This copy is available for loan.

THE UNIVERSITY OF ALBERTA

A Study of Cast-In-Place Concrete Friction Piles in the Edmonton Area

by

Randy Hillaby

A REPORT

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF Masters of Engineering

IN

Civil Engineering

Civil Engineering

EDMONTON, ALBERTA

Fall 1983

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a report entitled A Study of Cast-In-Place Concrete Friction Piles in the Edmonton Area submitted by Randy Hillaby in partial fulfilment of the requirements for the degree of Masters of Engineering in Civil Engineering.

....

Supervisor

....

....

Date............................

THE UNIVERSITY OF ALBERTA

RELEASE FORM

NAME OF AUTHOR Randy Hillaby

TITLE OF REPORT A Study of Cast-In-Place Concrete Friction Piles in the Edmonton Area

DEGREE FOR WHICH REPORT WAS PRESENTED Masters of Engineering

YEAR THIS DEGREE GRANTED Fall 1983

(SIGNED) ....

PERMANENT ADDRESS:

....

....

....

DATED 19

Abstract

This report is intended to provide the design methods for estimating the shaft carrying capacity and settlement of cast-in-place concrete friction piles. The results from eleven pile load tests from four different sites in the Edmonton area have been back analysed in order to evaluate the design methods presented herein.

The conventional total stress method of estimating the shaft carrying capacity, based on empirical evidence, has been evaluated by comparing the calculated α values to the local average reported value. The more modernistic effective stress method of estimating the shaft carrying capacity, based on fundamental soil mechanics, has also been evaluated by comparing the calculated β values to the average reported values.

The settlements of these cast-in-place concrete friction piles have been evaluated at a total load corresponding to the full shaft carrying capacity and the base load being one third of the ultimate base load. These settlements and related K values are believed to be the upper limit for piles loaded to design capacity in the Edmonton area.

The results of the back analyses suggest that the design methods are valid within reasonable limits for the estimation of shaft carrying capacity and settlement of cast-in-place concrete friction piles.

Table of Contents

Chapter 1

Introduction

Cast in place concrete friction piles constructed by drilling a cylindrical hole and subsequently filling it with concrete, can be designed to resist large axial loads. Foundations of this type have been in use for many years with the first drilled shaft by machine occuring in the 1920's (Greer, 1969). At the present time, cast-in-place concrete friction piles are being installed virtually all around the world. These friction piles prove to be most feasible in ground conditions where adequate skin resistance can be developed and end bearing resistance is of lesser importance.

It is the purpose of this report to present the design theory for estimating the shaft carrying capacity and settlement of cast-in-place concrete friction piles. The total and effective stress methods of design will be analysed for their validity, by examining pile load test results from four sites in the Edmonton area.

Chapter 2

Design Theory

2.1 General

Friction piles resist applied loads predominantly by mobilizing frictional resistance on vertical shaft surfaces and to a much lesser extent by mobilizing end bearing resistance. Therefore the carrying capacity of these piles depends considerably on the surface area of the shaft and on the soil parameters surrounding the shaft. Research on pile/soil interaction has been carried out by separating the shaft and base components of the total pile resistance (Whitaker and Cooke, 1966). Using this technique it was possible to measure the individual development of shaft and base resistance as the pile was loaded and settlement occured. It was found that the full mobilization of the shaft and base components of resistance occured at different amounts of pile settlement. The frictional resistance along the shaft develops quickly and linearly to full mobilization and then remains constant for any additional settlement the pile undergoes. Full mobilization of the shaft resistance occurs when the settlement is 0.5 to 2.0 percent of the shaft diameter. The base resistance on the other hand, is not fully mobilized untill settlement reaches at least 5.0 percent of the base diameter.

Current specifications for pile load tests in which the pile design is based, usually only recommends the piles to be loaded to 200 percent of their design load and frequently only a small amount of settlement has taken place. Consequently with this small settlement, the shaft resistance has been fully mobilized and the end bearing resistance is minimal. Hence, the design theory herein will concentrate on the shaft carrying capacity of friction piles in terms of total and effective stress methods.

2.2 Total Stress Method

The conventional approach for estimating the shaft carrying capacity of a friction pile is based on empirical evidence using the total stress method outlined by Burland and Cooke(1974). It has been the belief of many engineers that the interaction between the pile and surrounding soil is too complex to be studied in a theoretical manner, and therefore the total stress method based on pile load testing has been adopted. This method of design has proven to be very useful when applied to specific ground conditions and pile types for which empirical evidence exists.

The total stress method makes use of the average undrained shear strength of the soil(Cu) multiplied by an empirical coefficient(α) to estimate the average shaft adhesion(Ca) along the length of the pile.

$$Ca = \alpha Cu$$

The maximum shaft resistance for the entire length of the pile with a diameter D and a length L is given by:

$$R = (\pi DL)C\alpha$$

The estimation of the maximum shaft resistance is only as accurate as the prediction of the input soil parameters, and therefore these parameters should be discussed in some detail.

The average undrained shear strength(Cu) of the soil along the pile shaft should be determined from a plot of undrained shear strength against depth. The determination of a suitable Cu value may prove to be difficult due to the scatter of test results in many soil types, including the Edmonton clays and tills. It is questionable if the undrained shear strength of the soil should even be used, since drained or partly drained conditions may exist in the soil surrounding the pile shaft. These drained conditions have been verified by Cooke and Price(1973) where they showed that the major shear distortion is confined to a relatively thin zone around the pile shaft. Therefore, with a thin drainage zone, pore pressures will dissipate much quicker during loading and produce drained conditions.

The empirical coefficient(α) depends on the soil type and also accounts for the ground disturbance caused by the pile installation. Pile load tests should be performed to determine the α value for the conditions of the specific project. However, on the basis of a large number of pile load tests performed in the same general area, it is possible to assign ranges of α values to particular pile types in various ground conditions. The α value can vary from 0.3 to 1.5 (Burland, 1973) and even for a given set of site conditions it can be variable. This large variation of α values warrants high factors of safety to be used in design, possibly yielding highly conservative results. Pile load tests performed on cast-in-place concrete piles in London clay were examined by Skempton(1959) and it was concluded that the α value generally ranged from 0.3 to 0.6 with a suggested average of 0.45. Skempton attributed the less-than-unity α value to water entering the pile excavation from both the groundwater and the wet concrete and causing softening of the soil adjacent to the pile. Disturbance of the soil during the excavation procedure is believed to have a greater influence on reducing the α value then the softening effect. The change in ground stresses due to the pile installation procedure may also have an influence on the α value.

The α value is questionable when it is assumed to be constant over the full length of the pile. This is due to the fact that the α value is zero at surface and then increases with depth as the lateral stresses on the pile increase. If a constant α value, of say 0.45, is used for shallow piles the skin friction may be overestimated.

It is clearly evident that this method utilizes valuable practicle experience in estimating the shaft carrying capacity of piles (Thomson, 1980). However, problems can arise if this method is used in new or unusual ground conditions where no previous piling work has been done. Therefore good engineering judgement must be exercised which requires an understanding of the theoretical principles in terms of effective stress.

2.3 Effective Stress Method

The effective stress method of estimating the shaft carrying capacity of piles is not a new principle and has been studied by Johannessen and Bjerrum(1965), Chandler(1968), Burland(1973) and Parry and Swain(1977). The principle of effective stress has been consistent throughout these studies and the reason its application has been hindered is the indecisiveness in predicting the input soil parameters and pore pressures. However, engineers today have a better understanding of these variables and hence the effective stress method should be utilized.

The theory on which the effective stress method is based envisions the soil comprising a compressible soil skeleton of solid particles enclosing voids filled with air and water. Below the water table the voids in the soil are considered to be saturated with water. Shear stresses cannot be resisted by the water and therefore must be resisted entirely by the soil skeleton. The stress carried by the soil skeleton is termed the effective stress(σ') and is given by the difference between the total stress of the soil(σ) and the pore water pressure in the voids(u).

$$\sigma' = \sigma - u$$

The maximum drained shear(τ) that can be carried along any plane within a soil mass is given by:

$$\tau = C' + \sigma'\tan\phi'$$

where: C' is the effective cohesion and ϕ' is the effective angle of shearing resistance along the plane.

