The Capacity of Driven Steel H-Piles in Lacustrine Clay, Till and Karst Bedrock:

A Winnipeg Case Study

by

Ryan Belbas

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Abstract

Driven steel H-piles are becoming an increasingly practical deep foundation alternative in Winnipeg. They are a cost-effective and versatile pile type used to support heavy structures such as bridges and high rise buildings. The application of driven steel H-piles is relatively new in Winnipeg, consequentially limiting the knowledge of their ultimate load-carrying capacity (Q_{ult}) and load-displacement behavior. As a result of limited knowledge, the conventional limit states design (LSD) criteria is unreliable, as it bases pile capacity on the structural strength of steel as opposed to the strength and condition of the surrounding strata. Also, the serviceability limit state (SLS) criterion is incomplete as it ignores settlement. The research conducted for this thesis resulted in the development of comprehensive LSD criteria for driven steel H-piles in Winnipeg.

A substantial amount of Pile Driving Analyzer (PDA) test and Case Pile Wave Analysis Program (CAPWAP) data was collected and analysed to better understand the capacity and load-displacement behavior of driven steel H-piles in Winnipeg and to improve the conventional LSD criteria. Existing static and dynamic resistance analysis methods (RAMs) were refined to specifically estimate the Q_{ult} of driven steel H-piles in Winnipeg. Resistance factors (RFs) for the purpose of determining the ultimate limit state capacity (Q_{ULS}) are recommended on the basis of the CAPWAP-calculated capacities and refined RAMs. The CAPWAP-based load-displacement curves were examined to assess pile settlements under service loads to develop reasonable settlement design benchmarks and corresponding values of serviceability limit state capacity (Q_{SLS}).

Table of Contents

List of Tables

List of Figures

List of Copyright Material

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Glossary of Terms

AASHTO = American Association of State Highway and Transportation Officials

BOR = Beginning of Re-drive

CAPWAP = Case Pile Wave Analysis Program

CFEM = Canadian Foundation Engineering Manual

COV = Coefficient of Variation

EOID = End of Initial Drive

$f_{s\text{-}ult}$ = Ultimate Unit Shaft Friction Resistance

GRLWEAP = GRL Wave Equation Analysis Program

J_c = Case Damping

J_s = Smith Damping

LSD = Limit States Design

NBCC = National Building Code of Canada

PDA = Pile Driving Analyzer

$Q_{b\text{-}ult}$ = Ultimate End Bearing Capacity

$q_{b\text{-}ult}$ = Ultimate Unit End Bearing Resistance

Q_{SLS} = Serviceability Limit State Capacity

$Q_{s\text{-}ult}$ = Ultimate Shaft Friction Capacity

Q_{ULS} = Ultimate Limit State Capacity

Q_{ult} = Ultimate Capacity

q_u = Unconfined Compressive Strength

RAM = Resistance Analysis Method

RF = Resistance Factor

RQD = Rock Quality Designation

SLS = Serviceability Limit State

SLT = Static Load Test

SPT = Standard Penetration Test

s_u = Undrained Shear Strength

UC = Unconfined Compression

ULS = Ultimate Limit State

WEA = Wave Equation Analysis

Chapter 1: Introduction to Driven Steel H-Piles in Winnipeg

1.0 Introduction

Due to the rising construction costs of drilled shaft foundations, driven steel H-piles are becoming an increasingly viable deep foundation alternative. They are a cost-effective and versatile pile type used to support heavy structures such as bridges and high rise buildings. The conventional limit states design (LSD) criteria presumes that steel H-piles driven to refusal onto bedrock develop their capacity in end bearing as governed by the structural strength of steel. This assumption may be reasonable for piles seated on hard bearing strata such as massive granite bedrock but is not applicable for piles seated in weaker formations such as karst carbonate bedrock. The ultimate axial-compressive load-carrying capacity and load-displacement behavior of driven steel H-piles in this case is complex and cannot be defined by such simple criteria.

The simple and straightforward conventional LSD criteria lacks a geotechnical component, making it inadequate for H-piles seated in weak or karst bedrock formations and thick deposits of overburden soils. The capacity of driven steel H-piles is uncertain and varies depending on the condition of the bearing strata and the driving system. Labelling H-piles as entirely end bearing is fundamentally incorrect, since mobilization of the frictional shaft resistance must occur during loading (Tomlinson and Woodward 2008). This thesis investigates the shortcomings of the conventional LSD criteria and offers a more comprehensive examination of the bearing capacity and load-displacement behavior of driven steel H-piles in the Winnipeg area where karst bedrock formations preside beneath typically thick deposits of clay and till.

The application of driven steel H-piles is relatively new to Winnipeg when compared to other deep foundation alternatives; thus, relatively little is known regarding this pile type. Fortunately, the emergence of high strain dynamic load testing, via the Pile Driving Analyzer (PDA), and the Case Pile Wave Analysis Program (CAPWAP) analysis into the Winnipeg piling industry in recent years have resulted in the availability of an abundance of data. The new data has made it possible to perform a detailed study to increase the local knowledge of the actual ultimate capacity (Q_{ult}), and the load-displacement and driving behavior of driven steel H-piles. The research performed in this thesis is based on PDA test and CAPWAP analysis data compiled from within Winnipeg.

The PDA and CAPWAP are the most advanced resistance analysis methods (RAMs) that are used in Winnipeg to estimate the Q_{ult} of driven piles. The Case Pile Wave Equation Program has shown excellent correlation to static load tests (SLTs) and is considered by the author to be superior in terms of time, cost and ease of implementation. The Case Pile Wave Equation Program analysis should be the standard RAM at the design stage and the standard verification tool at the production stage for determination of ultimate H-pile capacity. Furthermore, the author believes CAPWAP analysis should be given the same weight in LSD as a SLT in terms of reliability.

Uncertainty and variability of loading conditions and ground conditions are managed by LSD (Likins et al. 2012). Limit states design has emerged into geotechnical engineering over the past few decades and is now the required design method for deep foundations in Winnipeg. To account for the uncertainty and variability of the ultimate limit state

(ULS), or failure, condition of a pile in LSD, a resistance factor (RF) is applied to the RAM that is used to estimate Q_{ult}. The resistance is based on the reliability associated with that particular RAM (Likins et al. 2012); the more reliable the RAM, the higher the RF. The serviceability limit state (SLS) condition in LSD is governed by the required level of performance of a pile during service.

Until recently, the working stress design method constituted the conventional design criteria. A single recommendation of allowable capacity was generally provided in geotechnical reports. Allowable capacity encompassed both the failure (ultimate) and performance (serviceability) that are handled with LSD. This thesis will provide engineers with the information to make the distinction between failure and performance and to provide recommendations for ULS capacity (Q_{ULS}) and SLS capacity (Q_{SLS}) of driven steel H-piles in Winnipeg.

Refined resistance analysis methods used to determine the Q_{ULS} and Q_{SLS} of driven steel H-piles in Winnipeg are presented in this thesis. These methods are based on the measured Q_{ult} and the load-displacement behavior of the H-piles as determined by CAPWAP analysis. The ultimate limit state capacity is determined by applying an appropriate RF to the Q_{ult} as predicted by the refined static or dynamic RAMs presented in the thesis. The serviceability limit state capacity is determined on the basis of tolerable amounts of settlement at the pile head as determined by the CAPWAP load-displacement curves. The refined RAMs form the basis of the recommended LSD criteria presented at the end of the thesis.

Results from sixty-four PDA tests and CAPWAP analyses performed on H-piles from twelve different sites within Winnipeg were collected. The data was back-analysed and correlated to static and dynamic parameters to form new and refine existing RAMs that can be used to specifically estimate the Q_{ult} of H-piles driven in Winnipeg. The refinement of the static RAMs focused on the ultimate unit shaft friction resistance (f_{s-ult}) of an H-pile in lacustrine clay and till, and on the ultimate unit end bearing resistance (q_{b-ult}) of an H-pile in till and bedrock. The principal goal of the dynamic analysis was to refine the dynamic RAM input properties to accurately predict the Q_{ult} of H-piles. General bearing graphs were produced, relating measured Q_{ult} to the measured penetration resistance, pile size, and measured transferred energy to evaluate their influence on H-pile capacity in lacustrine clay, till and, karst bedrock during driving, and to refine the conventional refusal criterion.

The static RAMs included in the analyses are the alpha-method (α-method), the standard penetration test method (SPT-method) and the q_u-method. The dynamic RAMs include the modified Gates formula, the GRL wave equation analysis program (GRLWEAP), the PDA, and CAPWAP. The RAMs were calibrated to agree with the measured capacities of the driven steel H-piles. The CAPWAP analysis data was correlated to laboratory and *in-situ* soil tests to improve the reliability of the static RAMs, resulting in the refined static RAMs. Increased knowledge of the dynamic variables was realized from the measured data determined during the PDA tests and CAPWAP analyses, which led to increased reliability of the dynamic RAMs and the formation of the refined dynamic RAMs. This is not meant to discourage the continued application of the PDA test and

CAPWAP analysis in any way, but to indicate their significance and value in a broader sense to their application as stand-alone RAMs.

