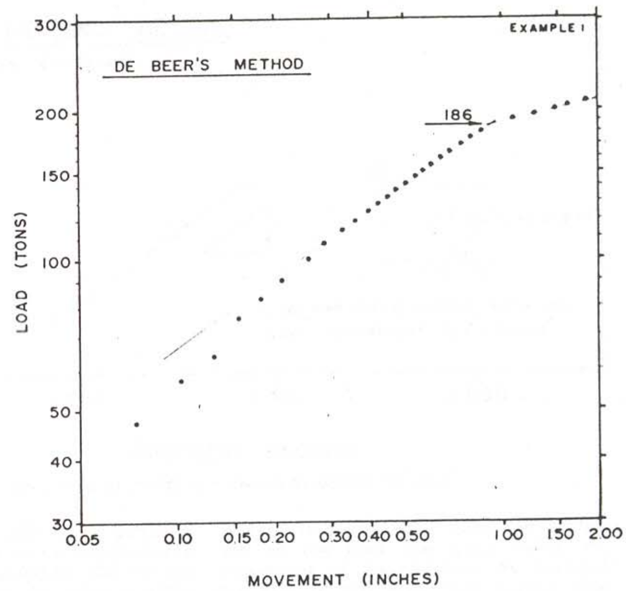


Fig. 3. Ultimate failure according to Chin



Fg. 4. Construction of De Beer's yield limit

months or even years. A 48 or 72 hour test is then vastly inadequate, and results only in confusion.

Tests which consist of load increments applied at constant time intervals of 5, 10 or 15 minutes are called Quick Maintained-Load Tests (ML tests) and are from both technical, practical and economical points of view superior to the slow ML tests. They have been relatively recently introduced into North America, but are steadily gaining acceptance. The latest version of the ASTM Designation has one quick ML method as an optional method. For instance, recently the Federal Highway Administration published an extensive users' manual for a Quick ML method (Butler & Hoy, 1977).

The Quick ML method should aim for 30 to 40 increments with the maximum load determined by the amount of reaction load available or the ultimate capacity of the pile. For routine cases, it may be diplomatically preferable to stay at a maximum load of 200% of the intended allowable load. For ordinary test arrangements, where only the load and the pile head movement are monitored, time intervals of 5 minutes are suitable and allow for the taking of 2 to 4 readings for each increment (for instance, when reaching the load, and at 2.5, 4.0 and 5.0 minutes after starting to load). When testing instrumented piles, where the instruments take a while to read (scan), the time interval may have to be increased. To go beyond 15 minutes, however, should not be necessary. Nor is it advisable, because of the potential risk of influence of time-dependent movements which may impair the test results. Usually, a Quick ML test is completed within two to three hours.

A quick test which has gained much use in Europe is the Constant Rate of Penetration test (CRP test), first proposed internationally for piles by Whitaker (1957 and 1963) and Whitaker & Cooke (1961). Manuals on the CRP test have been published by the Swedish Pile Commission (1970) and New York Department of Transportation (1974). In the CRP test, the pile head is forced to settle at a predetermined rate, normally 0.02in/min (0.5mm/min), and the load to achieve the movement is recorded. Readings are taken every two minutes and the test is carried out to a total penetration (i.e. movement of the pile head) of 2-3in (50-75mm) or to the maximum capacity of the reaction arrangement, which means that the test is completed within two to three hours.

The CRP test has the advantage over the Quick ML test in that it enables an even better determination of the load-deformation curve. This is of particular value in testing friction piles, when sometimes the force needed to achieve the penetration gets smaller after a peak value has been reached. It also agrees with the testing in most other engineering fields, which regularly use CRP methods to determine strength and stress-strain relations.

To perform a CRP test, access is required to a mechanical pump that can provide a constant and non-pulsing flow of oil. Ordinary pumps with a pressure-holding device, manual or mechanical, are not suitable because of unavoidable loading variations. Also, the absolute requirement of simultaneous reading of all load and deformation gauges (changing continuously) could be difficult to achieve without a trained staff. For these reasons, the Quick ML method is preferable for instrumented piles.

A fourth test method is cyclic testing. However, cyclic methods will not be described here; for details see Fellenius (1975), and references contained therein. In routine tests, cyclic loading or even single unloading and loading phases must be avoided. It is a common misconception that unloading a pile every now and then according to some more or less "logical" scheme will provide information on the tip movement. It will only result in a destruction of the chances to analyse the test results and the pile load-deformation behaviour. In non-routine tests and for a specific purpose, cyclic testing can be used, but then *after* completion of an initial test and when the pile is instrumented with at least a tell-tale to the pile tip.

There is absolutely no logic in believing that anything of value can be obtained from cyclic testing, or occasional unloadings, or one or a few resting periods at certain load levels, when it is realised that we are testing a unit which is subjected to the influence of several soil types, is already under stress of unknown magnitude, exhibits progressive failure, etc., and that all we know is what we apply and measure at the pile head, while we are really interested in what happens at the pile end.

Interpretation of failure load

For a pile which is stronger than the soil, the ultimate failure load is reached when rapid settlements occur under sustained or slightly increased load — the pile plunges. However, this definition is inadequate, because plunging requires very large movements and it is often less a

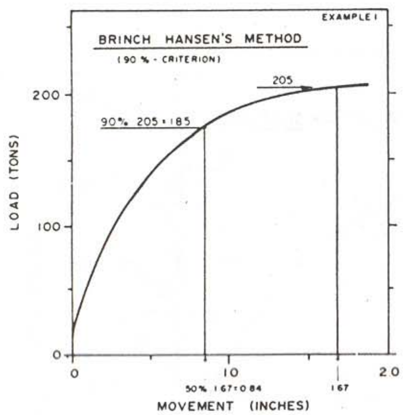


Fig. 5. Ultimate failure according to the 90% criterion by Brinch Hansen

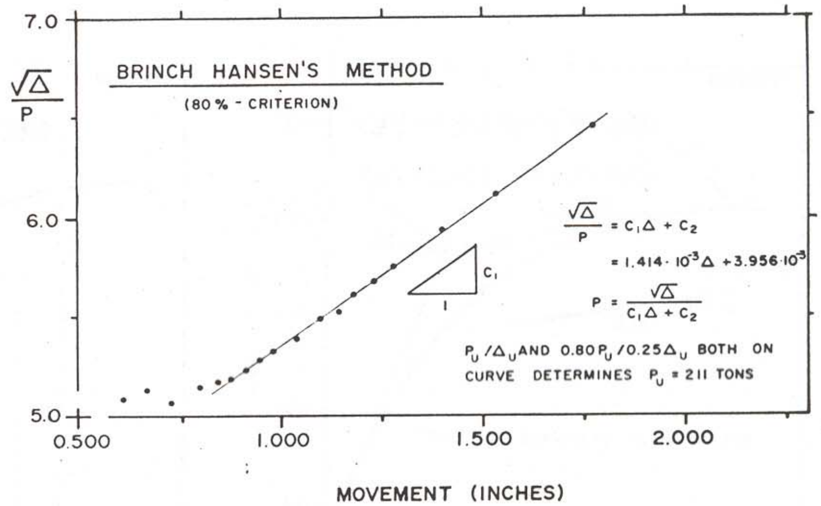


Fig. 6. Ultimate failure according to the 80% criterion by Brinch Hansen

function of the capacity of the pile-soil system and more a function of the capacity of the man-pump system.