When a pile is loaded the major shear distortion is confined to a relatively thin zone around the pile shaft (Cooke and Price,1973). If the rate of loading is slow enough to maintain drained conditions the lateral stresses on the pile shaft will be the horizontal effective stress(σ_h'). The maximum drained shear is then equivalent to the shaft friction which is given by:

$$F = C' + \sigma_h'\tan\phi'$$

The effective cohesion(C') is assumed to reduce to a negligible quantity due to softening of the soil in this thin zone as water is absorbed from both the groundwater and the wet concrete. The shaft friction then reduces to:

$$F = \sigma_h'\tan\phi'$$

The horizontal effective stress(σ_h') is related to the vertical effective overburden pressure(P') by the coefficient of earth pressure at rest(K_o).

$$\sigma_h' = K_o P'$$

$$P' = \gamma_s h - \gamma_w d$$

where:

γ_s is the bulk density of the soil.

h is the depth below ground surface.

γ_w is the density of water.

d is the depth below the groundwater table.

The shaft friction at any point along the pile shaft is given by:

$$F = K_o P' \tan\phi'$$

A unique relationship between the shaft friction and the vertical effective overburden pressure can be denoted by β:

$$F/P' = \beta = K_o \tan\phi'$$

The maximum shaft resistance for the entire length of the pile with a diameter D and a length L is given by:

$$R = (\pi DL)P'\beta$$

The estimation of the maximum shaft resistance for this method is dependent upon the best prediction of the input soil parameters and pore pressures, which will be discussed in some detail.

The vertical effective overburden pressure(P') can be calculated from the bulk density of the soil, taking the pore pressures into account. Since pore pressures are usually not measured for a typical site investigation, a static groundwater table may be assumed for this calculation. Predictions of the vertical effective overburden pressure can be made with fairly good precision, with the greatest uncertainty occuring in assuming the pore water pressure in the soil.

The β value accounts for the frictional forces along the pile, as well as for the ratio of horizontal to vertical soil stresses acting along the pile. Average β values can also be calculated from pile load tests where the average vertical effective overburden pressure and the maximum shaft resistance are known. Values of β obtained by Meyerhof(1976) from pile load tests in stiff clay showed scattered β values, generally ranging from 0.3 to 2.5. Burland(1974) suggests that for bored piles in stiff clay an average β value of 0.8 would be reasonable. The β value is similar to the empirical coefficient(α) in the total stress method with the difference being that β is related to the soil parameters K_o and ϕ'.

The coefficient of earth pressure at rest(K_o) depends on the soil type and its stress history and also on the pile installation technique. Earth pressures at rest have been studied experimentally by Brooker and Ireland(1965) who showed that K_o depended on the over-consolidation ratio (measure of stress history), effective angle of shearing resistance and the plasticity of the soil. Earth pressures at rest(K_o) conditions exist in the ground prior to installation of the pile. These K_o conditions will then be relieved as the excavation is being carried out. Once the pile concrete is placed in the excavation, the surrounding soil may swell due to absorbed water from the concrete and earth pressures will increase. It is believed that, if enough time is allowed between installation and loading, the K_o conditions will eventually be re-established. Finally, the rate of loading the pile will dictate if earth pressures will change depending on drained or undrained conditions. This earth pressure prediction is probably the most complex and has the least available data of all the input soil parameters. Therefore to simplify the calculations, K_o conditions will be assumed which should yield adequate results.

The effective angle of shearing resistance(ϕ') occurs along a plane between the major shear distorted zone around the pile shaft and the intact soil. A drained shear test with the appropriate confining pressure to represent the horizontal stresses acting along the pile should be used to determine ϕ'. Values of ϕ' for the local soil conditions are presented in Table A-1, Appendix A.

2.4 Settlement

The performance of a pile foundation is evaluated on the basis of load versus settlement criteria. The majority of the settlement that a pile foundation undergoes is believed to be immediate or elastic and the long term settlement is negligible (Burland and Cooke,1974; and Thomson,1980). The settlement of a pile is governed by the soil parameters and the pile geometry. The type of pile and quality of workmanship with which it is installed affect the performance of a pile.

The load/settlement relationship for the pile shaft is approximately linear up to the full mobilization of the shaft resistance which occurs at 0.5 to 2.0 percent of the shaft diameter. Poulos and Davis(1968) have developed an equation for predicting the settlement(ρ) corresponding to an applied load(P).

$$\rho = (P/LE)Ip$$

where:

L is the length of the pile shaft

E is the Young's modulus for the soil

Ip is an influence factor for an incompressible cylindrical pile in an elastic medium with a rigid stratum at depth.

When the depth of the rigid stratum is large compared with the length of the pile, the range of the influence factor is small and usually averages 1.8. Thus the equation becomes:

$$\rho = (P/LE)1.8$$

This equation should give an acceptable prediction of the settlement of a single friction pile. The major complication in this approach is in determining the Young's modulus which may vary with radial distance from the pile (due to boring) as well as with depth. Since neither laboratory testing of representable samples nor in-situ testing methods account for soil disturbance caused by boring, pile load tests remain as the best method of predicting the settlement of friction piles.

The friction pile does not carry the applied load entirely by shaft resistance, with a portion of the load being transferred to the base of the pile. Therefore, the load/settlement characteristics of the base should also be incorporated into a relationship in order to predict the total settlement with more accuracy. Burland, Butler and Duncan(1966) and Burland and Cooke(1974) studied the behaviour of the pile base by using plate load tests. A linear load/settlement relationship was found provided the base pressure does not exceed one third of the ultimate base pressure. The relationship is:

$\rho = KD(Q/Q_{ult})$

where:

D is the diameter of the pile base

Q is the applied base load

Q_{ult} is the ultimate base load

This relationship was used by Thomson(1980) in evaluating data from pile load tests in the Edmonton area where it was found that the K values under working load conditions generally ranged from 0.005 to 0.02 for the cases studied.

An evaluation of the settlement data has been carried out using this relationship for the pile load tests in this study. The K values were not determined under working load conditions, since skin friction piles at working load carry the majority of the load by shaft resistance and only a negligible amount by end bearing. Instead, the K values have been determined for a settlement at a total load corresponding to the shaft resistance being fully mobilized and the base load being one third of the ultimate base load. The K values were fairly consistent with a range from 0.012 to 0.049 for all the pile load tests with the exception of the University of Alberta Pile No. 3. This pile revealed a K value of 0.136, however, this pile has been notorious for yielding inconsistent results throughout this study. The higher than normally expected K values for piles in local soil conditions is possibly due to the bases not being properly cleaned since these piles carry the majority of the load by shaft resistance. The settlement and related K values for the respective piles are presented in Table A-4, Appendix A.

Chapter 3

Geology and Geotechnical Soil Parameters

The general geology of the Edmonton area has been described by May and Thomson(1978). For geotechnical studies only the near-surface geology consisting of the Horseshoe Canyon Formation, laid down near the end of the Cretaceous, and later deposits will be of interest. Subsequent to the Cretaceous, the Alberta plains were subjected to sub-aerial erosion. Portions of pre-glacial channels were filled with Saskatchewan Sands and Gravels which can be up to 20 metres thick in the Edmonton area. The continental glacier then advanced over the area during the late Pleistocene and two till sheets were laid down. The lower sheet was deposited by a glacier moving from slightly west of north, while the upper sheet was deposited from a glacier moving from east to north. These till sheets are geotechnically similar and consist of a conglomeration of clay, silt and sand with some gravel. These till sheets will be considered as one stratigraphic unit for the purpose of this study. Upon rapid recession of the glaciers, proglacial lakes were formed in lower areas and one such lake covered the area where the city of Edmonton is located. Glacial lake sediments were deposited during this lake environment until the lake was eventually drained by the North Saskatchewan River. These sediments consist of clays in the upper portion of the deposit which gradually become silty and sandy in the lower portion.