The investigation into the serviceability of driven steel H-piles in Winnipeg was based on the load-displacement curves created from the CAPWAP analyses. The load-displacement curves were examined to establish the amount of H-pile capacity that was mobilized at the pile toe and pile head at tolerable settlements. Settlements of 1 and 2 mm at the pile toe and 5 and 10 mm at the pile head were considered tolerable and therefore were considered as the benchmarks for the serviceability analyses. These settlements were considerably less than those mobilized at failure.

The thesis discusses the conventional Winnipeg design criteria and the fundamentals of driven steel H-piles. Existing static and dynamic RAMs that are currently employed in practice will be discussed as well as the theory behind the PDA test and CAPWAP analysis. The results of the research will be presented and compared to the results predicted by the conventional LSD criteria and existing RAMs. Refinement of the RAMs and exploration into the serviceability will follow. Finally, conclusions of the research and LSD recommendations for driven steel H-piles in Winnipeg will be provided.

1.1 Hypothesis

The main purpose of this thesis is to improve the conventional LSD methodology currently used in Winnipeg by making it more analytically complete, accurate, and comprehensive. The goals that must be met to fulfill the purpose of this thesis are fourfold: 1) to prove that driven steel H-pile capacity is based on geotechnical parameters, 2) to develop, evaluate, and refine RAMs to accurately predict the Q_{ult} of H-piles for the purpose of LSD, 3) to provide the analytical tools to develop a refined refusal criterion, and 4) to establish a level of performance in terms of settlement of H-piles under service loading. A secondary purpose of this thesis is to make known the applicability, importance, and benefit of PDA testing and CAPWAP analysis for H-pile design and quality assurance.

Optimistically, this thesis will accomplish four things: 1) refine the RAMs to more accurately estimate the Q_{ult} of driven steel H-piles, 2) establish a new RAM consisting of general bearing graphs used to predict the Q_{ult} and refine the refusal criterion, 3) justify using higher RFs, compared to the National Building Code of Canada (NBCC), for ULS design, and 4) convince engineers to employ the PDA test and CAPWAP as regular RAMs. Realistically, this thesis will improve the local knowledge of the Q_{ult}, load-displacement behavior, and driving behavior of driven steel H-piles and further encourage consultants to use the PDA test and CAPWAP analysis at the production stage to take advantage of high RFs.

This thesis document will examine the following four-part hypothesis:

1. The Q_{ult} of driven steel H-piles is governed by the strength and condition of the strata and the pile driving system; not the structural strength of steel. High variability will be observed in the measured Q_{ult} values of the H-piles due to the non-uniform

geotechnical components that govern capacity. Also, driven steel H-piles seated in lacustrine clay, till, and karst bedrock are not entirely end bearing piles.

2. Resistance analysis methods can be refined to more accurately predict the Q_{ult} of driven steel H-piles seated in lacustrine clay, till, and karst bedrock. Higher RFs (in comparison to the 2010 version of the NBCC) can be used to determine Q_{ULS} based on the refined RAMs.
3. General bearing graphs can be constructed and used as a RAM to predict Q_{ult} and the corresponding refusal criterion of driven steel H-piles in lacustrine clay, till, and karst bedrock by incorporating measured transferred hammer energy, measured Q_{ult} and ultimate end bearing capacity (Q_{b-ult}), pile size, and observed penetration sets.
4. The CAPWAP-based load-displacement curves can be used to estimate pile settlement and develop reasonable SLS criteria for calculation of Q_{SLS} for driven steel H-piles.

1.1.1 Thesis Objectives

The main objectives of this research are to:

1. Compile and analyze measured Q_{ult} results from PDA tests and CAPWAP analyses on sixty-four driven steel H-piles from twelve project sites in the Winnipeg area.
2. Compare CAPWAP-calculated capacities with Q_{ult} as predicted by the conventional LSD criteria.
3. Assess the shaft and end bearing resistance distribution.
4. Back-analyze adhesion values of driven steel H-piles in lacustrine clay and correlate to undrained shear strength (s_u).

5. Back-analyze $f_{s\text{-}ult}$ of driven steel H-piles in till and correlate to SPT N-values and moisture contents.
6. Back-analyze $f_{s\text{-}ult}$ of driven steel H-piles in bedrock and correlate to unconfined compressive strength (q_u).
7. Back-analyze $q_{b\text{-}ult}$ of driven steel H-piles seated in till and correlate to SPT N-values and moisture contents.
8. Back-analyze $q_{b\text{-}ult}$ of driven steel H-piles seated in bedrock and correlate to q_u.
9. Create bearing graphs based specific to driven steel H-piles in Winnipeg on the basis of measured values to assess the Q_{ult} and the corresponding penetration resistance, pile size and transferred driving system energy.
10. Evaluate and refine the accuracy of the dynamic RAMs, including the modified Gates formula, GRLWEAP and the Case method for predicting the Q_{ult} of driven steel H-piles on the basis of measured CAPWAP dynamic properties.
11. Evaluate the load-displacement behavior of driven steel H-piles on the basis of measured ultimate CAPWAP capacities and tolerable settlements at the pile head to develop SLS criteria and determine reasonable Q_{SLS} values.

1.2 Thesis Organization

The remainder of this chapter briefly describes deep foundations in Winnipeg including, historic and common present-day pile types, the emergence of driven steel H-piles, and the conventional design methodology currently employed. Chapter 2 contains a literature review presenting background information of driven steel H-piles, general pile behavior, and static and dynamic RAMs. Chapter 3 describes the engineering properties of lacustrine clay, till, and karst bedrock in Winnipeg, as well as the concept of LSD. This chapter also includes a brief description of the strata and construction details at the twelve sites. Chapters 4, 5, and 6 present the results of the field testing, geotechnical static and dynamic analyses, load-displacement behavior, and the procedure used to determine the recommended RFs. Chapter 7 provides a thesis summary, conclusions, and recommendations for design and future research.

1.3 Case Study: Driven Steel H-piles in Winnipeg

Driven steel H-piles are relatively new to Winnipeg. Their application has been unusual until recently, consequentially limiting the knowledge of the Q_{ult}, serviceability and driving behavior. The geotechnical static and dynamic components of driven steel H-piles are not well defined as other pile types commonly used. To the author's knowledge, only limited research has been conducted on this pile type within the city limits of Winnipeg. Research on driven steel H-piles may not have been justified in the past, but now has become important with their increased use in Winnipeg.

1.3.1 Early Foundations Types in Winnipeg

The City of Winnipeg is located at the eastern edge of the Canadian prairies in the basin of pro-glacial Lake Agassiz. The upper stratigraphic soil units in this area consist primarily of compressible lacustrine clays of relatively low bearing resistance in comparison to the underlying till and bedrock. These clays are also susceptible to frost heave and sensitive to seasonal moisture changes that lead to volume change. The clay layer has been problematic for the performance of shallow foundations and this led to the evolution of deep foundations as a preferred foundation alternative.

Many buildings were constructed on shallow foundations such as rafts and spread footings bearing on the compressible clay in the early development of Winnipeg. The understanding of bearing capacity during this era was uncertain and the performance of structures built on shallow foundations was variable due to settlement. As buildings seated on shallow foundations aged, differential settlements became evident and damaging (Fosness 1926). Shallow foundations experienced serviceability issues due to these settlements, as can often be seen buildings and homes in older neighborhoods. Major failures of shallow foundations have occurred such as case of the Transcona Grain Elevator collapse in 1913. The cost of repairing the structural damage of buildings and mitigating further settlements by underpinning was high. It had become evident that deep foundations were a more reliable foundation option.

Caissons advanced to bedrock were also common during this time to support heavy loads. They were excavated by hand, making it difficult and time consuming to construct. Caissons were limited to minimum diameters of about 1.2 m making them non cost-effective for loads less than 3000 kN (Fosness 1926). The cost of caissons increased substantially when unfavorable ground conditions were present, such as when wet sand and gravel layers were encountered during excavation. Driven end bearing piles and driven friction piles were also used at the time and were cost-comparable to shallow foundations (Fosness 1926). The piles had much smaller diameters than caissons and therefore could be used for reasonably lighter loads. Practically no settlements had been observed where deep foundations were used (Fosness 1926).

Uncertainty, risk, increased structural loading, a reduced tolerance for differential settlements, modernization of construction equipment (*i.e.* drilling equipment), and improved installation methods have made deep foundations a more practical foundation type for supporting buildings and bridges. Deep foundations are now used to resist axial-compressive loads for most homes and buildings in Winnipeg, for these reasons.

1.3.2 Present Day Pile Types in Winnipeg

Conventional pile types in Winnipeg are cast-in-place concrete piles and caissons, driven pre-cast concrete piles, and driven steel H-piles. Timber piles, steel pipe piles, micro-piles, and other pile types have their applications and are used occasionally, but are rare.