In the past, a common definition of failure load has been the load for which the pile head movement exceeds a certain value, usually 10% of the diameter of the pile end. This definition does not consider the elastic deformations of the pile, which can be substantial for long piles, while it is negligible for short piles. In reality, a limit movement relates only to the allowable deformation limits of the superstructure to be supported by the pile, and not to the load test results.

Sometimes, the failure value is defined as the load value at the intersection of two lines, approximating an initial pseudo-elastic portion of the load-movement curve and a final pseudo-plastic portion. This definition results in interpreted failure loads, which depend greatly on judgement and, above all, on the scales of the graph. Change the scales and the failure value changes also. A load test is influenced by many occurrences, but the draughting manner should not be one of these.

To be useful, a failure definition must be based on some mathematical rule and generate a repeatable value that is independent of scale relations and the opinions of the individual interpreter. In some way, it has to consider the shape of the load-movement curve or, if not, it must consider the length of the pile (which the shape of the curve indirectly does). Without such proper definition, every interpretation becomes meaningless.

The test results given as a load-movement curve in Fig. 1 will be used to present nine different definitions of failure. The example pile is a 12in (305mm) concrete pile installed through 60ft (18.3m) of sensitive clay, 10ft (3.0m) of clayey silt and 6ft (1.8m) of silt. The pile was tested six weeks after driving. Method of testing was the CRP method. The pile started to plunge when the test load reached 200 tons, but at the maximum load of 206 tons the load necessary to achieve the movement was still increasing.

In Fig. 2 is applied a method proposed by Davisson (1972), also referenced by Peck *et al* (1974). Davisson's limit value is defined as the load corresponding to the movement which exceeds the elastic compression of the pile by a value of 0.15in (4mm) plus a factor equal to the diameter of the pile divided by 120. For the 12in dia. example pile, the value is 0.25in (6mm). The Davisson limit was developed in conjunction with the wave equation analysis of driven piles and has gained widespread use in phase with the increasing popularity of this method of analysis. It is primarily intended for test results from driven piles tested in accordance with quick methods.

Fig. 3 gives the method proposed by Chin (1970 and 1971) for piles in applying the general work by Kondner (1963). The method assumes that the load-movement curve when the load approaches the failure load is of hyperbolic shape. By the Chin method, each load value is divided with its corresponding movement value and the resulting value is plotted against the movement. As shown in Fig. 3, after some initial variation, the plotted values fall on a straight line. The inverse slope of this line is the Chin failure load.

Generally speaking, two points will determine a line and a third point on the same line confirms the line. However, beware of this statement when using Chin's method. It is very easy to arrive at a false Chin value if applied too early in the test. Normally, the correct straight line does not start to materialise until the test load has passed the Davisson limit. As a rule, the Chin failure load is about 20% to 40% greater than the Davisson limit. When this is not the case, it is advisable to take a closer look at all the test data.

The Chin method is applicable to both quick and slow tests, provided constant time increments are used. The ASTM "standard method" is therefore usually not applicable. Also, the number of monitored values are too few in the "standard test"; the interesting development could well appear between load increment number seven and eight and be lost.

Fig. 4 presents a method proposed by De Beer (1967) and De Beer & Wallays (1972), where the load movement values are plotted in a double logarithmic diagram. When the values fall on two approximately straight lines, the intersection of these defines the failure value. De Beer's method was originally proposed for slow tests.

Fig. 5 illustrates a method proposed by Brinch Hansen (1963), who defines failure as the load that gives twice the movement of the pile head as obtained for 90% of that load. This method, also called the 90% criterion, has gained widespread use in Scandinavia (Swedish Pile Commission, 1970). Brinch Hansen (1963) also proposes an 80% criterion defining the ultimate load as the load that gives four times the movement of the pile head as obtained for 80% of that load. The 80% criterion failure load can be estimated by extrapolation from the curve to be about 210 tons. (Some references have confused the 80% and 90% criteria, and use, erroneously, for the 80% criterion the movement of the 90% criterion).

In Fig. 6, Brinch Hansen's 80% criterion is shown in a plot — which is very similar to that of Chin — $\sqrt{\Delta}/P$ plotted against Δ). The ultimate failure value is determined from the criterion that a point, coordinates P_u Δ_u, on the curve is the point of ultimate failure when the point, coordinates $0.80P_u$ $0.25\Delta_u$, also lies on the load-movement curve. The criterion gives the following simple relationships to use in calculating the ultimate failure, P_u:

$$P_u = \frac{1}{2\sqrt{C_1 C_2}}$$

$$\Delta_u = \frac{C_2}{C_1}$$

where C_1 is the slope of the straight line and C_2 is the y-intercept in the $\sqrt{\Delta}/P$ plot, Fig. 6.

When using the Brinch Hansen 80% criterion, it is important to check that the point $0.80P_u/0.25\Delta_u$ indeed lies on the measured load-movement curve.

In the example case, P_u is 211 tons, which agrees well with the value extrapolated from the load-movement curve, directly.

Brinch Hansen's 80% criterion postulates that the load movement curve is approximately parabolic. Chin postulates that it is approximately hyperbolic. The shape of the actual curve is obviously close enough to both mathematical curves to allow both approximations. Brinch Hansen's 80% criterion results generally in a failure value about 10% lower than Chin's value. Note that both methods allow the latter part of the curve to be

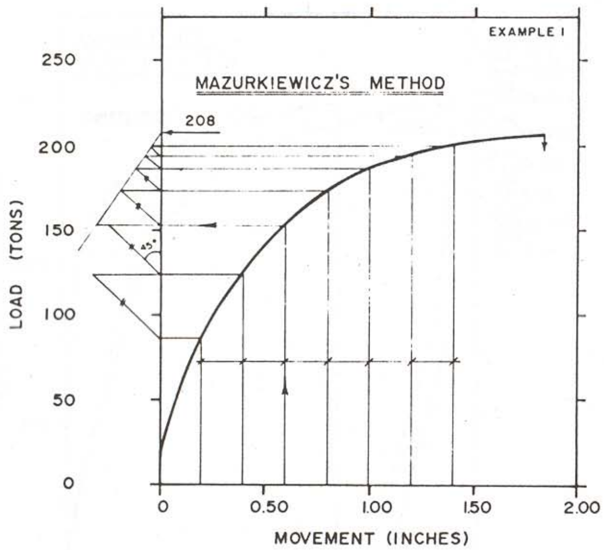


Fig. 7. Ultimate failure load according to Mazurkiewicz

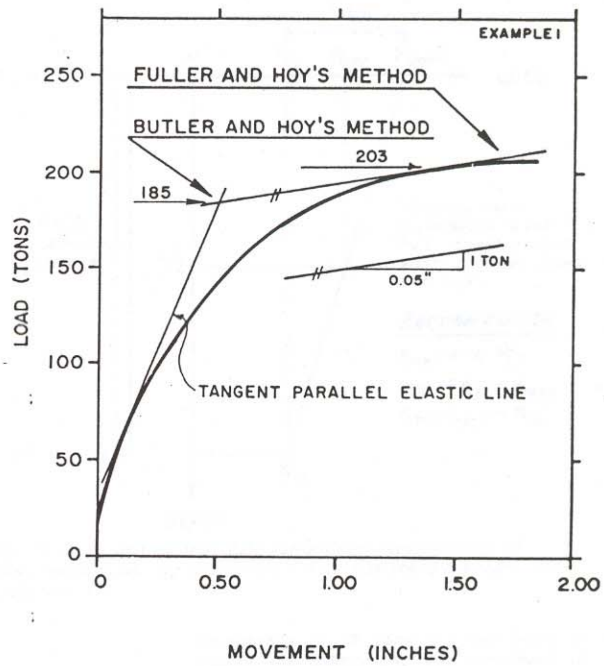


Fig. 8. Ultimate failure according to Fuller & Hoy and Butler & Hoy

plotted according to a mathematical relationship, and — which is often very tempting — they make an "exact" extrapolation of the curve possible. That is, it is easy to fool oneself into believing that the extrapolated part of the curve is as true as the measured.