Geotechnical soil parameters for the different soil types in the Edmonton area have been reported by Eisenstein(1982) and appear in Table A-1, Appendix A. These parameters have been determined from both laboratory and in-situ testing, performed for various projects by the University of Alberta and geotechnical firms in the city.

Chapter 4

Test Pile Analysis

4.1 General

A total of eleven pile load tests from four different sites in the Edmonton area will be analysed. The majority of the pile load tests were carried out for commercial purposes to confirm the load carrying capacity of a particular pile type. No attempt was made to separate the shaft carrying capacity from the end bearing capacity during the tests. However, an estimation of the amount of the load carried by the shaft and by the base has been made for this study.

In addition to these commercial pile load tests, four pile load tests were performed for research at the University of Alberta Farm by Bhanot(1968). These piles were equipped with load cells near their bases so that the amount of load carried by the shaft could be separated from that carried by the base during the loading stage.

All the piles were tested by a hydraulic jack reacting against a beam supported by anchor piles. The vertical movement of the top of the piles was monitored by dial gauges. The testing procedure for most of the pile load tests consisted of a cyclic loading which allowed an increase in load increment when the settlement rate fell below 0.25 mm/hr or 2 hours had elapsed. The loading was increased to approximately 200 percent of the assumed design load which caused sufficient settlement to mobilize the full shaft friction.

The pile load test results have been back analysed by both the total and effective stress methods in order that a comparison of calculated and average reported α and β values can be made.

With the total stress method the shaft carrying capacity is estimated by:

$R = (\pi DL)Ca$

where $Ca = \alpha Cu$

Given the pile geometry and the shaft carrying capacity the average shaft adhesion can be determined. The empirical coefficient(α) can then be calculated as the ratio of the average shaft adhesion over the average undrained shear strength. The α values along with other pertinent information for the total stress method are presented in Table A-2, Appendix A.

For the effective stress method, the shaft carrying capacity is estimated by:

$R = (\pi DL)P'\beta$

where $\beta = K_o \tan\phi'$

As before, given the pile geometry, shaft carrying capacity and average vertical effective overburden pressure along the pile shaft, the average β value can be calculated. The calculated β value is then compared to the average reported β value determined from the test site soil properties K_o and ϕ'. The details and results of this analysis are presented in Table A-3, Appendix A.

4.2 University of Alberta Test Piles

These pile load tests were performed at this site by Bhanot(1968), to study the behaviour of cast-in-place concrete piles. The site was chosen because of its soil stratigraphy being representative of the conditions in the Edmonton area. The stratigraphy at the site consists of glacial lake sediments from surface to approximately 8.0 metres, underlain by glacial till. The lake sediments were predominantly clay near the surface grading into silt near the bottom of the deposit. The till was typical of the Edmonton area, but exhibited a higher than average undrained shear strength.

A total of four piles were installed at the test site. Two of the piles were cast entirely in the overlying lake sediments, while the other two extended down into the till and were unsupported on the sides through the lake sediments. Diagrams showing the soil conditions and pile installation details are shown in Appendix A. As stated earlier, load cells were installed near the base of these piles to record the load being carried by end bearing. The shaft carrying capacity was then determined by subtracting the end bearing load from the total load.

For the total stress method the calculated α values for the clay were fairly consistent and only slightly greater than the local average reported value of 0.45. On the other hand, the calculated α values for the till were more variable with the values being 18 and 90 percent greater than the same local average reported value of 0.45.

With the effective stress method the calculated β values for the clay were reasonably consistent and were 15 and 28 percent greater than the average reported values. The calculated β values for the till, like the α values for the till were also variable. One calculated β value was only 11 percent greater than the average reported value, while the other calculated value was approximately 100 percent greater.

This significant variation in the α and β values for the till is believed to be due to a very hard strata in the upper till region that was not identified during the soil testing program. With the exception of Pile No.3 which was greatly affected by this hard strata, the calculated values from both the total and effective stress methods compared reasonably well with the average reported values. Since the average reported values were all less than the calculated values, an estimation of the shaft capacity based on the average reported values would be conservative.

4.3 Woodbend Apartments Test Piles

These pile load tests were carried out in May, 1977 by Canada Caisson Co. Ltd. and R.M. Hardy and Associates Ltd. to evaluate the performance of the piles for tendering purposes. Evaluation of these piles consisted of monitoring the settlement of the top of the pile during the loading stage.

The foundation investigation report for this site was unavailable, however some limited information was obtained during drilling of the test piles. The soil conditions consisted of glacial lake clay from surface to approximately 5.3 metres, underlain by glacial till. The till was predominantly clay with silt, sand and gravel throughout. A sand lense approximately 0.75 metres thick was noted at a depth of about 9.0 metres in both the pile excavations.

Two piles were installed at this test site, to depths of 12.52 and 12.65 metres. The piles extended through the glacial lake clay and into the glacial till. Diagrams showing the soil conditions and pile installation details are presented in Appendix A. Since these piles were installed in both the clay and till only the average α and β values could be calculated for the entire length of the piles

For the total stress method the average calculated α values for the two tests were fairly consistent, but were 40 and 50 percent greater than the local average reported value of 0.45.

The effective stress method yielded average calculated β values that were similar and slightly less conservative than the α values, by only being 27 and 34 percent greater than the average reported values. Once again the average reported values are less than the calculated values for both the total and effective stress methods and an estimation of the shaft carrying capacity would be conservative. However, if estimations of the shaft carrying capacities are based on average reported values that are highly conservative the design may prove to be uneconomical.

4.4 West Edmonton Mall Test Piles

The pile load tests performed at this site were also done by Canada Caissons Co. Ltd. and Hardy Associates(1978) Ltd., in June and July, 1980. These tests were carried out to establish an allowable shaft adhesion for the purpose of a total stress estimation of the shaft carrying capacity.

Prior to these pile load tests, four geotechnical investigations were conducted over the entire area of the site. The soil conditions at the site consist of glacial lake sediments from surface to approximately 8.0 metres, underlain by glacial till and eventually bedrock at about 11.5 metres. The Saskatchewan Sands and Gravels which usually exist between the till and bedrock are absent from this site. The glacial lake sediments can be separated into an upper portion which extends down to 4.5 metres, and consists of stiff, high plasticity, silty clay. In the lower portion the clay is very silty, sandy and firm. The glacial till exhibited a lower consistency then what is expected of the till in the Edmonton area. The bedrock to the depth drilled consisted of fine to medium grained, dense sandstone.

Four piles were installed, however, an anchor pile pulled out during the testing of Pile No. 2 so the results have been disregarded. The other three piles were installed through the clay and till and approximately one metre into the sandstone. The soil conditions, pile installation details and calculations are presented in Appendix A. Since these piles were installed in the clay, till, and sandstone only the average α and β values could be calculated for the entire length of the pile.

With the total stress method the average calculated α values were fairly consistent, and ranged from 7 to 18 percent less than the local average reported value of 0.45.

For the effective stress method the average calculated β values were also fairly consistent and ranged from 32 to 47 percent less than the average reported values. The calculated values are less than the average reported values for both the total and effective stress methods. It therefore becomes apparent that the soil conditions at this site are weaker than typically found in the Edmonton area.

4.5 Santa Rosa Underpass Test Tie-backs

These tie-back load tests were performed in December, 1976 and January, 1977 by Western Caissons Ltd. and Bernard and Hoggan Engineering Ltd. to evaluate the tie-back anchor capabilities for support of a sheet pile wall. Two tie-backs were installed at an inclination of 30 degrees to the horizontal at an upper and lower level. Tie-back No. 1 was installed at the upper level and protruded out of the sheet pile wall 0.9 metres below the ground surface and was cast entirely in the glacial lake sediments. Tie-back No. 2, installed at the lower level, protruded out of the sheet pile wall 3.9 metres below the ground surface and extended through the glacial lake sediments and into the till deposit. The glacial lake sediments consisted of low to high plasticity stiff clay typical of the Edmonton area. The till is predominantly clay with sand lenses throughout. Diagrams showing the soil conditions and pile installation details are presented in Appendix A.