Most pile types in Winnipeg are designed using conventional geotechnical methods. Cast-in-place concrete piles and caissons are typically designed using empirical geotechnical static analyses based on soil strength properties. Driven pre-cast concrete piles are designed on the basis of accepted empirical research which correlates the dynamic behavior of the pile to SLTs and penetration resistance. Driven steel H-piles are relatively new to Winnipeg and have not been given the same analytical attention that other pile types have in regard to developing a conventional geotechnical design criteria.

The following subsections briefly discuss the common types of piles currently used in Winnipeg and the corresponding geotechnical-based design analyses used to determine their bearing capacity under axial-compressive loads.

1.3.2.1 Cast-In-Place Concrete Friction Piles

Cast-in-place concrete friction piles are commonly used to support light to medium loads that coincide with residential homes or one to two storey office buildings. Friction piles are relatively small diameter piles that are installed in the upper clay layer consisting of adequate thickness above the till. Under loading, pile capacity is developed through mobilization of shaft friction resistance. The entire load is taken up in the pile shaft. The end bearing capacity of clay is considered negligible and is generally not included in design. The α-factors used in Winnipeg for design are a function of s_u, which is commonly determined by unconfined compression (UC), torvane, lab vane, or pocket penetrometer tests. Observation of sub-surface conditions during the geotechnical investigation establishes the constructability of this pile type.

1.3.2.2 Cast-In-Place Concrete Spread-Bore Piles

Cast-in-place concrete spread-bore piles are generally used to support light to medium loads and are usually installed in till of sufficient bearing resistance. Moisture contents, SPTs and good documentation of sub-surface conditions observed during the geotechnical investigation provide sufficient information to determine the constructability and end bearing pressures used for design. Spread-bore piles are generally selected in lieu of friction piles for lighter loads due to an insufficient available thickness of clay, resulting in inadequate shaft friction resistance for a reasonably sized pile. The shaft friction resistance of belled piles is fully mobilized under the applied load. Load beyond the ultimate shaft friction capacity ($Q_{s\text{-}ult}$) is transferred to the pile toe and resisted in end bearing. The shaft friction resistance is not expected to be significant relative to the end bearing resistance and is not generally included in the design.

1.3.2.3 Cast-In-Place Concrete Caissons

Cast-in-place concrete straight-shaft caissons end bearing in very dense till are large diameter piles used to support medium to heavy loads. Moisture contents, SPTs, and good documentation of ground conditions observed during the geotechnical investigation is usually sufficient information to estimate the available end bearing resistance. Pre-construction test caissons are sometimes incorporated in the design phase to determine constructability. Down-hole inspection during the construction of production piles is often required to confirm high design bearing pressures. Shaft friction resistance is considered insignificant and is typically ignored in design.

1.3.2.4 Rock Socketed Caissons

Rock socketed caissons are used to support heavy structural loads such as bridge piers and abutments and high rise buildings. Rock quality designation (RQD) and UC tests of rock cores retrieved during the geotechnical investigation are important design parameters for rock-socketed caissons and provide the basis for side shear values used. The most important aspect of this deep foundation alternative is constructability and for this reason it is common to perform test caissons to aid in the design process. Rock socketed caissons are not utilized frequently, but are a choice foundation for large structural loads or when dense to very dense till is absent. Shaft friction resistance above the rock socket is considered insignificant and ignored. End bearing resistance is rarely considered during design, unless down-hole inspection or proof drilling is performed.

1.3.2.5 Driven Precast Prestressed Concrete Hexagonal Piles

Driven precast prestressed concrete hexagonal piles are the most common deep foundation option for medium to heavy axial loads in Winnipeg. The bearing capacity of these piles is based on three pile studies compiled into a single document titled, *The Manitoba Precast Prestressed Concrete Pile Capacity Study*. The study was conducted in 1978 by M. Block and Associates, R.M. Hardy and Associates and Klohn Leonoff Consultants Ltd. The ultimate and allowable pile capacities and refusal criterion of driven PPCH piles, discussed in this study, were developed by correlating the SLT results to the penetration resistances recorded during installation. These piles are driven to practical refusal in the dense to very dense till. The estimated refusal depth can be determined by moisture contents and SPTs. The results of the study include recommended refusal criterion and related pile capacity, which are described in full in the document.

1.3.2.6 Driven Steel H-piles

Driven steel H-piles are becoming more common in Winnipeg for supporting heavy axial-compressive loads. The understanding of the bearing capacity and behavior and of this pile type is relatively deficient. Steel H-piles, when driven to hard rock stratum, are regarded as completely end bearing and transfer practically the entire load to the pile toe (Tomlinson and Woodward 2008). The design of H-piles in this case is based on the strength of steel (Bowles 1997). Theoretical and empirical geotechnical RAMs are not typically employed to estimate bearing capacity or settlement in Winnipeg, and no rigid dynamic refusal criterion exists. A majority of local knowledge of steel H-piles is a result of experience and traditionalism.

The Red River Floodway Expansion Project in the early 2000s was the first major use of driven steel H-piles within the Winnipeg area. They were used in every bridge foundation that was constructed as part of this major project. The realized cost and time savings during this project spear-headed a new generation of pile foundations to support heavy loads in Winnipeg. Many buildings and bridges that are constructed in Winnipeg today are founded on driven steel H-piles. One of the reasons why driven steel H-piles were not employed prior to the Red River Floodway Expansion Project was the concern of corrosion of the steel. Engineers were reluctant to use steel piles, in the early to mid-1900s, due to the errant currents in the ground created by the electric train cars.

1.3.3 Design Recommendations for Driven Steel H-Piles in Winnipeg

Limit state design recommendations provided in geotechnical reports in Winnipeg for driven steel H-piles are quite generic. They are based on the now outdated working stress design method. A single global formula is provided to calculate the Q_{SLS} with a general refusal criterion; the Q_{ult} is estimated by multiplying the Q_{SLS} by a safety factor; and the Q_{ULS}, in turn, is determined by applying a RF to the Q_{ult}. More detailed geotechnical reports may include recommendations for wave equation analysis (WEA) at the design stage to specify refusal criterion, hammer type, pile size, and to assess driveability. Dynamic load tests are often recommended during the production stage to confirm the Q_{ult} and refusal criterion, to assess the efficiency of the driving system, and to monitor driving stresses and pile integrity. Geotechnical investigations are always performed and the soil and bedrock properties are always available whether or not they are used. A typical geotechnical investigation and the conventional design criteria and its shortcomings are described below.

1.3.3.1 Geotechnical Investigation and Soil Testing

Recommendations for foundations are always preceded by a geotechnical investigation. The geotechnical investigation will include one or more test holes, depending on the scale of the project, to classify the overburden soils and bedrock, observe the sub-surface conditions (*i.e.* groundwater seepage, sloughing, etc.) and to determine feasible foundations alternatives. Test holes are often drilled at each pier and abutment location for bridges and multiple locations around the proposed site for buildings to minimize the uncertainty resulting from spatial variability of the stratigraphy. The number of test holes depends on the scale of the project and the risk and uncertainty that the client or consultant is willing to absorb. A detailed visual classification of the soil and bedrock is completed during the geotechnical investigation. *In-situ* strength testing such as SPTs are frequently conducted and disturbed and undisturbed soil samples and rock core samples are regularly collected for laboratory testing.

Classification of clay is completed by performing laboratory testing on undisturbed Shelby tube samples and disturbed grab samples. Laboratory testing to determine s_u may include torvane, pocket penetrometer, Q_u, lab vane and on rare occasions, triaxial tests. Testing may also include unit weight measurements, Atterberg limits and grain size analyses. Moisture contents are usually conducted on all soil samples.

Classification of the bearing strength of till is completed from *in-situ* SPTs and determination of moisture contents. Shelby tube samples can be retrieved in the till for determination of unit weights if a low fraction of gravel and cobbles is present. Atterberg limits are at times conducted for further classification of the till or to estimate the state of consolidation by comparing the moisture content to the plastic and liquid limits.

Classification of the bedrock is conducted to estimate the side shear and bearing resistance of the rock. Rock quality designation and a general visual classification are usually performed during the time of the investigation. Pressuremeter tests performed at the time of drilling have been conducted in the bedrock in the past, but are very rare today. Unconfined compression tests of intact core samples are occasionally completed in the laboratory to determine the q_u of the rock. The detailed lithology of the bedrock is usually performed in the laboratory as well by a geologist or a geotechnical engineer experienced in geology.

Good observation of ground conditions during drilling of the test hole(s) is of the utmost importance for determining foundation constructability. The frequency of cobbles and boulders in the till becomes a factor with both driven and drilled shaft installation. Stability of the test hole and ground water seepage is an essential constructability issue for drilled shafts.