In Fig. 7, the method put forward by Mazurkiewicz (1972) is illustrated. A series of equal pile head movement lines are arbitrarily chosen and the corresponding load lines are constructed from the intersections of the movement lines with the load-movement curve. From the intersection of each load line with the load axis, a 45° line is drawn to intersect with the next load line. These intersections fall, approvimately, on a straight line, the intersection of which with the load axis defines the failure load. Also, this method is based on the assumption that the load-movement curve is approximately parabolic. Consequently, the interpreted failure load of Mazurkiewicz's method is close to that of Brinch Hansen's 80% criterion. However, when drawing the line through the intersections according to Mazurkiewicz, some disturbing freedom of choice is usually found.

In Fig. 8 a simple definition proposed by Fuller & Hoy (1970) is shown. The failure load is equal to the test load for where the load movement curve is sloping 0.05in/ton (0.14mm/kN).

Fig. 8 also shows a development of the above definition proposed by Butler & Hoy (1977) defining the failure load as the load at the intersection of the tangent sloping 0.05in/ton, and the tangent to the initial straight portion of the curve, or to a line that is parallel to the rebound portion of the curve. As the latter portion is more or less parallel to the elastic line, (see Fig. 2), the author suggests that the intersection be that of a tangent parallel to the elastic line, instead.

The Fuller & Hoy method penalises the long pile, because the larger elastic movements occurring for a long pile, as opposed to a short pile, cause the slope of 0.05in/ton to occur sooner. The Butler & Hoy development takes the elastic deformations into account, substantially offsetting the length effect.

Fig. 9 shows the construction of the failure load as proposed by Vander Veen (1953). A value of the failure load, P_{ult}, is chosen and values calculated from $1n\ (1 - P/P_{ult})$ are plotted against the movement. When the plot becomes a straight line, the correct P_{ult} has been chosen. The Vander Veen method was proposed long before programmable pocket calculators were available. Without those, however, the application is very time-consuming.

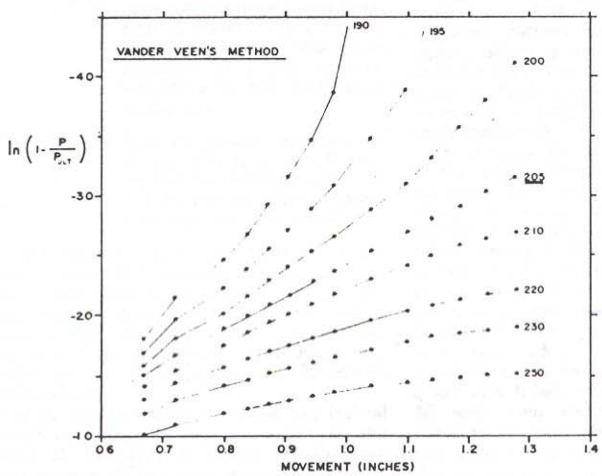


Fig. 9. Ultimate failure according to Vander Veen

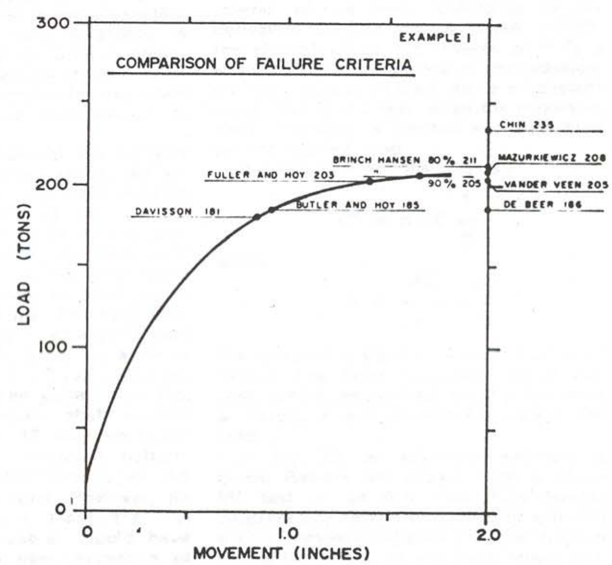


Fig. 10. Comparison of nine failure criteria

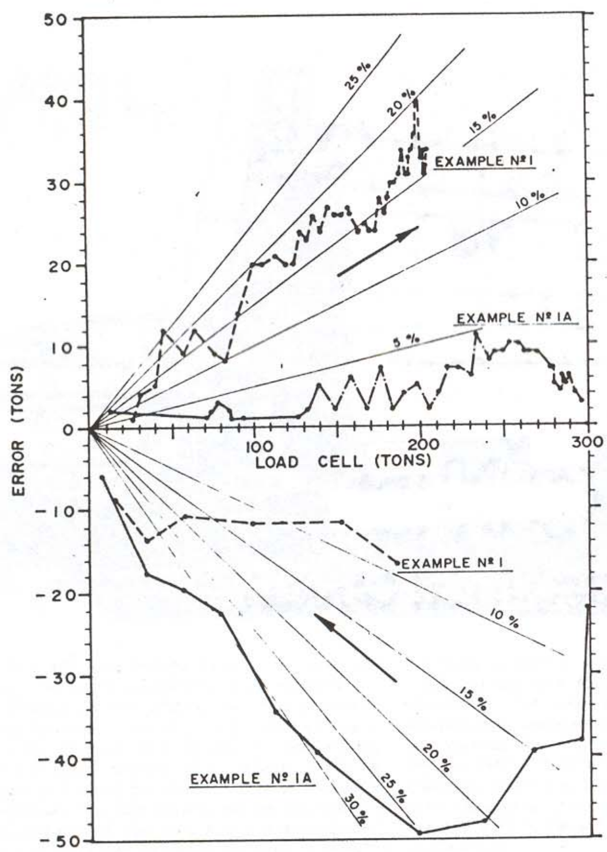


Fig. 11. Error in jack-load determined with manometer vs. correct load determined with load cell

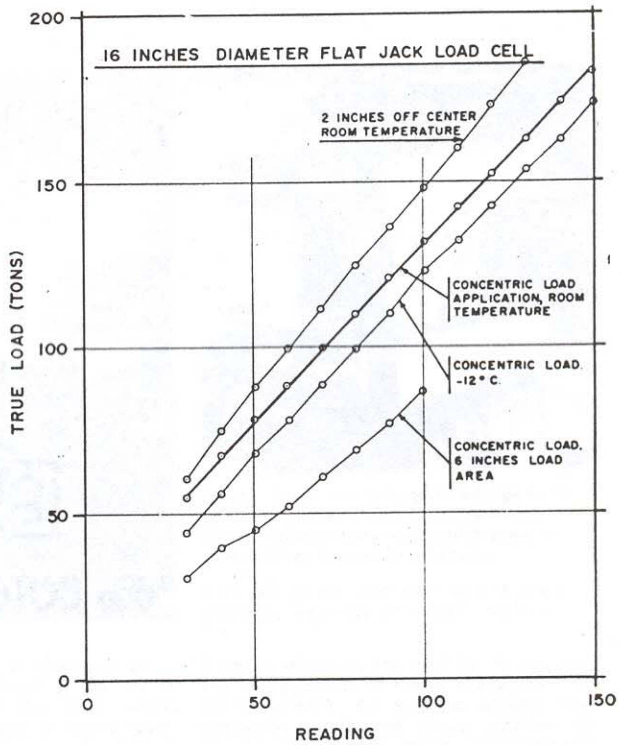


Fig. 12. Variations in load cell calibration due to eccentric load application, temperature change, and reduced loading area

In Fig. 10, the nine values determined above are plotted together. As shown, the Davisson limit of 181 tons is lower than all the others and the Chin value of 235 tons is the highest. The other seven values are grouped more or less together around an average of 200 tons.