The analyses performed on the tie-back tests produced calculated α and β values for the clay from the Tie-back No. 1 results and average calculated α and β values for the clay and till from the Tie-back No. 2 results.

For the total stress method the calculated α values were consistent and were 25 and 32 percent less than the local reported average of 0.45.

The effective stress method yielded calculated β values that were reasonably consistent, but the calculated β value for the clay was 12 percent greater than the average reported value, where as for the clay and till the average calculated β value was 10 percent less.

With the calculated α values being approximately 30 percent less than the local average reported value, an estimation of the shaft carrying capacity based on this local reported average would lead to an unsafe design. These low calculated α values are believed to be due to low lateral stresses on the piles since they are inclined, and therefore only a small portion of the undrained shear strength is developed as shaft adhesion.

Chapter 5

Discussion

A total of eleven pile load tests from four different sites in the Edmonton area have been included in this study. The results of pile load tests have been analysed to check the validity of the total and effective stress methods of pile design. In addition, the settlement data from these pile load tests has also been analysed.

The majority of the pile load tests were carried out for commercial purposes and only limited information for research was available. Some discrepancies may occur between the estimated and actual pile carrying capacities due to the quality of the workmanship and adverse ground conditions that may harm the pile performance. In addition, estimations are based on the nominal shaft diameter and excavation usually increases this diameter, thus increasing the pile carrying capacity by 10 percent or more. Apart from the installation problems, the method of testing can also influence the results. Nonetheless, the information obtained from these pile load tests is believed to be sufficient for the analyses performed for this study.

The back analyses has revealed that the total stress method is a very useful approach for the design of piles, mainly because it takes into account valuable practicle experience gained in the specific area. This experience is reflected in the empirical coefficient(α) term which is believed to be appoximately 0.45 for the clay and till in the Edmonton area. The empirical coefficients back calculated from the pile load tests were somewhat scattered with the values ranging from approximately 30 percent less than to 50 percent greater than the local average reported value. An estimation of the shaft carrying capacity based on the local average reported value would be conservative in some cases, and lead to an unsafe design in other cases. The low calculated α values from the West Edmonton Mall test piles are believed to have occured due to the soil in that area being weaker than typically encountered in the Edmonton area. The low calculated α values from the Santa Rosa Underpass test tie-backs are believed to be due to low lateral stresses on the tie-backs since they were inclined at 30 degrees to the horizontal. This is the same problem that occurs in shallow piles where the lateral stresses are low and only a small portion of the undrained shear strength is developed as shaft adhesion. Design of inclined and shallow piles or standard piles in new or unusual ground conditions by the total stress method may be dangerous unless good engineering judgement is exercised.

A back analyses of the pile load test results based on the effective stress method compared fairly well with average reported results. The calculated β values ranged from approximately 50 percent less than to 40 percent greater than the average reported values. The calculated β values were less than the average reported values for the pile load tests performed at the West Edmonton Mall. This is believed to be due to softer clay and till at this site then usually encountered, and the average reported β values were determined from typical K_o and ϕ' values of the Edmonton area.

The effective stress method seems to adjust to different and unusual circumstances and can be adopted to almost any situation. Although, a number of simplifying assumptions are required, the method appears to account for the interaction between the pile and surrounding soil. Probably the main difficulty with this method lies in predicting the coefficient of earth pressure at rest(K_o) at various depths. However, new methods, like the in-situ self boring pressure meter and laboratory tests are now available for measuring K_o values.

Evaluation of the settlement data revealed that both the shaft and base load/settlement relationships must be considered when predicting settlement. The load/settlement relationship for the shaft increases linearly up to full mobilization at a small amount of settlement and then remains constant for any settlement thereafter. Whereas the load/settlement relationship for the base remains as a continuous function for a greater amount of settlement. Hence, at design capacity loads the shaft resistance will most likely be fully mobilized and only the base resistance will have a linear relationship with settlement. Therefore the K values in this study have been determined for settlement at a total load corresponding to the shaft resistance being fully mobilized and the base load being one third of the ultimate base load. These K values are believed to be an upper limit for piles at working loads in the Edmonton area.

Chapter 6

Recommendations

The effective stress method outlined in this paper is not intended to replace the conventional total stress method of estimating the shaft carrying capacity of a pile. However, it should be advantageous to use the total stress method backed up by a preliminary design based on effective stress. This procedure should also be utilized in new or unusual ground conditions, where the effective stress method can help in the selection of appropriate total stress α values since it accounts for the stress conditions around the pile. The effective stress method should not be used as a basis for final design recommendations since the pile/ground interaction is still too simplified. More research must be performed on the distribution of both the normal and shear stresses along the pile shaft. It is believed that the primary value of this method at the present time is to provide a simple theoretical model for a better understanding of the fundamental principles governing pile behaviour.

In order to evaluate the effective stress method in practice, site investigation work must include in-situ and laboratory testing to determine the effective stress properties. The installation of piezometers should become a normal practice in order to have a better idea of the pore pressures in the ground.

For a preliminary estimation of settlement of friction piles in the Edmonton area the load/settlement relationships for the pile shaft and base must be considered. Hence, settlements and related K values may be estimated for a total load corresponding to full shaft capacity and the base load being one third of the ultimate base load. The settlements and K values estimated by this method will most likely be upper limits since the base load is usually less than one third of the ultimate base load.

References

Burland,J.B., Shaft friction of piles in clay - a simple fundamental approach, Building Research Establishment Current Paper, Nov. 1973.

Burland,J.B., and Cooke,R.W., The design of bored piles in stiff clays, Building Research Establishment Current Paper, Dec. 1974.

Chandler,R.J., The shaft friction of piles in cohesive soils in terms of effective stress, Civil Engineering and Public Works Review, pp48-51, Jan. 1968.

Chandler,R.J., and Martin,J.P.,An experimental study of skin friction around piles in clay, Geotechnique 32, No.2, pp119-132, Sept. 1982.

Cooke,R.W., The settlement of friction pile fondations, Building Research Establishment Current Paper, Jan. 1975.

Cooke,R.W., Piled foundations: a survey of research at the Building Research Station, Building Research Establishment Current Paper, Feb. 1974.

Cooke,R.W. and Price,G., Strains and displacements around friction piles, Building Research Establishment Current Paper, Oct. 1973.

Meyerhof,G.G., Bearing capacity and settlement of pile foundations, A.S.C.E. Journal of Geotechnical Engineering, Volume 102, pp197-228, Aug. 1976.

O'Riordan,N.J., The mobilization of shaft adhesion down a bored, cast-in-situ pile in the Woolwich and Reading beds, Ground Engineering, Nov. 1979.

Parry,H.G., and Swain,C.W., A study of skin friction on piles in stiff clay, Ground Engineering, Volume 10, No.3, pp33-37, Nov. 1977.

Searle,I.W., The design of bored piles in overconsolidated clays using effective stresses, Recent Developments in the Design and Construction of Piles, I.C.E., London, 1979.

Thomson,S., Evaluation of pile load tests in the Edmonton area, Canadian Geotechnical Journal, Volume 18, pp313-316, Nov. 1980.

Vijayvergiya,V.N. and Focht,J.A., A new way to predict capacity of piles in clay, Offshore Technology Conference, OTC 1718, May 1972.

Wright,S.J. and Reese,L.C., Design of large diameter bored piles, Ground Engineering, Volume 12, No.8, pp19-23, 50-51, Nov. 1979.