1.3.3.2 Conventional Limit State Design Criteria

The LSD recommendations for driven steel H-piles in Winnipeg presented in this section will be referred to as the conventional design criteria for the remainder of this thesis. These criteria may not be considered as conventional by all engineers; however, on the basis of the author's experience and for the purpose of this thesis, they are considered as conventional. The conventional design criteria for driven steel H-piles in Winnipeg include the following components:

- The Q_{ult} of steel H-piles driven to practical refusal onto competent bedrock is governed by the structural strength (or permissible stress) of the steel and the cross-sectional area of the H-pile section as expressed by the following equation,

  $$Q_{ult} = 0.9 f_y' A_p \tag{1.1}$$

  Where, f_y' is the yield stress of the steel and A_p is the cross-sectional area of the H-pile section. The factor of 0.9 is a structural RF that is applied to the equation to take into account the uncertainty of the steel strength and stability properties and chemical and electrical corrosion. Other common criteria for calculating Q_{ult} have been provided in geotechnical reports and include the following equations,

  $$Q_{ult} = 0.6 f_y' A_p \tag{1.2}$$

  $$Q_{ult} = 0.75 f_y' A_p \tag{1.3}$$

The concept of Equations 1.2 and 1.3 is identical to Equation 1.1, in the manner that Q_{ult} is based on the structural strength of steel and the cross-sectional area of the H-pile section. The factors of 0.6 and 0.75 are determined by applying factors of safety of 2 and 2.5, respectively, to the Q_{SLS} as expressed by Equation 1.4 (below). These are typical safety factors used in the old working stress design method.

- The Q_{SLS} of driven steel H-piles is also governed by the structural strength of the steel and the cross-sectional area of the H-pile section. Serviceability limit state capacity is equal to the "allowable capacity" as determined from the working stress design method. It is calculated by applying a global factor of safety of 3 to Equation 1.1, and is expressed as,

$$Q_{SLS} = 0.3 f_y' A_p \tag{1.4}$$

The factor of safety of 3 was applied to take into account all geotechnical, construction, and loading uncertainty.

- The Q_{ULS} is determined by applying a RF to Q_{ult} and is expressed as,

$$Q_{ULS} = \Phi Q_{ult} \tag{1.5}$$

Where, Φ represents the RF. The NBCC (2010) suggests a RF of 0.6 if a SLT is performed, 0.5 if dynamic monitoring is used as the RAM, and 0.4 if the RAM is based on *in-situ* or laboratory testing. These RFs are equivalent to factors of safety equalling 1.7, 2.0 and 2.5, respectively, in terms of the working stress design method.

- The refusal criterion is considered to be three consecutive sets of 1.7 mm of pile penetration per hammer blow, or inversely, 15 blows per 25 mm of pile penetration. However, values as low as 1.3 and as high as 2.5 mm per blow (20 to 10 blows per 25 mm) have been recommended.

- The H-piles are to be driven with a diesel hammer having a minimum rated energy of 50 kJ per blow or a hydraulic drop hammer having a minimum rated energy of 30 kJ per blow.
- Wave equation analysis, including a driveability analysis, should be performed after the pile size and hammer type are selected to refine the refusal criterion.
- Dynamic load testing should be conducted at the on-set of construction to verify the pile capacity and refusal criterion, and to monitor pile stresses and integrity. Pile stresses induced during driving should be limited to a maximum of 75 or 90 percent of the yield stress of steel.

The conventional design criteria for driven steel H-piles in Winnipeg are incomplete as it generally lacks comprehensive geotechnical ULS and SLS components. The geotechnical constituents are generally limited to feasibility of construction, the depth to massive bedrock, and the frequency of boulders in the till. The strength properties of the overburden soils and bedrock that are provided in the report are typically ignored in providing recommendations for calculating Q_{ult}, Q_{ULS}, and Q_{SLS} of driven steel H-piles. The perceived justification for use of the above design criteria is based on the theory that the structural strength of steel governs the capacity of piles when they are seated on strong and massive bedrock, where the rock mass is considered to be stronger than the pile material. However, in the author’s opinion, the karst bedrock underlying Winnipeg is weaker than steel. Therefore, the rock’s strength should govern capacity in contrast to the steel’s strength.

1.3.3.3 Shortcomings of Conventional Design Practice

What should be noticed in the conventional design criteria is that they almost completely ignore geotechnical constituents by basing Q_{ult}, Q_{ULS}, and Q_{SLS} of driven steel H-piles solely on the strength and size of the structural member. Defining these capacities on the basis of the structural strength of steel may not be justified for a couple of reasons when considering the geotechnical strength properties of till and karst characteristics of the bedrock underlying Winnipeg. First, the q_u of intact cores of the bedrock is typically less than the yield stress of steel (Baracos et al. 1983a). The same can be said about the weaker bearing pressures of till. Second, a discontinuous formation is comprised of vertical and horizontal fractures, voids, and solution cavities that may make the rock mass weaker than intact rock (Kulhawy and Carter 1992). If H-piles are driven into weak, fractured or weathered rocks, and strong massive bedrock is absent, they should be considered as partly friction piles (Tomlinson and Woodward 2008). The theory that H-pile capacity is governed by the structural strength of steel would be reasonable if Winnipeg was underlain by a massive and strong formation exhibiting bearing pressures greater that the yield stress of steel. The understanding of the potential bearing resistance of the till and underlying bedrock is of great importance, based on these circumstances.

The main concern with the SLS criterion is that is ignores settlement. The performance of a pile is evaluated by the amount of settlement it undergoes under service loads. Therefore, omitting settlement from the SLS criterion essentially ignores the main component that defines serviceability. Ideally, during the recommendation process, the structural engineer would provide a value of tolerable settlement to the geotechnical engineer. To provide a complete SLS criteria recommendation, the geotechnical engineer should be able to provide a Q_{SLS} with a maximum expected settlement.

One of the main issues with driven steel H-pile design is the interpretation of the term refusal. Absolute refusal should be considered to be a state in which the H-pile is seated on strong massive strata where continued penetration is almost zero and further driving may damage the H-pile. This may occur when an H-pile is seated on close-jointed Precambrian bedrock such as granite or on massive intact carbonate bedrock exhibiting very high strengths. The structural strength and H-pile size governs capacity in this situation and resistance to axial-compressive loading is almost entirely in end bearing with negligible shaft friction (Tomlinson and Woodward 2008). Some limestone rock is relatively soft however and absolute refusal cannot be achieved according to Chellis (1961); in this case H-piles are driven to practical refusal. Practical refusal is the state of pile driving where the desired H-pile capacity has been achieved and further driving is not justified or further gain in capacity is not expected. Practical refusal of H-piles can occur on boulders, in thick layers of till, or in relatively weak karst bedrock. In these cases, H-piles are expected to resist loads partially in end bearing and partially in shaft friction as governed by the geotechnical and geological elements.

1.4 Chapter 4 Discussion

Understanding the development of the axial-compressive load-carrying capacities (*i.e.* Q_{ult}, Q_{ULS}, and Q_{SLS}) of driven steel H-piles and the refusal criterion is more complex than simply stating that the structural strength of steel governs the bearing capacity when driven to 15 blows per 25 mm, because in many cases it does not. It should be conceded that the bearing capacity of a steel H-pile driven in lacustrine clay, till, and karst bedrock formations should not be determined on the basis of the pile material, but the strength of the strata. It should also be recognized that steel H-piles should be expected to develop shaft friction resistance under loading, since shaft friction resistance must mobilize prior to mobilization of end bearing resistance; long steel H-piles should be expected to develop significant shaft friction resistance in overburden soils.

In Winnipeg, the geotechnical properties of clay, till and bedrock are readily available and need to be considered for estimation of H-pile capacity. Many static and dynamic RAMs exist for piles and will be discussed in detail in the next chapter. This thesis undertook the task of acquiring the knowledge of the capacity, serviceability and driving behavior of driven steel H-piles, to provide the framework for the recommended LSD design criteria presented at the end of the thesis.

Chapter 2: Literature Review

2.0 Introduction

Chapter 2 presents the theoretical information required to understand the research analyses and results presented in the following chapters. This chapter begins by reviewing the advantages (and disadvantages), applications, installation, driveability, soil plugging, and failure modes of driven steel H-piles. The Q_{ult}, load transfer, resistance mobilization, and potential failure modes of piles in general are also examined. To finish this chapter, the background of the static and dynamic RAMs (that are discussed in Chapter 1) and the load-displacement curves used in this thesis to calculate Q_{ult}, Q_{ULS}, and Q_{SLS} of H-piles subjected to axial-compressive loading are explained.

2.1 Piles

A pile is a slender structural member composed of concrete, steel, or wood (Budhu 2000). A pile transfers structural load through weak compressible soil to hard stratum of adequate end bearing resistance (Tomlinson and Woodward 2008), or provides sufficient length to develop resistance in shaft friction. Piles are used in place of shallow foundations when the bearing resistance of the upper soil layer is insufficient or the layer is intolerably compressible.