It is difficult to make a rational choice of the best criterion to use, because the one preferred is heavily dependent on one's past experience. One of the main reasons for having a strict criterion is, after all, to enable a set of compatible reference cases to be established. The author prefers to use not one but three or four of the criteria given in Fig. 10. The preferred criteria are the Davisson limit load, the Chin failure load, the Brinch Hansen 80% criterion, and the Butler & Hoy failure load. In the case of an engineering report, the preference and experience of the receiver of the report may result in the use of one of the other criteria, in addition.

The Davisson limit is chosen because it has the tremendous merit of allowing the engineer, when proof testing a pile for a certain allowable load, to determine *in advance* the maximum allowable movement for this load with consideration of the length and size of the pile. Thus, as proposed by Fellenius (1975), contract specifications can be drawn up including an acceptance criterion for piles proof-tested according to quick testing methods. The specifications can simply call for a test to at least twice the design load, as usual, and declare that at a test load equal to a factor, *F*, times the design load, the movement shall be less than the elastic column compression of the pile, plus 0.15in, plus a value equal to the diameter divided by 120. The factor *F* is a safety factor and should be chosen to a value of 1.5 to 1.8 depending on circumstances.

The Chin method is chosen because it allows a continuous check on the test, if a plot is made as the test proceeds, and a prediction of the maximum load that will be applied during the test. Sudden kinks or slope changes in the Chin line indicate that something is amiss with either the pile or with the test arrangement. The Chin value has the additional advantage of being less sensitive to imprecisions of the load and movement values.

The Brinch Hansen 80% criterion is chosen because it usually gives a P_u value which is close to what one subjectively accepts as the true ultimate failure value. The value is smaller than the Chin value. However, the criterion is more sensitive to inaccuracies of the test data than is the Chin criterion.

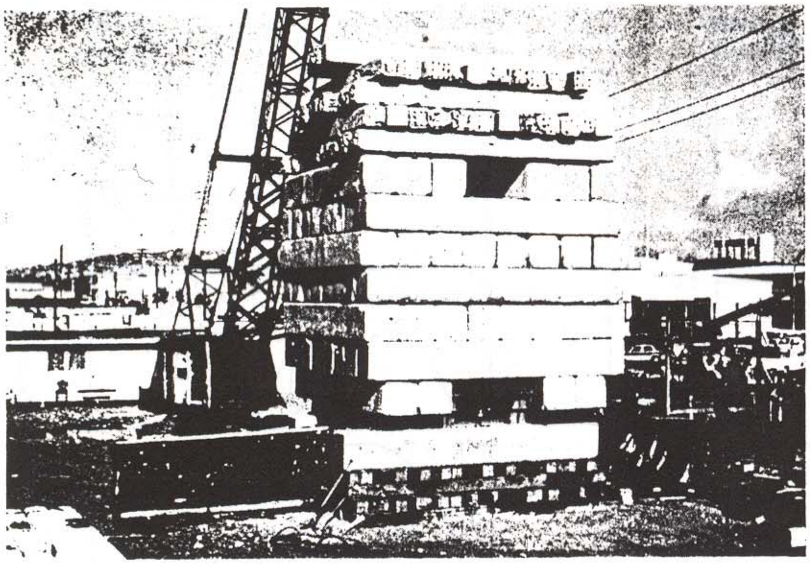
The Butler & Hoy method is chosen primarily because of its resemblance to the Davisson method. In some cases a Davisson limit load can be obtained without the interpreter being willing to accept intuitively that the pile has reached failure. (In such cases, the Chin value will be much higher than the Davisson limit). However, the Butler & Hoy slope of 0.05in/ton is not approached unless failure is imminent, and absence of a Butler & Hoy failure indicates — in addition to a high Chin value — that the Davisson value is imprecise. The reasons for the latter can be wrongly chosen values of pile elastic modulus or pile length, or imprecise or erroneous load or movement values. Also the Butler & Hoy method permits an acceptance criterion for proof-tested piles to be formulated and included in the specifications. However, the Butler & Hoy method requires the pile head movement to be large enough to reach the Fuller & Hoy point, which restricts the use of the definition in this context.

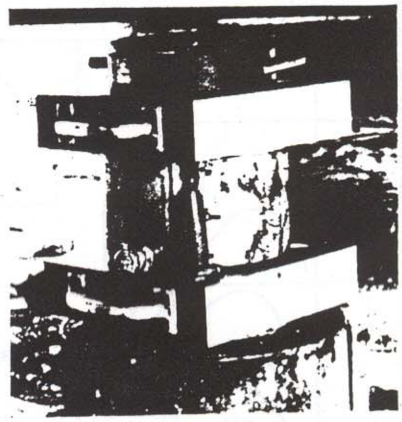
Influence of errors

The test results shown in Fig. 1 and used in the preceding discussions are from a test where an electrical strain gauge load cell was used to determine the load applied on the pile. In the test, the pressure in the jack was monitored by means of a manometer, which had been calibrated together with the jack. Yet the load determined from the manometer readings was inaccurate. Fig. 11 shows the difference between the load determined from the jack pressure and the load determined by the load cell, as plotted against the load cell load.

The error (overestimation) in the jack pressure load is substantial and varies between 10 and 25%, being mostly 15-20%. In unloading the pile, the error was much smaller. This is not the worst, nor the best, example the author has met, but is a typical case for the equipment used in the industry of today.

Fig. 11 also shows similar results from another test, called Example IA, when in loading the error was less than 5%. On the other hand, the error in unloading was large. This seems to have involved jacking equipment of a much better quality than that used in Example 1. However, Example 1A is from an identical pile located about 20ft (6m) away at the





(Left). A 650 ton kentledge arrangement (for testing 200ft long 16.5in concrete piles). Note that the measuring beam is shielded from sunshine and wind

(Above). Load cell and swivel plate (spherical bearing) on a hydraulic jack

same site and tested two days later using the same equipment and method.

Based on the above and many similar measurement results, the author concludes that if one wants to ensure an imprecision smaller than about 20%, a load cell must be used. The jack and jack pressure are too erratic to be reliable. A calibration of the jack and manometer for one pile is not relevant to even a neighbouring test pile. The reason for the unreliability is that the system is being required to do two things at the same time; both provide the load and measure it, and load cells with moving parts are considerably less reliable than those without. Calibrating testing equipment in the laboratory ensures that no eccentric loadings, bending moments, or temperature variations influence the calibration. However, in the field, all these factors are present to influence the test results to an unknown extent, unless a load cell is used.