Appendix A

TABLE A-1

GEOTECHNICAL SOIL PARAMETERS

Soil Type	Bulk Density KN/m^3	Coef of Earth Pressure at Rest (K_o	Effect. Angle of Shearing Resistance	Effective Cohesion (KPa)	Undrained Shear Strength (KPa)
Lake Edmonton Clay and Silt	19	1.0 - 4.0 (2.0)	18 - 23.5 (20)	0 - 10 (0)	45 - 220 (100)
Glacial Till	21	0.5 - 1.2 (0.85)	37 - 59 (40)	0.28 (0)	140 - 828 (200)
Saskatchewan Sands and Gravel	21	0.25 - 0.40 (0.3)	43 - 53 (43)	0	-
Bedrock	22	0.8 - 1.2 (1.0)	14 - 24 (19)	52 - 58 (55)	-

TABLE A-2

TOTAL STRESS RESULTS

Project and Pile Number	Shaft Diameter (m)	Shaft Length (m)	Ultimate Shaft Capacity (KN)	Average Shaft Adhesion (KPa)	Average Undrained Shear Strgth (KPa)	Empirical Coefficient a	Soil Type
University of Alta.							
Pile No. 1	0.44	4.50	289	46	89	0.517	Clay
Pile No. 2	0.56	3.31	249	43	92	0.467	Clay
Pile No. 3	0.56	2.96	1400	269	314	0.856	Till
Pile No. 4	0.75	6.12	2305	160	300	0.533	Till
Woodbend Apartments							
Pile No. 1	0.61	12.52	2488	104	166	0.627	Clay/Till
Pile No. 2	0.76	12.65	3280	109	163	0.669	Clay/Till
West Edmonton Mall							
Pile No. 1	0.51	12.95	875	42	100	0.420	Cly/Till/SST
Pile No. 2	0.61	13.11	Anchor pile pulled out during the testing of the pile.				
Pile No. 3	0.76	12.57	1128	38	103	0.369	Cly/Till/SST
Pile No. 4	0.51	12.95	799	39	103	0.379	Cly/Till/SST
Santa Rosa							
Tie-back No. 1	0.41	6.40	276	33	97	0.340	Clay
Tie-back No. 2	0.30	9.14	445	52	145	0.359	Clay/Till

The Santa Rosa test tie-backs were installed at an inclination of 30 degrees to the horizontal.

TABLE A-3

EFFECTIVE STRESS RESULTS

Project and Pile Number	Shaft Diameter (m)	Shaft Length (m)	Ultimate Shaft Capacity (KN)	Calculated β Value	Avg. Reported β Value	Soil Type
University of Alta.						
Pile No. 1	0.44	4.50	289	0.817	0.728	Clay
Pile No. 2	0.56	3.31	249	0.904	0.728	Clay
Pile No. 3	0.56	2.96	1400	1.750	0.839	Till
Pile No. 4	0.75	6.12	2305	0.916	0.839	Till
Woodbend Apartments						
Pile No. 1	0.61	12.52	2488	1.04	0.817	Clay/Till
Pile No. 2	0.76	12.65	3280	1.09	0.814	Clay/Till
West Edm. Mall						
Pile No. 1	0.51	12.95	875	0.423	0.690	Clay/Till/SST
Pile No. 2	0.61	13.11	Anchor pile pulled out during the testing of the pile			
Pile No. 3	0.76	12.57	1128	0.382	0.726	Clay/Till/SST
Pile No. 4	0.51	12.95	799	0.386	0.701	Clay/Till/SST
Santa Rosa						
Tie-back No. 1	0.41	6.40	276	0.815	0.728	Clay
Tie-back No. 2	0.30	9.14	445	0.712	0.789	Clay/Till

TABLE A-4

SETTLEMENT

Project and Pile Number	Settlement at Qu/Qult = 1/3 (mm)	Calculated K Values	Soil Type at Base
University of Alberta			
Pile No. 1	1.8	0.012	Clay
Pile No. 2	2.3	0.012	Clay
Pile No. 3	25.4	0.136	Till
Pile No. 4	12.2	0.049	Till
Woodbend Apartments			
Pile No. 1	9.9	0.049	Till
Pile No. 2	12.2	0.048	Till
West Edmonton Mall			
Pile No. 1	8.0	0.047	Sandstone
Pile No. 3	8.5	0.034	Sandstone
Pile No. 4	7.1	0.042	Sandstone
Santa Rosa Underpass settlement results unavailable for the test tie - backs			

NOTE: The above settlement results and related K values have been determined at a total load corresponding to the full shaft capacity and the base load being one third of the ultimate base load.

University of Alberta Test Piles

Information from Bhanot(1968) Ph.D. Thesis

Average Reported Soil Properties

γ(Clay) = 19 KN/m³

γ(Till) = 22 KN/m³

β(Clay) = 0.728

β(Till) = 0.839

Cu(Clay) = 89 KPa, Pile#1

Cu(Clay) = 92 KPa, Pile#2

Cu(Till) = 314 KPa, Pile#3

Cu(Till) = 300 KPa, Pile#4

Soil Profile

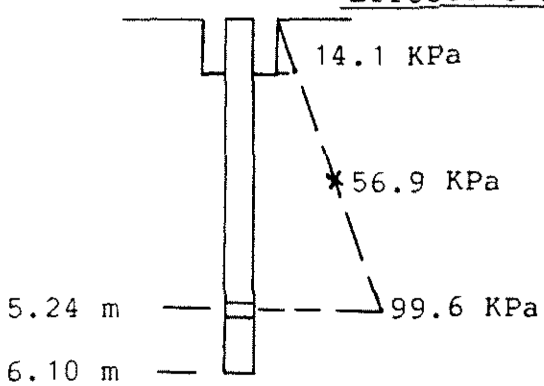
Pile No. 1

Shaft Diameter = 0.44 m

Shaft Length = 4.50 m

Ultimate Shaft Capacity = 289 KN

Effective Stress Method



Circumference = 1.382 m

$4.50(56.9)(1.382)\beta = 289$ KN

$\beta = 0.817$ (calculated)

$\beta = 0.728$ (reported average)

Total Stress Method

Surface area = 6.22 m^2

(6.22)Ca = 289 KPa

Ca = 46 KPa (calculated)

Cu = 89 KPa (reported average)

a = Ca/Cu = 0.517

Pile No. 2

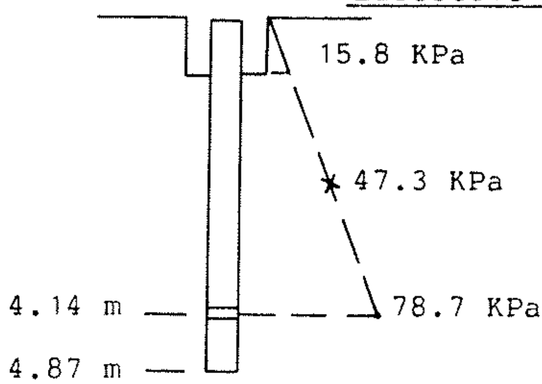
Shaft Diameter = 0.56 m

Shaft Length = 3.31 m

Ultimate Shaft Capacity = 249 KN

Effective Stress Method

Circumference = 1.759 m



$3.31(47.3)(1.759)\beta = 249$ KN

$\beta = 0.904$ (calculated)

$\beta = 0.728$ (reported average)

Total Stress Method

Surface area = 5.82 m^2

(5.82)Ca = 249 KPa

Ca = 43 KPa (calculated)

Cu = 92 KPa (reported average)

a = Ca/Cu = 0.467

Pile No. 3

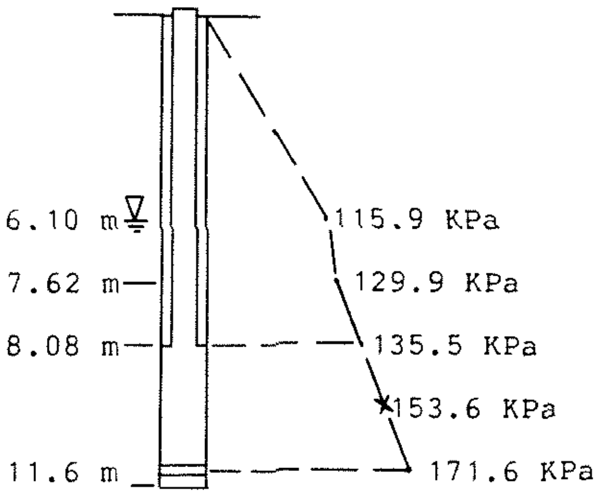
Shaft Diameter = 0.56 m

Shaft Length = 2.96 m

Ultimate Shaft Capacity = 1400 KN

Effective Stress Method

Circumference = 1.759 m



$2.96(153.6)(1.759)\beta = 1400$ KN

$\beta = 1.750$ (calculated)