2.1.1 Driven Steel H-piles

Steel H-piles are high-capacity, low-displacement piles that are meant to be driven to refusal onto hard stratum. The name H-pile refers to the shape of the pile cross-section. In the Handbook of Steel Construction (2004), H-piles are referred to as HP shapes.

They consist of two flanges and a web of varying widths, depths, and thicknesses. The H-piles are formed using a metal forming process called rolling. They are rolled to the required ASTM Structural Steel Specifications (Handbook of Steel Construction 2004), which include a variety of sizes with varying grades of steel. H-piles can be driven using conventional piling equipment capable of transferring enough energy to penetrate the pile to the desired depth and mobilize the full end bearing capacity in a hard stratum.

2.1.1.1 Advantages and Disadvantages

The benefits of driven steel H-piles are attractive to both contractors and designers due to their low costs, simple design, easy installation, and high bearing capacity. H-piles are versatile, easy to handle, and have good driving characteristics. Based on Tomlinson and Woodward (2008), Chellis (1961), Peck et al. (1974), and the experience of the author, the advantages of using driven steel H-piles are numerous and include the following:

- High load-carrying capacity
- Can be installed with conventional driving equipment
- Can withstand high driving stresses
- Penetration proficient
- Have good resistance to buckling
- Reduced ground vibrations
- low displacement of the bearing soil – minimal heave of adjacent piles or structures
- Easy to cut and weld
- Durable
- Low supply and construction costs for heavy structural loads

Driven steel H-piles are capable of shearing through stiff (or dense) overburden soils and penetrating through fractured bedrock due to their small cross-sectional area and the relatively high strength of the steel. Evidence of significant penetration proficiency of driven steel H-piles has been observed in hardpan (very dense cemented till) and cemented sands (Chellis 1961). The low soil displacement of driven steel H-piles is beneficial in the case where infill projects require piles to be driven close to existing structures and heave is unacceptable (Chellis 1961). Significant cost savings can be achieved with driven steel H-piles when driven through thick deposits of overburden soils to a hard stratum.

There are a few disadvantages to using driven steel H-piles. They are not cost-effective for supporting light structural loads or when used as short friction piles. They should not be expected to reach refusal in thin layers of dense material, due to their penetration proficiency. They may be re-directed during driving and become vertically out of plumb when large boulders are encountered.

2.1.1.2 Applications

Driven steel H-piles are applicable to many different geological settings. They are the preferred foundation choice in terms of cost and constructability to support heavy loads when driven through thick weak deposits to refusal in a hard stratum (*i.e.* clay overlying bedrock). H-piles are favoured in Winnipeg when dense till is absent or too deep, rendering straight-shaft end bearing caissons too expensive. They are also favourable when the bedrock is highly fractured and rock-socketed caissons are too expensive to install. H-piles are more practical than driven concrete piles in terms of driveability when the presence of boulders is frequent.

2.1.1.3 Installation

Steel H-piles can be driven using conventional diesel and hydraulic hammers. The hammers must provide enough energy to drive the pile through dense soils to overcome the resistance of the soil and achieve permanent displacement. The amount of hammer energy required to drive an H-pile to bedrock depends on the size of the pile. In Winnipeg, large H-piles (*e.g.* HP360x132 section) are commonly driven with hammers consisting of rated energies as high as 60 kN·m. In general, hammers used in Winnipeg do not cause pile damage due to the high yield stress of the steel.

Some other general construction aspects of driven steel H-piles should be discussed. H-piles should be equipped with a protective shoe at the toe to resist toe damage during driving. Likins et al. (1977) illustrated the damage incurred in H-piles not equipped with a driving shoe. The majority of unprotected piles experienced buckling at the toe during hard driving. Also, full penetration butt welds are required for splicing multiple H-pile sections together. It is not uncommon to have one or two splices per pile.

2.1.1.4 Driveability and Durability

H-piles have excellent driveability characteristics. They can withstand high driving stresses when driven through hard layers and layers with reasonably sized boulders without compromising pile integrity. H-piles can penetrate through fractured, weathered, and weak rock to reach more massive and stronger rock. Steel H-piles are fairly rugged and can be roughly handled without breaking. They are easy to move around a job site and prepare for driving.

2.1.1.5 Effects of Soil and Rock Plugs

Opinions on the formation of a soil plug vary. According to (Hussein et al. 2003), a soil plug may form between the flanges and the web of an H-pile during driving, changing the driving characteristics to that of a displacement pile. According to Tomlinson and Woodward (2008), an H-pile driven into silty and sandy soils does not form a soil plug. The Federal Highway Administration (FWHA 2006), states that it can usually be assumed that a soil plug is formed during driving of an H-pile in both fine-grained and coarse-grained soils. The FWHA (2006), also states that steel H-piles driven to rock should not be considered to have a rock plug as the fractured rock, travelling up the flanges and web as the pile penetrates the ground, alters the plug to a weaker state than the underlying rock. Chellis (1961) states that it is important to identify the presence of a soil plug for calculation of shaft friction resistance, since shaft friction develops around the outside (box) perimeter of the pile when a soil plug forms.

The author has the following opinions regarding H-piles and the formation of soil and rock plugs in lacustrine clay, till, and karst bedrock, and the influence the plugs have on pile capacity:

- It is common practice in Winnipeg to assume that an H-pile forms an intact soil plug when driven through clay. The plug forms as the pile shears through the soil and the soil becomes confined between the flanges and web. Shaft friction resistance is assumed to only develop around the outside (box) perimeter of the pile as a result of the formation of the soil plug, as shown in Figure 2.1a. The clay plug is not expected to influence driveability since the H-pile shears through the clay with relative ease.

- A driven steel H-pile forms disturbed soil and rock plugs when driven through compact to very dense till and karst bedrock. The weight of the overburden material and the subsequent lateral stresses maintain the contact between the disturbed plugs and the surrounding soil and rock at the plug interface. Shaft friction resistance therefore also develops around the outside (box) perimeter of the H-pile as illustrated in Figures 2.1b and 2.1c. The till plug is not expected to influence driveability of an H-pile due to the disturbed nature of the plug. H-piles driven through loose and moist to wet till are liable to form soil plugs in a manner similar to those driven in clay.
- Figure 2.2 illustrates a presumed rock plug at the pile toe at refusal. The disturbed rock plug at the pile does not increase the end bearing area of an H-pile. The rock plug will yield prior to the intact rock at the toe when a load is applied, reducing the end bearing area to that of the cross-sectional area of the steel (Tomlinson and Woodward 2008).

2.1.1.6 Failure Modes in Karst Bedrock

The end bearing failure mode of a driven steel H-pile is difficult to determine due to the complexity of its shape. The slenderness of the H-pile generates a small zone of influence that is not relatively deep (Tomlinson and Woodward 2008), possibly negating the scale effects of a relatively large rock mass. High stress concentrations in the rock that occur at the toe during driving, due to the small cross-sectional area, may lead to degradation of the rock due to crushing or splitting. The ability of rock to withstand high stresses depends partly on the q_u of the intact rock and the discontinuities in the rock mass, which consist of vertical and horizontal joints, fractures and voids. Considering these characteristics of driven steel H-piles and karst bedrock formations, the author suggests that failure in compression is the probable failure mechanism.

Failure of an H-pile due to corrosion of the steel may be a concern for engineers, due to the presence of sulphate in the lacustrine clay layer and the potentially saline ground water. This concern may be alleviated by a study presented by Bowles (1997) on the effect of corrosion on steel piles. The study suggested that the corrosion of steel piles driven in natural soil deposits is not great enough to affect the strength of the piles significantly due to low oxygen concentration present in the undisturbed soils. However, it would be wise to take into account the potential for corrosion and the subsequent reduction of cross-section by using larger H-pile sections in the design. An increase of 1.5 mm in H-pile thickness is typical (CFEM 2006). Buried steel piles can also be protected from corrosion by using cathodic protection (Tomlinson and Woodward 2008).

Currents in the ground created by electric cars were believed to accelerate the rate of corrosion; however, since electric cars are no longer in service in Winnipeg, the author is of the opinion that accelerated corrosion of a steel H-pile due electrical currents should not be a concern today.

2.2 Effects of Pile Driving on Long Term Capacity

It should be noted that the pile analyses completed in this thesis are based on the static resistance to driving. The PDA test and CAPWAP analysis cannot predict long-term pile capacity, since each blow is instantaneous and does not represent sustained loading.

Pile driving disturbs the surrounding soil laterally and vertically, changing the soil's *in-situ* state (Craig 2004). A pile driven in clay causes remoulding of the clay and an increase in pore water pressure, resulting in a temporary reduction of the short-term shear strength (Hussein et al. 1993), and thus the pile's resistance. Coarse-grained soils experience changes in the level of compaction during driving. Loose coarse-grained soils densify and very dense soils dilate resulting in negative pore water pressures and a temporary increase in short-term pile resistance (Peck 1974).