Naturally, many structures are safely supported on piles which have been tested with erroneous loads, and as long as we are content to stay with the old rules, loads and piling systems, we do not need to improve the precision. The error is included in the safety factor. That is why factors as large as 2.0 and 2.5 are applied and such numbers are really more ignorance factors than safety factors. However, if we want to economise and continue to increase allowable loads as geotechnical knowledge increases, we cannot accept potential errors as large as 20 to 25%. In the author's opinion, we cannot accept errors exceeding 10%, and this requirement necessitates the use of load cells.

However, the fact that a load cell is used is no guarantee for precise loads. Fig. 12 shows calibrations performed on a flat-jack load cell under varying conditions. The heavy centre line is a regular calibration curve obtained when using 3in (76mm) thick full-width steel plates on both sides of the load cell and applying the load through a spherical bearing (swivel plate). This curve is readily repeatable. However, by moving the load only 2in (51mm) off-centre, a different calibration was obtained. By letting the temperature drop, a third line was obtained. The greatest influence was obtained by removing the steel plates and loading only a centre area of the load cell.

Of course, the load cell of Fig. 12 is unsuitable for use in the field, where temperature variations and eccentric loading cannot be avoided. In a load test, the geometric centre does not necessarily coincide with the load centre. Therefore, it is necessary to check the calibration of the cell and its sensitivity to eccentric load application.

The foregoing discussion has dealt with the imprecision of the load value. But the precision of the movement values can also be critical. If the "failure" criterion is a maximum settlement of 1.75in (44mm), an error of 0.25in (6.5mm) is of no consequence when the maximum movement recorded is 1.5in (38mm) or less, which it is on most proof testing occasions. However, errors of this degree of magnitude greatly influence the shape of the curve and the various methods of interpretation of failure loads. In particular, Davisson's limit is sensitive to these errors.

It must be remembered that the minimum distances from the supports of measuring beam to the pile and the platform etc., as recommended in the ASTM Designation, are really minimum values; generally they give rise to errors of little concern for ordinary testing, but they are too close for research or investigative testing purposes.

One of the greatest villains for spoiling a load test is the sun. The measuring beam must be shielded from sunshine at all times.

Analysis of results using tell-tale data

Fig. 13 shows the results from a Quick ML test on a 130ft long (40m) 12in (300mm) precast concrete pile. The pile had a total cross-section of 124in² (800cm²), the area of steel reinforcement was 1.9in² (12cm²), and the pile circumference was 41in (107cm). The pile was loaded in steps of 22.4 tons, and a load cell was used to determine the test load. The failure loads evaluated in accordance with the nine methods are given in the graph. Scatter of the values is similar to that shown in Fig. 10.

In the test, a centre pipe had been cast in the pile allowing a tell-tale to be inserted down to the pile tip to monitor the compression of the pile and the pile tip movement. As will be shown, this relatively simple and cheap addition to the test arrangement greatly enhanced the value of the test results.

The graph in Fig. 13 also shows the movement of the pile tip and the measured compression of the pile. After a load of 70-90 tons, the measured compression plots in a straight line, indicating that the part of the added load used for overcoming shaft resistance is constant. It would be highly improbable that the constant value is other than zero. Therefore, the applied additional load remains unreduced by shaft friction straight down to the pile tip, and the slope of the compression line is equal to the slope of the elastic line. The combined elastic modulus of the pile determined from this slope is 5.1×10^6 psi (35 000 Mpa).

According to a method of analysis proposed by Trow (1967), the pile tip starts to move when the elastic line becomes tangential to the load movement curve of the pile head, and the load applied thereafter goes straight to the pile tip. The analysis by Trow is valid for a linear, i.e. triangular or rectangular, distribution of shaft resistance. The test results presented in Fig. 13 show that at a load of about 70-90 tons, the elastic line, established from the measured compression, becomes parallel to the load-movement curve. Consequently, according to Trow's method of analysis, the shaft friction must be approximately linearly distributed, and the shaft friction value cannot be greater than about 70-90 tons.

When assuming constant unit shaft friction, i.e. rectangular shaft resistance, the distribution of load in the pile becomes linear and from knowledge of the compression of the pile, Fellenius (1969) has shown that simple relations can be established for the load at the pile end and the total shaft resistance, as shown in Fig. 14. At pile head loads of 224, 246 and 280 tons, the measured compressions were 0.96, 1.07 and 1.24in, respectively. The values result in calculated pile tip loads of 160, 182 and 216 tons, respectively. The corresponding calculated pile shaft resistance was 64 tons for all three pile loads.

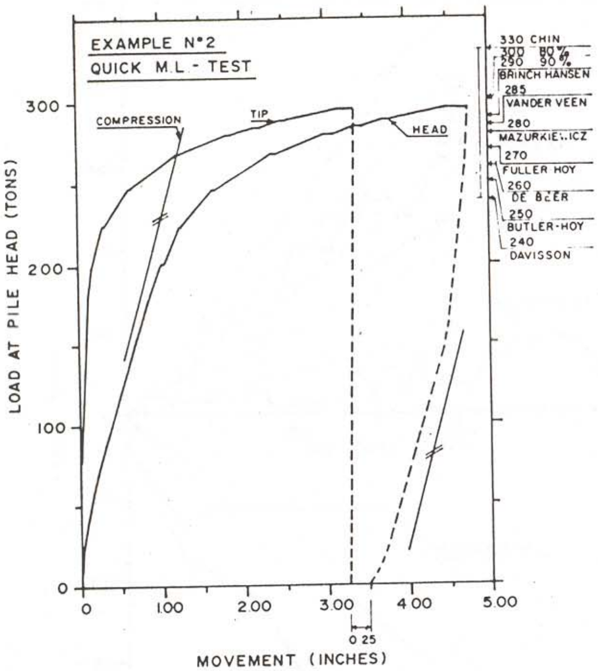


Fig. 13. Load-movement diagram from Quick M.L. test with measurement of pile tip movement. Example 2

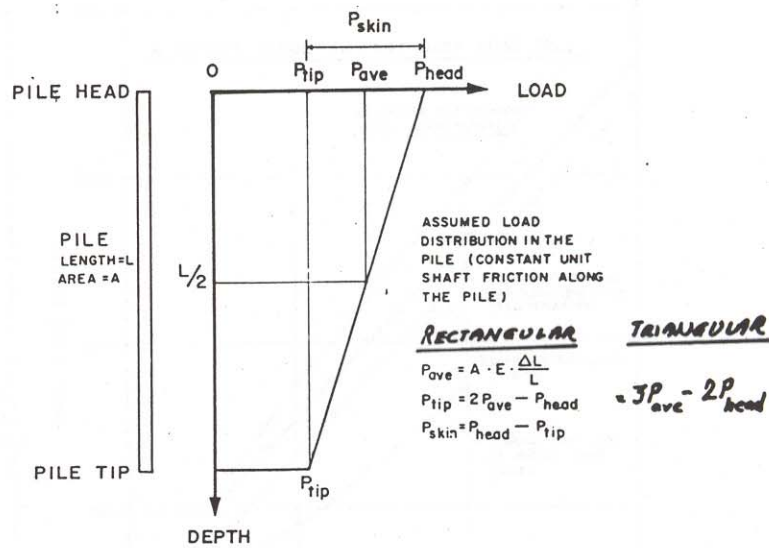


Fig. 14. Shaft and tip load calculation from measurement of pile compression (tip movement) when assuming constant unit shaft friction

The value of 64 tons is less than the previously established maximum possible of about 70-90 tons. For many reasons, it is probable that the unit shaft resistance is not constant. Recently, Leonards & Lovell (1978) proposed a method of analysis using measured pile compression, which allows a variety of distributions of shaft resistance to be tried in the analysing of the test data.