$\beta = 0.839$ (reported average)

Total Stress Method

Surface area = 5.21 m^2

(5.21)Ca = 2305 KPa

Ca = 269 KPa (calculated)

Cu = 314 KPa (reported average)

a = Ca/Cu = 0.856

<u>Pile No. 4</u>

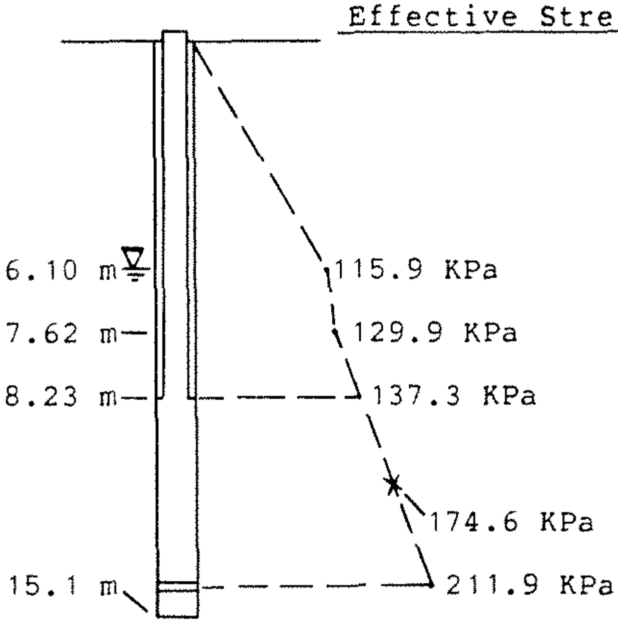
Shaft Diameter = 0.75 m

Shaft Length = 6.12 m

Ultimate Shaft Capacity = 2305 KN

<u>Effective Stress Method</u>

Circumference = 2.356 m



$6.12(174.6)(2.356)\beta = 2305$ KN

$\beta = 0.916$ (calculated)

$\beta = 0.839$ (reported average)

<u>Total Stress Method</u>

Surface area = 14.42 m^2

(14.42)Ca = 2305 KPa

Ca = 160 KPa (calculated)

Cu = 300 KPa (reported average)

α = Ca/Cu = 0.533

Settlement

Pile No.1 Shaft Diameter = 0.44 m

Shaft Length = 4.50 m above load cell

Shaft Length = 0.86 m below load cell

$Q_{ult} = (0.44^2\pi/4)(89)9 = 122$ KN

If $Q/Q_{ult} = 0.33$, then $Q = 41$ KN

$R = 289 + 0.44\pi(0.86)(89)0.517 = 344$ KN

$Q_t = R+Q = 344 + 41 = 385$ KN

Corresponding settlement(ρ) = 1.8 mm

$K = (\rho/D)(Q/Q_{ult})$

$K = (1.8/440)3 = 0.012$

Pile No.2 Shaft Diameter = 0.56 m

Shaft Length = 3.31 m above load cell

Shaft Length = 0.75 m below load cell

$Q_{ult} = (0.56^2\pi/4)(92)9 = 204$ KN

If $Q/Q_{ult} = 0.33$, then $Q = 68$ KN

$R = 249 + 0.56\pi(0.75)(92)0.467 = 306$ KN

$Q_t = R+Q = 306 + 68 = 374$ KN

Corresponding settlement(ρ) = 2.3 mm

$K = (2.3/560)3 = 0.012$

Settlement(continued)

Pile No.3 Shaft Diameter = 0.56 m

Shaft Length = 2.96 m above load cell

Shaft Length = 0.54 m below load cell

$Q_{ult} = (0.56^2\pi/4)(314)9 = 696$ KN

If $Q/Q_{ult} = 0.33$, then $Q = 232$ KN

$R = 1400 + 0.56\pi(0.54)(314)0.856 = 1655$ KN

$Q_t = R+Q = 1655 + 232 = 1887$ KN

Corresponding settlement(ρ) = 25.4 mm

$K = (25.4/560)3 = 0.136$

Pile No.4 Shaft Diameter = 0.75 m

Shaft Length = 6.12 m above load cell

Shaft Length = 0.73 m below load cell

$Q_{ult} =(0.75^2\pi/4)(300)9 = 1193$ KN

If $Q/Q_{ult} = 0.33$, then $Q = 398$ KN

$R = 2305 + 0.75\pi(0.73)(300)0.533 = 2580$ KN

$Q_t = R+Q = 2580 + 398 = 2978$ KN

Corresponding settlement(ρ) = 12.2 mm

$K = (12.2/750)3 = 0.049$

Woodbend Apartments Test Piles

Information from Pile Load Test Report

Average Reported Soil Properties

γ(Till) = 21 KN/m³

γ(Clay) = 19 KN/m³

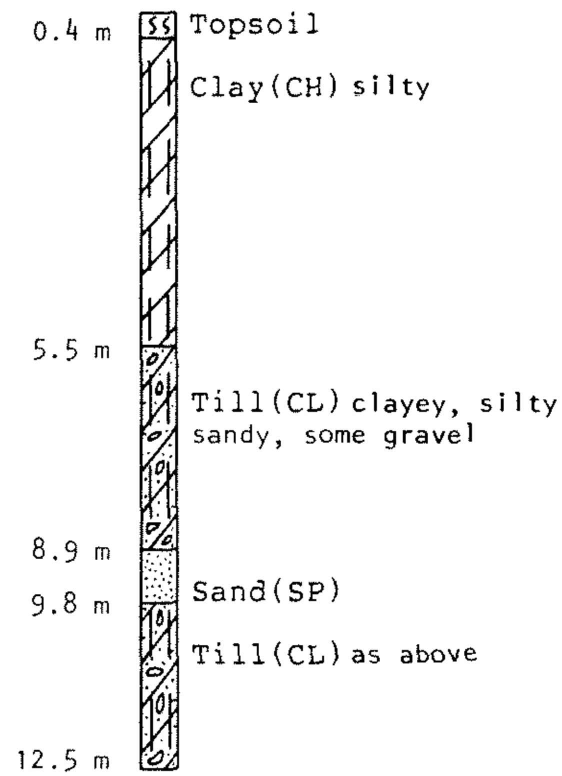
β(Till) = 0.839

β(Clay) = 0.728

Cu(Till) = 212 KPa

Cu(Clay) = 100 KPa

Soil Profile



Pile No. 1

Pile Diameter = 0.61 m

Pile Length = 12.52 m

Ultimate Pile Capacity = 2669 KN

Maximum Settlement = 9.9 mm

Percent settlement/pile dia. = 9.9/610 = 1.62%

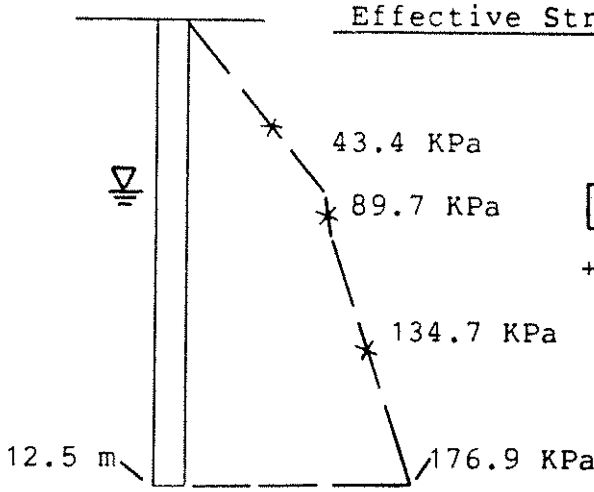
Amount of end bearing mobilized = 1.62/5.0 = 32.4%

Ultimate end bearing = $(0.61^2\pi/4)(212)9$ = 558 KN

Mobilized end bearing = 558(0.324) = 181 KN

Ultimate Shaft Capacity = 2669 - 181 = 2488 KN

Effective Stress Method



Circumference = 1.916 m

$$\left[4.57(43.4) + 0.61(89.7) + 7.35(134.7)\right]1.916\beta = 2488 \text{ KN}$$

β = 1.04 (calculated)

β = 0.817 (reported avg.)