An increase in pile resistance over time is referred to as pile set-up, which dictates the long term behavior of the pile (Hussein et al. 1993). A set-up factor between 1 and 6 has been documented by Bullock (2008) in a range of soil types with the highest set-up occurring in clay and the lowest in sands and gravels. Pile set-up primarily occurs along the pile shaft with insignificant contribution to end bearing resistance (Bullock 2008). Lukas and Bushell (1989) indicate that the majority of set-up gains occur within the first 10 days after the initial drive. Lukas and Bushell (1989) presented data of steel pipe piles and H-piles driven in stiff clay that show increases in capacity up to 25 percent between 10 and 32 days after initial drive.

A decrease in pile resistance is referred to as relaxation. This phenomenon may occur when negative pore water pressures build up in very dense sands and gravels that underwent dilation during driving, resulting in temporarily higher effective stresses than expected in long-term conditions (Hussein et al. 1993). Pile relaxation may also occur in weak, foliated sedimentary rocks such as shale as driving stresses relax due to creep (Bullock 2008). Soil relaxation may also occur in stiff heavily over consolidated clays. Soil relaxation is not as common as soil set-up and may occur along the pile shaft and at the pile toe.

The potential for pile set-up and relaxation should be addressed during construction to ensure the pile capacity meets specifications. Pile Driving Analyzer testing and CAPWAP analysis should be performed under beginning of re-drive (BOR) conditions, after sufficient time has passed for set-up or soil relaxation to occur. A case where soil set-up is expected may not be detrimental to a project as the pile design would likely be conservative. However, if soil relaxation is a concern the actual pile capacity may lessen to a value lower than the design capacity which may be problematic.

The effects of pile set-up and relaxation on the Q_{ult} should be considered during the construction stage. These phenomena are easily assessed by comparing EOID and BOR records from the PDA test and CAPWAP analysis (Goble et al. 1980). Comparison of the EOID and BOR records will identify changes in capacity that may occur over time. The early blows of BOR testing conditions, up to a maximum of about 25, should indicate any gains or losses. Beyond 25 blows, the pile reverts to initial drive conditions. Both EOID and BOR testing conditions were attained for this thesis.

2.3 Ultimate Capacity of a Single Pile

Ultimate capacity is a function of strength and integrity of the pile, the soil resistance, the pile-soil interaction characteristics, and the applied load (Goble and Hussein 1995). Ultimate capacity is governed by the weaker of the pile material and the bearing resistance of the strata. The estimation of Q_{ult} is based on the manner in which the load is transferred to the soil and underlying rock. Load-transfer analysis is often referred to as static analysis (Fellenius 2011). A pile resists an applied axial-compressive load in end bearing, shaft friction, or a combination of the two, as illustrated in Figure 2.3. Shaft friction capacity develops along the pile shaft through friction (or adhesion) at the pile-soil interface and end bearing capacity develops at the pile toe from the bearing strength of the underlying stratum (Fellenius 2012). The maximum amount of shaft friction capacity that a pile can develop is referred to as the $Q_{s\text{-}ult}$. The maximum amount of end bearing capacity that a pile can develop is referred to as the $Q_{b\text{-}ult}$. The summation of $Q_{s\text{-}ult}$ and $Q_{b\text{-}ult}$ equates to Q_{ult} and is expressed as,

$$Q_{ult} = Q_{s\text{-}ult} + Q_{b\text{-}ult} \tag{2.1}$$

The most accurate and desirable way to determine the Q_{ult} of a pile is by static load testing (Craig 2004). However, static load testing is usually not an option and other approaches to resistance analysis are required. According to Likins and Rausche (2004), CAPWAP analysis has shown excellent correlation to SLTs. Therefore, CAPWAP may be considered as reliable as a SLT, which is the opinion of the author. Many theoretical

RAMs are also available to estimate the Q_{ult} of piles. They are based on soil mechanics and empirical methods established by testing, observation and experience (Tomlinson and Woodward 2008).

2.3.1 Effects of Strata on Resistance Distribution

Piles installed through a weak stratum and seated on a hard stratum (*e.g.* clay overlying rock) will transfer practically the entire load to the pile toe, resulting in an end bearing pile (Peck et al. 1974). Load applied to a pile seated in a strong stratum overlying a weak stratum (*e.g.* stiff clay overlying soft clay) will be distributed along the shaft and resist practically the entire load in shaft friction, resulting in a friction pile (Peck et al. 1974). Piles installed in soils with a high affinity to friction overlying a soil with comparable resistance in end bearing may distribute the load along the shaft and the toe relatively equally. This type of pile can be described as a partial friction pile.

2.3.2 Effects of Applied loading on Resistance Distribution

Displacement of the pile must take place to mobilize soil resistance. This can be shown by a typical load-displacement curve of a load tested-pile as depicted in Figure 2.4. Progressive stages of loading and mobilization of shaft friction and end bearing resistance, from the onset of loading to failure are described by Chellis (1961), Peck et al. (1974), Bowles (1997), and Tomlinson and Woodward (2008):

The pile is elastically compressed at the onset of loading, stressing the surrounding soil and initiating mobilization of shaft friction resistance. The amount of elastic compression is dependent on the stiffness of the pile material relative to the surrounding strata. The resistance is distributed in the upper portion of the shaft at this stage. The soil initially resists loading in the elastic zone (in terms of the stress-strain relationship) until the ultimate elastic strain is reached. Loading beyond the elastic zone causes plastic deformation of the soil until it fails and the pile displaces, transferring the load down the shaft. This is referred to as partial slip, which can be defined as the summed difference in shaft strain (elastic compression) and the soil strain at any point along the pile shaft. Partial slip results in further elastic compression and additional stressing of the soil along the pile, which continues until the load reaches the pile toe. The entire pile shaft is engaged in frictional resistance to the applied load when full slip occurs at the pile-soil interface. At this point, additional loading will initiate mobilization of the end bearing resistance. The pile shaft will resist additional loading in unison with end bearing resistance until $Q_{s\text{-}ult}$ is reached. Loading beyond the $Q_{s\text{-}ult}$ will be resisted in end bearing and the shaft frictional resistance is conserved (strain softening may occur to the point at which a post peak value is conserved). The end bearing capacity continues to increase with applied loading until it is fully mobilized and the $Q_{b\text{-}ult}$ and Q_{ult} are reached. Loading beyond this point would result in the failure or excessive displacement (settlement) of the pile.

A service load is not intended to mobilize any end bearing capacity in the case of a friction pile and is expected to be resisted entirely in shaft friction. The service load is not expected to develop any shaft friction in the case of an end bearing pile and is expected to be transferred to the toe and resisted entirely in end bearing. However, shaft friction capacity must mobilize in the pile for end bearing capacity to develop, as described above. A long or large diameter pile will likely develop significant shaft friction capacity even if it is designed as an end bearing pile. Therefore, shaft friction capacity cannot be discounted for long or large diameter piles.

2.3.3 Mobilizing Displacements

The ultimate shaft friction capacity is mobilized at a displacement of about 0.3 to 1 percent of the pile diameter according to Tomlinson and Woodward (2008). Fellenius (2012) asserts that $Q_{s\text{-}ult}$ is achieved within a few millimetres of relative displacement and rarely more than 10 mm. The $Q_{b\text{-}ult}$ is achieved at a displacement of about 10 to 20 percent of the pile diameter (Tomlinson and Woodward 2008). Fellenius (2012) suggests that $Q_{b\text{-}ult}$ is typically mobilized at toe displacements less than 10 mm, and seldom greater. Likins and Rausche (2012) suggest that low-displacement piles driven to bedrock and soil require at least 1 and 2.5 mm of displacement per blow, respectively, to mobilize the $Q_{b\text{-}ult}$.

2.4 Static Analysis of Pile Capacity

The capacity of a pile is entirely dependent on the soil or rock in which it is seated (Peck et al. 1974). Static analysis estimates pile capacity from soil and rock strength properties attained by *in-situ* tests and laboratory testing (Likins et al. 2012). The preferred methods of static resistance analysis depend on the data obtained in a given locale (Bowles 1997); thus, the static analyses presented herein are focused on common testing procedures conducted for piles in the lacustrine clay, till, and karst bedrock underlying Winnipeg. The $Q_{s\text{-}ult}$ and $Q_{b\text{-}ult}$ are analysed separately in static analysis.

The capacity of a pile in clay is often related to the s_u and the adhesion between the pile and soil (Peck et al. 1974). Since the lacustrine clay in Winnipeg is quite homogenous and the s_u is practical to measure, RAMs based on these parameters seem quite rational. This total stress analysis approach is a conservative short-term analysis that represents initial loading conditions during construction.