Leonards & Lovell established the following relations:

$$\alpha = \frac{C' - C}{1 - C}$$

where

α = ratio between the pile tip load and the load applied to the pile head ($P_{tip} = \alpha \times P$).

C' = ratio of measured compression to column compression, the latter being the compression of a free column subjected to the same load as the pile.

C = ratio of elastic compression of the pile at a load P supported totally by shaft friction to the column compression for the same load.

The ratio C' is known from the measured data. The purpose of the analysis is, either from knowledge of α, i.e. the tip load, to determine C, i.e. the relative distribution of shaft resistance, or, inversely, from knowledge of relative distribution of shaft resistance determine the tip load.

It is not necessary to know the factual shaft resistance in order to establish the ratio C. Leonards & Lovell (1978) have determined C for two principal patterns of shaft friction, and these are presented in the nomograms in Figs. 15 and 16. The case of constant unit shaft resistance is a special case of the nomogram of Fig. 15; the two friction values are equal and C is 0.5. The previously mentioned three loads and average loads give values of α of 0.714, 0.740 and 0.772, respectively. Insertion in the Leonards & Lovell relation gives values of the tip loads, which are equal to the ones calculated previously, i.e. 160, 182 and 216 tons.

The nomogram of Fig. 16 is applicable when assuming a triangular distribution of shaft resistance. In this case, the ratio C becomes 0.667 and the calculated values of α are 0.571, 0.610 and 0.658 respectively, resulting in the corresponding pile tip loads of 128, 150 and 184 tons, and a shaft resistance of 96 tons for all three loads.

In these days of the pocket calculator, it is easier to work with the equations directly, as opposed to using nomograms. In Fig. 17, the equations behind the nomograms in Figs. 2 and 3 are presented. A third pattern of shaft resistance is added, which is useful for piles in homogeneous clay. The reduction of shaft resistance at depth λ is intended for use when analysing a progressive mobilisation of shaft resistance.

To discuss the results of the analysis of the load test presented above, the assumption of constant unit shaft friction along the entire length of the pile resulting in a shaft load of 64 tons is probably incorrect. However, the shaft load of 96 tons calculated on the assumption of triangular distribution of shaft resistance is greater than the maximum possible shaft load. To arrive at a shaft load in between 64 and 96 tons, the analysis could be repeated with a C ratio between 0.500 and 0.667, chosen either from Fig. 15 with two rectangular shaft friction patterns or from Fig. 16 with an upper triangular and a lower rectangular pattern. For instance, C = 0.58 determines the shaft load to be 76 tons. However, no justification is available for further refinement. Such justification would have been, for instance, a definite change of soil profile at some depth. Also, the load increments of 22 tons are too large to justify the refinement. An increment of 10 tons, instead, would have shown much more precisely the load-movement development during the first 100 tons of applied load.

Lacking adequate soil data, the true tip and shaft loads cannot be determined. However, they lie somewhere in between the mentioned figures. The results of the complete analysis have been plotted in Fig. 18 showing the load-movement curves for the tip and the shaft (*vs* head movement) for the two extreme distributions of the shaft resistance. Detailed knowledge of the soil profile could narrow the ranges. However, for most practical purposes, determining the shaft resistance to be somewhere in between 64 and 96 tons, as in the subject case, is good enough.

The C' value has additional analytical significance. The ratio C' is plotted in Fig. 19, both as a function of the load at the pile head, and as a function of the inverse of the load. According to the derivation by Leonards & Lovell (1978), the plot of C' *vs* the inverse of P is a straight line, if the change of compression, $d\Delta$, for a change of load, dp, is a constant value. This is the case when the maximum shaft resistance is reached and surpassed by the applied load.

The equation for the line is:

$$C' = n - K \frac{1}{P}$$

where

$$n = \frac{AE}{L} \times \frac{d\Delta}{dp}$$

The factor n is equal to 1 only if all shaft friction has been mobilised, as in this case, which, as pointed out by Leonards & Lovell, is not necessarily always the case.

In Fig. 20, an additional example is given. Results are shown from a Quick ML test on an 84ft long, 12in precast concrete pile driven through clay and into a very competent glacial till. The diagram shows the measured pile head movement, pile compression, and the head load versus

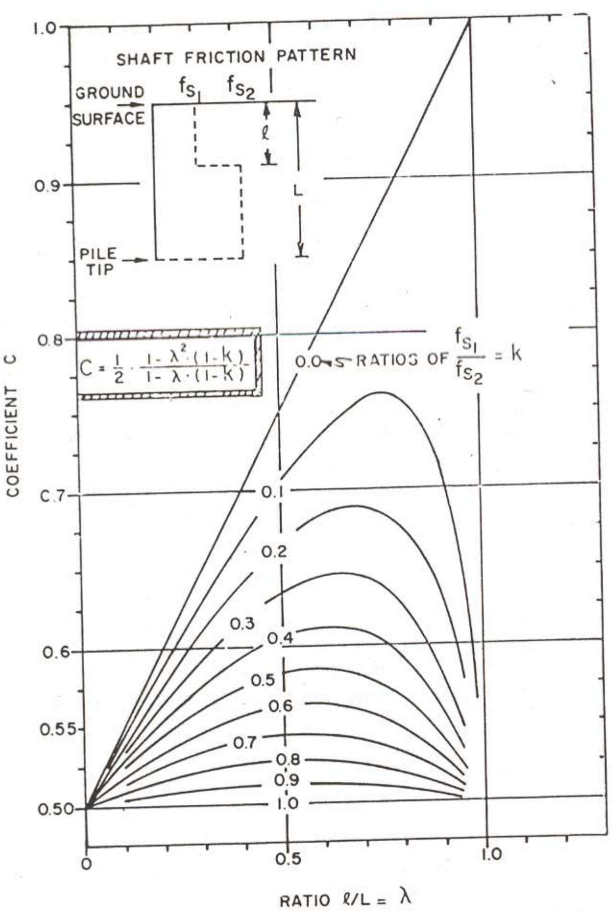


Fig. 15. Coefficient C for various distributions of unit shaft friction

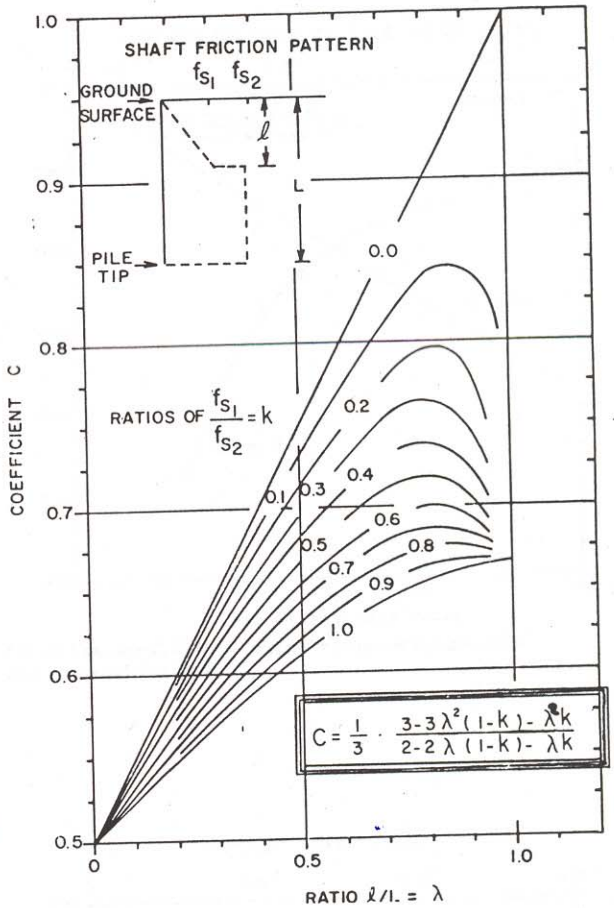


Fig. 16. Coefficient C for various distributions of unit shaft friction

measured tip movement, plus the calculated shaft resistance and tip load. The calculations are performed assuming that the distribution of shaft resistance follows the third pattern in Fig. 17 with $l = L$, and k equal to the same ratio of shear resistance as found by vane shear testing. (For details on the soil profile, and an older, much more time-consuming and arbitrary, method of analysis of the test results, see Fellenius & Samson, 1976).