Total Stress Method

Surface area in clay = 9.93 m^2

Surface area in till = 14.07 m^2

$$(9.93 + 14.07)Ca = 2488 \text{ KN}$$

Ca = 104 KPa (calculated)

Cu = 166 KPa (reported average)

α = Ca/Cu = 0.627

Pile No. 2

Pile Diameter = 0.76 m

Pile Length = 12.65 m

Ultimate Pile Capacity = 3559 KN

Maximum Settlement = 12.2 mm

Percent settlement/pile dia. = 12.2/760 = 1.61%

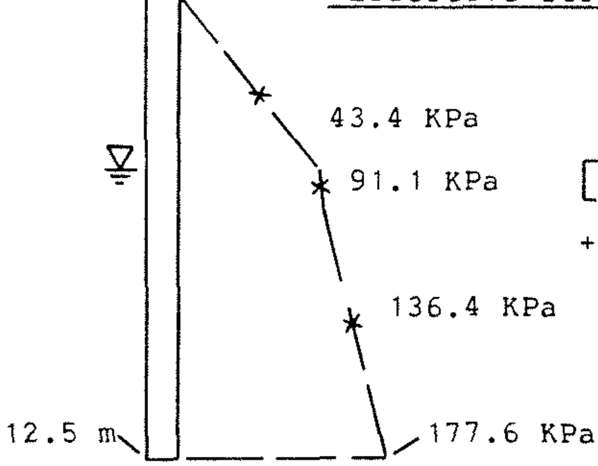
Amount of end bearing mobilized = 1.61/5.0 = 32.2%

Ultimate end bearing = $(0.76^2\pi/4)(212)9 = 866$ KN

Mobilized end bearing = 866(0.322) = 279 KN

Ultimate Shaft Capacity = 3559 - 279 = 3280 KN

Effective Stress Method



Circumference = 2.388 m

$$\left[4.57(43.4) + 0.914(91.1) + 7.16(136.4)\right]2.388\beta = 3280 \text{ KN}$$

$\beta = 1.09$ (calculated)

$\beta = 0.814$ (reported avg.)

Total Stress Method

Surface area in clay = 13.10 m^2

Surface area in till = 17.10 m^2

$(13.10 + 17.10)Ca = 3280$ KN

$Ca = 109$ KPa (calculated)

$Cu = 163$ KPa (reported average)

$a = Ca/Cu = 0.669$

Settlement

Pile No.1 Shaft Diameter = 0.61 m

Shaft Length = 12.52 m

Ultimate End Bearing(Q_{ult}) = 558 KN (as shown on A-9)

If Q/Q_{ult} = 0.33, then Q = 186 KN

Ultimate Shaft Capacity(R) = 2488 KN (as shown on A-9)

Q_t = R+Q = 2488 + 186 = 2674 KN

Corresponding settlement(ρ) = 9.9 mm

K = (9.9/610)3 = 0.049

Pile No.2 Shaft Diameter = 0.76 m

Shaft Length = 12.65 m

Ultimate End Bearing(Q_{ult}) = 866 KN (as shown on A-10)

If Q/Q_{ult} = 0.33, then Q = 289 KN

Ultimate Shaft Capacity(R) = 3280 KN (as shown on A-10)

Q_t = R+Q = 3280 + 289 = 3569 KN

Corresponding settlement(ρ) = 12.2 mm

K = (12.2/760)3 = 0.048

West Edmonton Mall Test Piles

Information from Pile Load Test and Foundations Reports

Average Reported Soil Properties

γ(Clay) = 19 KN/m^3

γ(Till) = 21 KN/m^3

γ(Sandstone) = 21 KN/m^3

β(Clay) = 0.728

β(Till) = 0.839

β(Sandstone) = 0.344

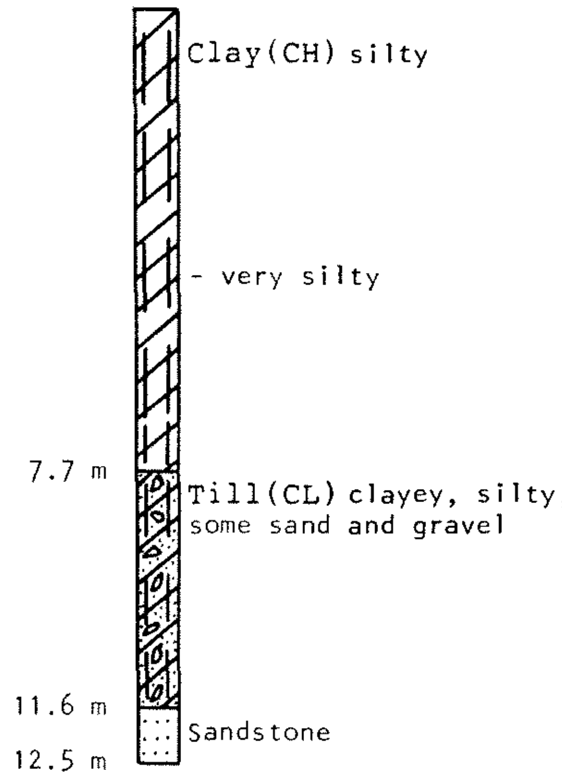
Cu(Clay,0-4.5m) = 83 KPa

Cu(Clay,4.5m-8.0m) = 32 KPa

Cu(Till) = 120 KPa

Cu(Sandstone) = 350 KPa

Soil Profile



Pile No. 1

Pile Diameter = 0.51 m

Pile Length = 13.0 m

Ultimate Pile Capacity = 1334 KN

Maximum Settlement = 18.2 mm

Percent settlement/pile dia. = 18.2/510 = 3.57%

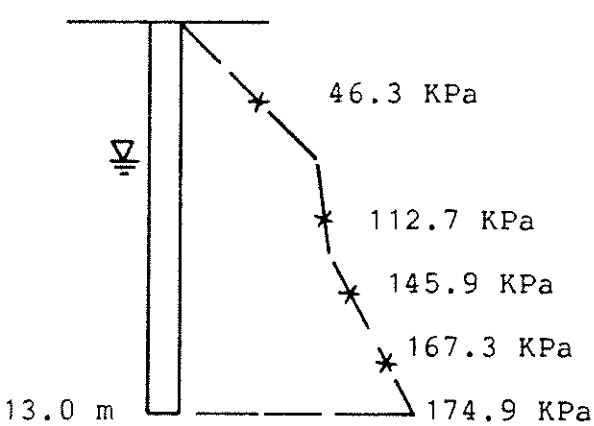
Amount of end bearing mobilized = 3.57/5.0 = 71.3%

Ultimate end bearing = $(0.51^2\pi/4)(350)9 = 643$ KN

Mobilized end bearing = 643(0.713) = 459 KN

Ultimate Shaft Capacity = 1334 - 459 = 875 KN

Effective Stress Method



Circumference = 1.602 m

$$[4.88(46.3) + 4.27(112.7) + 2.44(145.9) + 1.37(167.3)]\,1.602\beta = 875 \text{ KN}$$

β = 0.423 (calculated)

β = 0.690 (reported avg.)