The capacity of a pile in non-cohesive soil such as till is commonly determined by means of *in-situ* testing. The SPT is a practical and inexpensive *in-situ* test performed in the silt matrix-dominant till in Winnipeg. *"The standard penetration test (SPT) is a subjective and highly variable test. The test and the N-index have substantial qualitative value, but should be used only very cautiously for quantitative analysis"* (Fellenius 2006, p.7-14). The heterogeneity and variability of till makes it challenging to determine the quality of the stratum (Weltman and Healy 1978). Provided the heterogeneity of a site is not excessive, the SPT is performed correctly, and good judgment is exercised, the SPT N-value may be related to the *in-situ* shear strength of the till (Weltman and Healy 1978). Since the till in Winnipeg is generally considered silt matrix-dominant and, according to Weltman and Healy (1978), the engineering properties of silt matrix-dominant till are almost completely contributed by the matrix material, then the SPT N-values in the silt matrix can be considered as an indicator of frictional resistance. Judgement and experience is of the utmost importance when SPT N-values are used in design, since poor execution and misinterpretation might lead to erroneous results.

Rock masses are complex and consist of a wide range of engineering properties and behaviors (Goodman 1980). Bedrock consists of discontinuities such as joints, seams, faults and bedding planes (Goodman 1980). Karst formations have the potential for the presence of secondary geological features such as solution cavities and enlarged joints that could result in design problems (Kulhawy and Prakoso 2007). The Q_{b-ult} of piles driven into bedrock is difficult to estimate and analysis must consider the condition of both intact rock and the discontinuity characteristics.

"The bearing capacity of foundations on rock masses is complex because it is usually a function of both the intact rock material and the rock discontinuities" (Prakoso and Kulhawy 2005, para. 5). The end bearing failure mode of a pile in bedrock depends on the strength of the rock mass and the physical characteristics and frequency of fractures and joints. End bearing failure may occur by uniaxial compression, general shear, splitting, flexure, or puncture as described in Sowers (1979). However, many of these failure mechanisms are not applicable to karst conditions, or their parameters are not practical to measure, such as rock cohesion and friction angle for the general shear case. The author considers that failure related to compression is appropriate and practical for analyzing discontinuous karst bedrock in Winnipeg. Kulhawy and Carter (1992) suggest that the end bearing resistance of a rock mass under compression can be simply related to its q_u. Attaining rock core samples during the geotechnical investigation is routine and

performing UC tests on the core samples is quick, simple, and inexpensive. Therefore, relating the q_u of bedrock core samples to the $q_{b\text{-}ult}$ of H-piles seated in bedrock is justifiable and practical.

2.4.1 Ultimate Shaft Friction Capacity

The $Q_{s\text{-}ult}$ of a pile is likely the principal load-carrying mechanism in all but the softest of soils under service loads (Bowles 1997). The RAMs described in this section are used to estimate the $f_{s\text{-}ult}$ developed between the soil and pile surface. Common practice in Winnipeg is to assume constant $f_{s\text{-}ult}$ for design, although the resistance distribution along a pile is non-linear (Clark and Meyerhof 1972), as shown previously in Figure 2.3. Determination of the $f_{s\text{-}ult}$ is dependent on whether the soil is cohesive or non-cohesive. Piles in Winnipeg clays are often based on empirical total stress analysis methods like the α-method. Piles in till are usually based on empirical correlation to *in-situ* testing such as the SPT-method. The $Q_{s\text{-}ult}$ is calculated by multiplying the $f_{s\text{-}ult}$ by the surface area of the pile (A_s), as expressed in Equation 2.2,

$$Q_{s\text{-}ult} = f_{s\text{-}ult} A_s \tag{2.2}$$

Winnipeg design usually excludes the upper 3 m of the clay to account for weathering, frost action and potential shrinkage away from the pile due to seasonal moisture changes. “Soil-gapping” may be of concern during driving. It is caused by pile disturbance and “fluttering” of H-piles during driving (Tomlinson and Woodward 2008). “Soil-gapping” usually occurs in firm to stiff clays in the upper 1.2 to 1.8 m below the ground surface (Bowles 1997).

2.4.1.1 α-Method

Fellenius (2011) refers to the α-method as very limited in its application and the Canadian Foundation Engineering Manual (CFEM 2006) states that it has not proved to be reliable. However, the α-method is the preferred and most common method used in Winnipeg for estimating $f_{s\text{-}ult}$ of piles in clay. This method is favoured because it is practical, and measuring the input parameters is simple. The α-method is an empirically based, total stress analysis that estimates the short-term $f_{s\text{-}ult}$ of a pile in cohesive soil (CFEM 2006). This method provides a conservative estimate, since shaft frictional resistance of a pile in clay generally increases with time. The $f_{s\text{-}ult}$ is calculated by the following equation,

$$f_{s\text{-}ult} = \alpha s_u \tag{2.3}$$

The α-factor represents the adhesion between the pile and the soil. It is empirically related to s_u. Semple and Rigden (1984) present a correlation between the α-factor and s_u on the basis of 35 SLTs performed on offshore steel pipe piles driven into clay. The correlation is represented by the equations below,

$$\alpha = 1 - \left(\frac{S_u - 35}{90}\right) for\ 35\ kPa < S_u < 80\ kPa \tag{2.4.1}$$

$$\alpha = 0.5\ for\ S_u\ \geq 80\ kPa \tag{2.4.2}$$

$$\alpha = 1.0\ for\ S_u\ \leq 35\ kPa \tag{2.4.3}$$

It has been documented in other literature that the α-factors in very stiff clays can be as low as 0.2 (Tomlinson and Woodward 2008). Alpha-factors greater than 1 have been observed in very soft to firm clays (s_u < 50 kPa) due to remoulding and consolidation during driving (Bowles 1997). Winnipeg clays usually have a s_u ranging between 35 and 85 kPa (Baracos et al. 1983a).

2.4.1.2 β-Method

The beta-method (β-method) is an effective stress analysis approach used to estimate the $f_{s\text{-}ult}$ of pile (CFEM 2006). The β-method is based on the vertical effective stress of the soil along the pile shaft and a combined shaft resistance factor referred to as the β-coefficient. The β-method is expressed by the following equation,

$$f_{s\text{-}ult} = \beta\sigma_v' \tag{2.5}$$

Where, σ_v' is the vertical effective stress. The β-coefficient encompasses the angle of friction between the soil and the pile and the coefficient of lateral earth pressure of the soil. Analysis of shaft friction capacity based on an effective stress approach is rational (CFEM 2006). However, a total stress approach to assess shaft friction capacity for piles in clay is preferred in Winnipeg.

2.4.1.3 SPT-Method

The SPT is the most common method in Winnipeg for assessing the strength of the till. Meyerhof (1976) found, from the results of a number of SLTs, that the $f_{s\text{-}ult}$ of driven steel H-piles in non-cohesive soils is empirically and linearly related to the SPT N-value. The $f_{s\text{-}ult}$ of steel H-piles driven into non-cohesive soil is expressed as,

$$f_{s\text{-}ult} = N_{\mathrm{corr}} \leq 100\ kPa \tag{2.6}$$

Where, N_{corr} is N-value corrected for effective overburden pressure at depths along the embedded shaft. A limiting maximum value of the $f_{s\text{-}ult}$ is 100 kPa.

2.4.1.4 Piles in Bedrock

It is difficult to quantify the penetration depth and shaft frictional resistance of a pile driven into a karst formation that consists of discontinuities and layers of weak rock. Weak rock with brittle characteristics is presumed to shatter around a pile shaft during driving, significantly reducing or eliminating the shaft friction capacity (Tomlinson and Woodward 2008). No relevant literature was found by the author regarding the $f_{s\text{-}ult}$ of piles driven into bedrock. Dynamic methods may be more reliable than RAMs that are based on rock strength properties to estimate the $f_{s\text{-}ult}$ of driven piles in karst rock.

2.4.2 Ultimate End Bearing Capacity

The $q_{b\text{-}ult}$ of driven piles depends on the strength of the bearing stratum. The $q_{b\text{-}ult}$ is multiplied by the cross-sectional area at the pile toe to calculate the $Q_{b\text{-}ult}$ of a pile, as expressed in Equation 2.7,

$$Q_{b\text{-}ult} = q_{b\text{-}ult}A_b \tag{2.7}$$

Where, A_b is the cross-sectional area. The $q_{b\text{-}ult}$ of piles seated in till is commonly empirically correlated to the *in-situ* SPT N-value. Methods to calculate the $q_{b\text{-}ult}$ of piles seated in bedrock are based on theoretical and empirical relationships of the strength and discontinuities of the rock. Estimating the $q_{b\text{-}ult}$ of a pile based on the q_u of the bedrock is an attractive approach, because UC strength tests are inexpensive and quick to perform.

2.4.2.1 SPT-Method

The SPT N-value is directly and empirically related to the $q_{b\text{-}ult}$ of piles in non-cohesive soil (Meyerhof 1976). The $q_{b\text{-}ult}$ can be calculated from the basic empirical equation,

$$q_{b\text{-}ult} = CN_{corr} \tag{2.8}$$

Where, N_{corr} is the N-value corrected for effective overburden pressure, C is a coefficient equal to 0.3 for non-plastic silts and 0.4 for sands and gravels, which is reached at a pile embedment of 10 pile diameters. The value of N_{corr} is averaged within three pile diameters below the tip of the pile.