The pile test is not carried to ultimate failure. However, the Leonards-Lovell method of analysis of the simple telltale measurements of the tip movement makes it possible to establish that the maximum shaft resistance acting along the pile is 240 tons and the maximum tip load mobilised is 185 tons. Indeed, this is a result well worth the expenditure of a bit of time and money.

As in the previous example, additional supporting information is gained from a plot of C' versus $1/P$ for the results, and also C' versus P. As seen in Fig. 21, when going from right to left in the diagram, the C' *vs* $1/P$ is at first a curved line later becoming a straight line pointing to $C' = 1.00$, $1/P = 0$. The P value for when the curve becomes a straight line determines the point (load) where all shaft friction is mobilised, in this case for $P =$ 300 tons.

The results of the second test are almost certainly affected by a residual load caused by the reconsolidation of the clay after driving and estimated to be about 20 tons. This load acts on the pile before the start of the test loading and, as a result, the compression of the pile does not start from zero, but has an initial value of about 0.08in. Adjusting the C' values accordingly, and recalculating the results show that the adjusted maximum shaft resistance is 210 tons and the maximum tip load is 235 tons. The difference in calculated tip load is 50 tons, 2.5 times the estimated residual load.

The above shows how sensitive the method of analysis is to residual loads, and, therefore, also to inaccuracies of the measurements. However, the old subjective methods are actually even more sensitive, but because of their arbitrary nature, this is not always evident. In contrast, the Leonards & Lovell method allows a determination of the extent of uncertainties influencing a test. It provides, therefore, the engineer interpreting the data with a zone of reliability and a confidence he would otherwise have felt only because of his ignorance of the uncertainties involved and of their effects.

To take full advantage of the Leonards & Lovell method of analysis, the testing method should not be the "standard loading procedure", which for all technical and economical purposes is the worst method to use, but the "Quick load test method". The load increments must be applied at constant intervals (for practical reasons usually 5 minutes, to allow for at least three readings per increment). The increments should be small enough to allow for at least 30 or 40 increments before

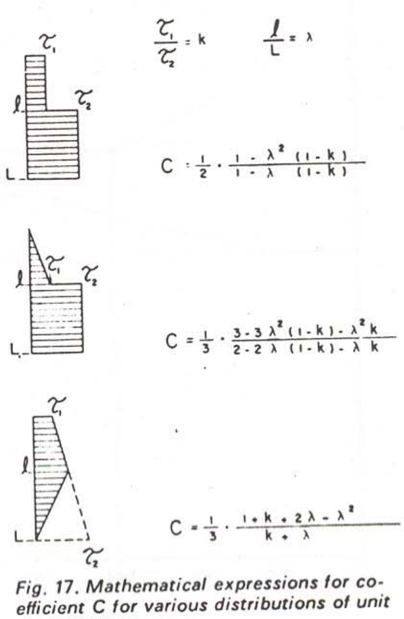


Fig. 17. Mathematical expressions for coefficient C for various distributions of unit shaft friction

reaching the maximum test load. Naturally, a reliable load cell should be used to supplement the jack manometer, and every effort must be made to ensure reliable movement values.

Fig. 22 shows another method of pre-

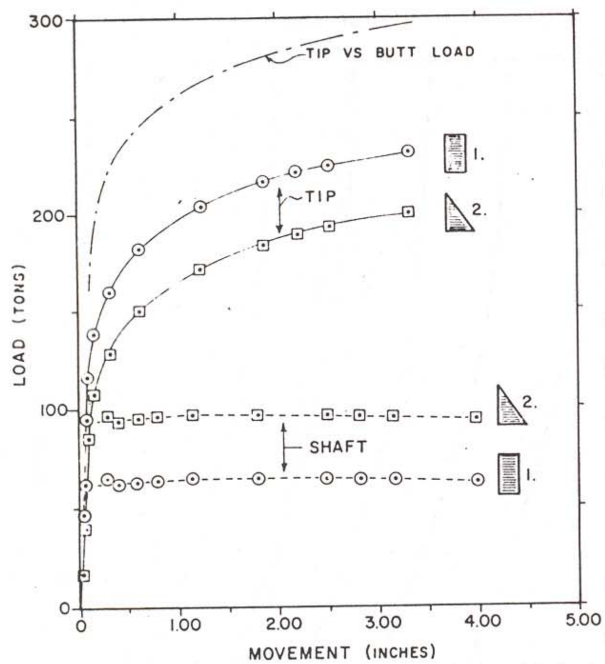


Fig. 18. Example 2. Load-movement diagram for shaft and tip loads

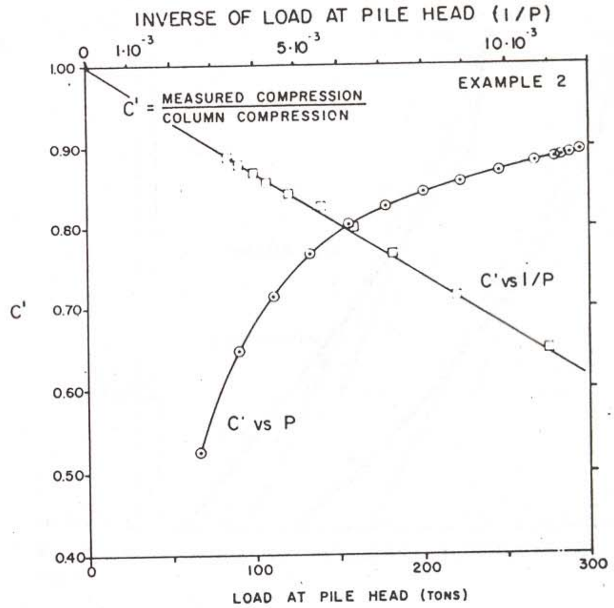


Fig. 19 Example 2. Ratio C' plotted vs load at the pile head and vs the inverse load