Total Stress Method

Surface area in clay = 14.65 m^2

Sfc. area in till = 3.91 m^2, Sfc. area in SST = 2.20m^2

$(14.65 + 3.91 + 2.20)Ca = 875$ KN

Ca = 42 KPa (calculated)

Cu = 100 KPa (reported average)

α = Ca/Cu = 0.420

Pile No. 3

Pile Diameter = 0.76 m

Pile Length = 12.57 m

Ultimate Pile Capacity = 1835 KN

Maximum Settlement = 18.8 mm

Percent settlement/pile dia. = 18.8/760 = 2.47%

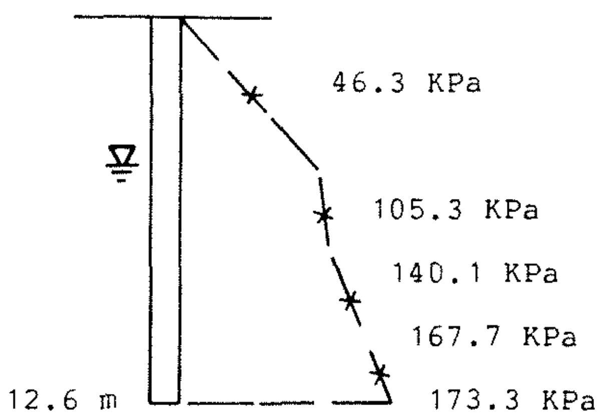
Amount of end bearing mobilized = 2.47/5.0 = 49.5%

Ultimate end bearing = $(0.76^2\pi/4)(350)9$ = 1429 KN

Mobilized end bearing = 1429(0.495) = 707 KN

Ultimate Shaft Capacity = 1835 - 707 = 1128 KN

Effective Stress Method



Circumference = 2.388 m

$$\left[4.88(46.3) + 2.74(105.3) + 3.96(140.1) + 0.99(167.7)\right]$$

$$2.388\beta = 1128 \text{ KN}$$

β = 0.382 (calculated)

β = 0.726 (reported avg.)

Total Stress Method

Surface area in clay = 18.20 m^2

Sfc. area in till = 9.46 m^2, Sfc. area in SST = 2.37 m^2

$(18.20 + 9.46 + 2.37)Ca = 1128$ KN

Ca = 38 KPa (calculated)

Cu = 103 KPa (reported average)

$a = Ca/Cu = 0.369$

Pile No. 4

Pile Diameter = 0.51 m

Pile Length = 12.95 m

Ultimate Pile Capacity = 1334 KN

Maximum Settlement = 21.2 mm

Percent settlement/pile dia. = 21.2/510 = 4.16%

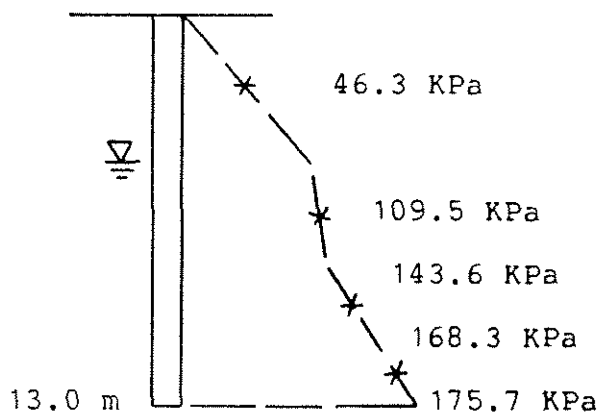
Amount of end bearing mobilized = 4.16/5.0 = 83.2%

Ultimate end bearing = $(0.51^2\pi/4)(350)9$ = 643 KN

Mobilized end bearing = 643(0.832) = 535 KN

Ultimate Shaft Capacity = 1334 - 535 = 799 KN

Effective Stress Method



Circumference = 1.602 m

$$\left[4.88(46.3) + 3.65(109.5) + 3.11(143.6) + 1.31(168.3)\right] 1.602\beta = 799 \text{ KN}$$

β = 0.386 (calculated)

β = 0.701 (reported avg.)

Total Stress Method

Surface area in clay = 13.67 m²

Sfc. area in till = 4.98 m², Sfc. area in SST = 2.10 m²

$$(13.67 + 4.98 + 2.10)Ca = 799 \text{ KN}$$

Ca = 39 KPa (calculated)

Cu = 103 KPa (reported average)

α = Ca/Cu = 0.379

Settlement

Pile No.1   Shaft Diameter = 0.51 m, Shaft Length = 13.0 m

Ultimate End Bearing(Q_{ult}) = 643 KN (as shown on A-13)

If Q/Q_{ult} = 0.33, then Q = 214 KN

Ultimate Shaft Capacity(R) = 875 KN (as shown on A-13)

Q_t = R+Q = 875 + 214 = 1089 KN

Corresponding settlement(ρ) = 8.0 mm

K = (8.0/510)3 = 0.047

Pile No.3   Shaft Diameter = 0.76 m, Shaft Length = 12.57 m

Ultimate End Bearing(Q_{ult}) = 1429 KN (as shown on A-14)

If Q/Q_{ult} = 0.33, then Q = 476 KN

Ultimate Shaft Capacity(R) = 1128 KN (as shown on A-14)

Q_t = R+Q = 1128 + 476 = 1604 KN

Corresponding settlement(ρ) = 8.5 mm

K = (8.5/760)3 = 0.034

Pile No.4   Shaft Diameter = 0.51 m, Shaft Length = 12.95 m

Ultimate End Bearing(Q_{ult}) = 643 KN (as shown on A-15)

If Q/Q_{ult} = 0.33, then Q = 214 KN

Ultimate Shaft Capacity(R) = 799 KN (as shown on A-15)

Q_t = R+Q = 799 + 214 = 1013 KN

Corresponding settlement(ρ) = 7.1 mm

K = (7.1/510)3 = 0.042

Santa Rosa Underpass Test Tie-Backs

Information from Foundation Investigation Report

Average Reported Soil Properties

γ(Clay) = 19 KN/m^3

γ(Till) = 21 KN/m^3

β(Clay) = 0.728

β(Till) = 0.839

Cu(Clay) = 97 KPa

Cu(Till) = 200 KPa

Soil Profile

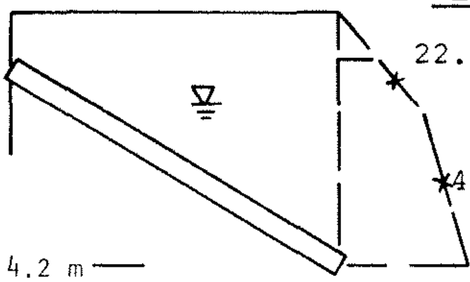
Tie-Back No.1

Shaft Diameter = 0.41 m

Shaft Length = 6.40 m

Ultimate Shaft Capacity = 276 KN

Effective Stress Method



Circumference = 1.288 m

$$\left[1.2(22.8) + 5.4(43.6)\right]1.288\beta = 276 \text{ KN}$$

β = 0.815 (calculated)

β = 0.728 (reported average)

Total Stress Method

Surface area in clay = 8.24 m^2

(8.24)Ca= 276 KN

Ca = 33 KPa (calculated)

Cu = 97 KPa (reported average)

a = Ca/Cu = 0.340

Tie-Back No.2

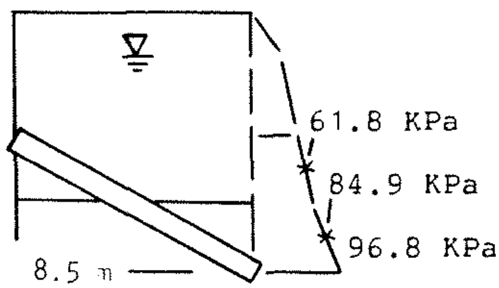
Shaft Diameter = 0.30 m

Shaft Length = 9.14 m

Ultimate Shaft Capacity = 445 KN

Effective Stress Method

Circumference = 0.942 m



$$\left[4.88(61.8) + 4.26(84.9)\right]0.942\beta = 445 \text{ KN}$$

β = 0.712 (calculated)

β = 0.789 (reported average)

Total Stress Method

Surface area in clay = 4.60 m^2

Surface area in till = 4.01 m^2

(4.60 + 4.01) = 445 KN

Ca = 52 KPa (calculated)

Cu = 145 KPa (reported average)

a = Ca/Cu = 0.359