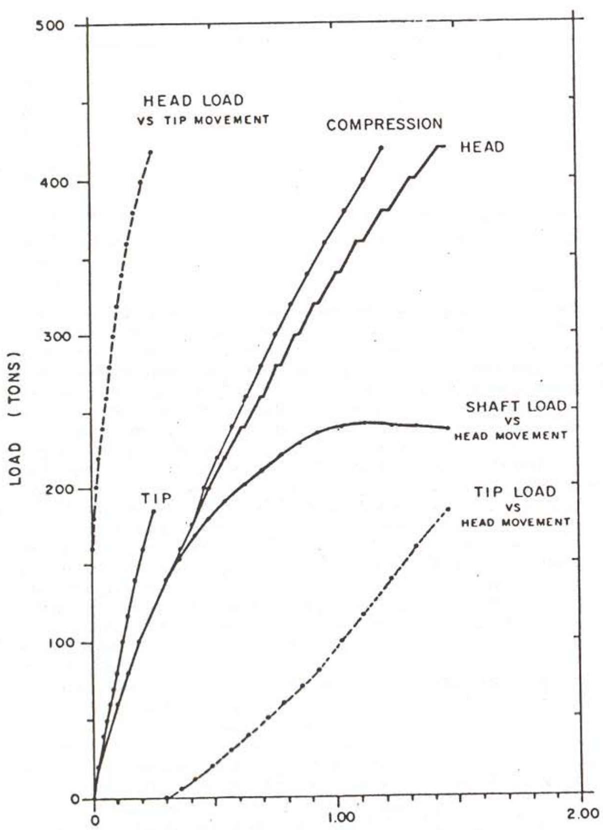


Fig. 20. Example 3. Load-movement diagram from Quick M.L. test with measurement of pile tip movement

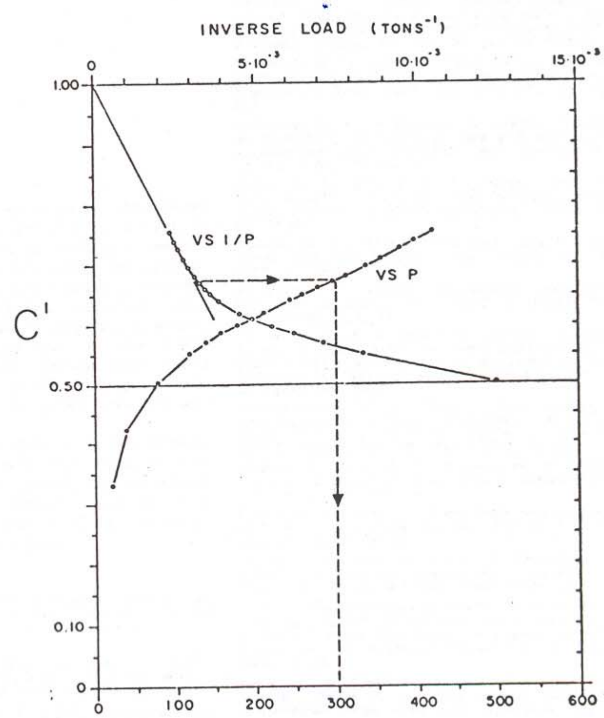


Fig. 21. Example 3. Ratio C' plotted vs load at the pile head and vs the inverse load

senting the results of an analysis of a pile test (Example 2). The load distribution in the pile is shown for three different loads at the pile head. The straight line represents the load distribution for constant unit shaft resistance (rectangular distribution) and the curved line that for a triangular distribution of shaft resistance. The interesting point in this graph is that to fulfill the condition that both load distributions give the same average load in the pile, the two areas *A'* and *A''* must be equal.

The condition indicated in Fig. 12 can be developed to determine non-linear load

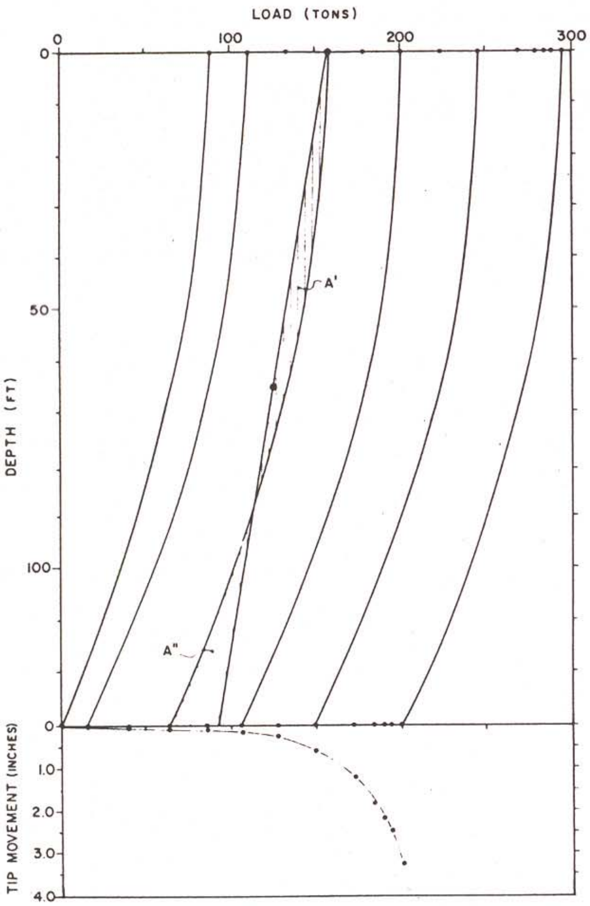


Fig. 22. Example 2. Vertical load distributions in the pile during the test

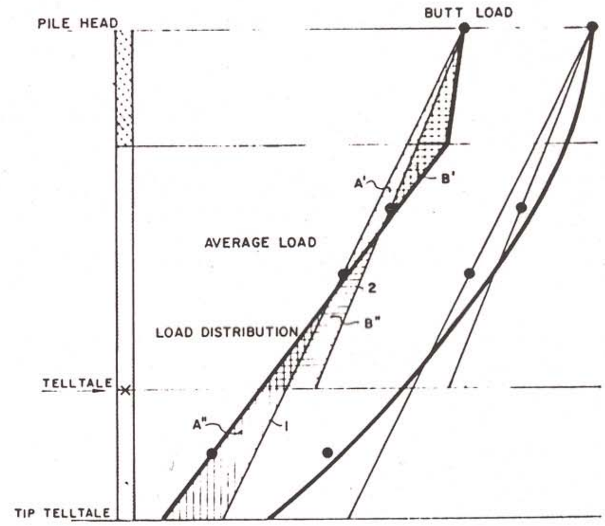


Fig. 23. Determination of load distribution from tell-tale measurements

distributions in test piles, as proposed by Fellenius (1969) — for instance, in a two-layer soil profile comprising backfill and sand, where two tell-tales have been inserted in the pile. In Fig. 23, the diagram shows one tell-tale placed at the pile tip and a second one located some distance above. The measurements of the tell-tales give three values of average load in the pile, as marked in the graph (filled circles ●). The straight lines *1* and *2* from the pile head load value through the two upper average loads, plotted at mid-points between the tell-tale locations and the pile head, will be mathematically possible load distributions considering one tell-tale at a time. The true load distribution line must fulfill the conditions that the areas, *A'* and *A"* and *B'* and *B"*, between the true line and the mathematical lines must be equal. In the graph, the "true" load distribution has first been assumed to consist of two straight lines, which necessitates the load distribution line passing through the bottom average load. It also means that constant unit shaft resistance has been assumed. A triangular distribution of unit shaft resistance results in the curved load distribution line shown to the right with the same geometric conditions.

In the analysis of Examples 2 and *3*, use was made of measurements of tip movement (pile compression). The increase to the total cost of the test was minimal. The purpose of this presentation has been to demonstrate the tremendous value that measurements of tip movement can provide in the analysis of a test. It is an addition that is strongly recommended also for routine tests. For closed-end steel tube piles, it is simple to arrange. For precast prestressed concrete piles it requires a little bit of advance planning so that a centre tube can be cast in the pile. H-piles could necessitate some field welding and a few hours of preparation. Cast-in-place piles are not excluded.

In view of the simplicity and low relative cost coupled with the large amount of extra information gained, by no means exhausted in this Paper, there is little excuse for not incorporating tip measurements even in routine tests.

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