2.4.2.2 q_u-Method

Rowe and Armitage (1987) suggested that the $q_{b\text{-}ult}$ of rock containing vertical joints is directly related to the q_u of intact rock. The relationship takes the following form,

$$q_{b\text{-}ult} = N_{cr}Q_u \tag{2.9}$$

Where, N_{cr} is an empirical bearing capacity factor for rock. According to Peck et al. (1974), it is conservative to base the $q_{b\text{-}ult}$ of a rock mass on the q_u of the rock, considering the likelihood of tight joints and the influence of confining overburden stresses. Tomlinson and Woodward (2008) state that rock can exhibit $q_{b\text{-}ult}$ more than 10 times the q_u (N_{cr} = 10). A recent study by Prakoso and Kulhawy (2002), on the basis on 14 load tests at 9 different project sites in several rock types, recommends a $q_{b\text{-}ult}$ of rock 2.5 times (N_{cr} = 2.5) the q_u.

2.5 Dynamic Analysis of Pile Capacity

The more effort it takes to drive a pile into the ground the greater the bearing capacity. This is the basic principle of pile dynamics. In the early times of driven pile analysis it was thought that since piling deforms and fails the surrounding soil then measurements attained during driving could be used to predict capacity (Rausche et al. 1985). This belief led to the development of the simplest of the dynamic RAMs, the pile driving equation or dynamic formula.

Dynamic pile analysis has improved over time, becoming more mathematically complex and theoretically advanced. Several levels of dynamic RAMs exist in terms of accuracy and reliability; dynamic formulas are the oldest, simplest and crudest methods; wave equation analysis is more modern and complex; the Case method uses actual field measurements to assess pile capacity; the Case Pile Wave Analysis Program combines field measurements and WEA, making it the most advanced and reliable dynamic RAM. The CAPWAP method forms the basis of all the research conducted in this thesis and serves as the benchmark for comparison of the accuracy of the other RAMs. Section 2.5 focuses on the theory, practice, and accuracy of four dynamic RAMs commonly used to predict the Q_{ult} of a driven pile. It should be mentioned that Long et al. (2002) found no evidence that dynamic analysis of driven steel H-piles was different from other pile types.

2.5.1 Bearing Graph, Driveability, and the Pile Driving System

The geotechnical engineer should be proficient in understanding the bearing graph, pile driveability, the pile driving system, and testing conditions for accurate dynamic assessment of the bearing capacity and behavior of driven piles.

The bearing graph is a practical tool for estimating Q_{ult} and a corresponding refusal criterion. Figure 2.5 shows a typical bearing graph which illustrates the relationship between the Q_{ult} and penetration resistance. Hussein et al. (2003) assert that the purpose of a bearing graph is twofold: 1) for a specified pile capacity, the required penetration resistance can be verified and 2) for an observed penetration resistance the pile capacity can be estimated. Penetration resistance, or blow count, can be defined as the number of hammer blows required to drive a pile 25 mm into the ground. The pile capacity increases as the penetration resistance increases. The set is the inverse of the penetration resistance and is defined by the amount of pile penetration per hammer blow. The bearing graph may be in the form of Q_{ult} versus penetration resistance or set.

Bearing graphs are handy for quality assurance during construction at the time of driving. However, understanding the variables of a bearing graph is important. Low hammer energy and inaccurately observed blow counts make the bearing graph ineffectual (Hussein et al. 2003). Estimating the Q_{ult} using an observed set when the hammer is not developing sufficient energy could show to very generous results.

Driveability is defined as the ability to drive a pile to a required stratum. A driveability analysis is often performed to determine the amount of energy required to drive a pile to a required depth, estimate driving stresses, and to determine the final blow count (Rausche et al. 2000). The hammer must be able to deliver enough energy to the pile to overcome the soil resistance when hard layers are encountered. Meanwhile the pile material and size must be sufficient to withstand potentially destructive driving stresses.

The pile driving system consists of the pile driver (hammer), the pile, and the soil. The pile driver delivers energy to the pile; as the pile penetrates the ground it transfers the energy and resulting forces to the surrounding soil; the soil then strains to resist the transferred forces. The energy delivered to the pile must be sufficient to strain the soil beyond its elastic state to achieve plastic soil deformation, mobilize the ultimate soil resistance, and achieve a permanent set. The Q_{ult} cannot be discovered if the hammer energy is not sufficient to displace the pile enough to fully mobilize the soil resistance (Goble and Hussein 1995).

Common pile drivers used to drive steel H-piles in Winnipeg are single-acting internal combustion diesel hammers, illustrated in Figure 2.6a, and hydraulic hammers (external combustion hammers), shown in Figure 2.6b. The most important aspects of the pile driver system, for the purpose of resistance analysis, are the rated energy and efficiency. The efficiency is referred to as the percentage of the rated energy that is transferred to the pile head. Energy losses through the pile driving system occur through friction, elastic collisions, and gas compression (Likins and Rausche 1981). Tomlinson and Woodward (2008) give a broad range of hammer efficiencies from 20 to 80 percent for diesel hammers and 65 to 90 percent for hydraulic hammers. The author has observed efficiencies as high as 95 percent for hydraulic hammers, as confirmed by a PDA. A recommended hammer efficiency of 0.8 for internal combustion hammers and external combustion hammers is provided in GRLWEAP. Rausche and Klesney (2007) provide a full description of different types of pile drivers, how they work, and their efficiency.

2.5.2 Dynamic Formulas

The first documented use of dynamic formulas was in the United States as early as 1851 (Fellenius 2012). They have undergone many stages of evolution ever since. Dynamic formulas are attractive resources for predicting pile capacity because of their simplicity (Hussein and Goble 2004). Advances in pile driving technology combined with comprehensive research have led to improvement in the accuracy of some pile dynamic formulas and the demise of many. Some dynamic formulas predict pile capacity well in certain locales, but perform poorly in others; they have shown great variability and uncertainty in some cases and have been relatively accurate in others.

Dynamic formulas are considered semi-empirical since they are based on observed piling behavior and Newton's Law of rigid body impact mechanics. A common dynamic formula used today is the modified Gates formula which is a relatively recent dynamic formula that is based on the use of modern construction equipment and experience. The Q_{ult} is a function of the rated hammer energy (E), the hammer efficiency (H_E) and the blow count (N). The Q_{ult} is expressed in SI units as,

$$Q_{ult} = 6.7\sqrt{EH_E}\ \log_{10}(10N) - 445 \tag{2.10}$$

The modified Gates formula has been improved over time by correlating empirical coefficients to observed piling behavior. It has shown correlations of the estimated Q_{ult} to the measured Q_{ult} as determined by SLTs of 0.96 and 1.33 for EOID and BOR cases, respectively, with COVs of 0.41 and 0.48, respectively (Rausche et al. 1996). The modified Gates formula may be considered fairly accurate in terms of dynamic formulas; however it exhibits a significant amount of uncertainty and variability.

2.5.2.1 Shortcomings of Dynamic Formulas

Dynamic formulas are fundamentally incorrect. Hussein and Likins (1991) refer to dynamic formulas as incomplete, crude, and oversimplified. The main fundamental problems with dynamic formulas include the following:

- Applied theory incorrect - piles are treated as a single rigid body with no flexibility
- No differentiation between shaft friction and end bearing capacity
- No differentiation between static or dynamic soil resistances
- Pile length, size and stiffness are not considered
- Cannot evaluate long-term bearing capacity, creep behavior, or settlement

In addition to the general problems associated with dynamic formulas listed above, the modified Gates formula has an obvious and major error; when the hammer energy or the blow count is low, the formula produces a negative capacity, which does not make sense. Wave equation analysis addresses the shortcomings of the dynamic formulas.

2.5.3 WEA and the Smith Model

The following description of WEA and the Smith model (1960) is based on the author's readings of Fellenius (1984), Fellenius (1999), Goble et al. (1980), Likins (1981), Hussein and Likins (1991), Rausche et al. (1992), Rausche et al. (1994), Abou-matar et al. (1997), Rausche et al. (2000), Hussein et al. (2003), Hussein and Goble (2004), Rausche et al. (2004), and Rausche et al. (2010).

Piles behave as elastic rods when impacted by a hammer. Therefore, one-dimensional stress wave theory in elastic rods can be used to simulate and analyze the hammer, pile and soil into a combined interrelated system. Figure 2.7 illustrates wave propagation along a driven pile. The top of the pile is elastically compressed at initial impact of the hammer, creating a stress wave which travels down the pile at a constant wave speed. The wave attenuates due to soil resistance in a manner depending on the soil type, causing wave reflections. A net stress wave is reflected at the toe and returns to the top of the pile. Easy driving conditions result in tensile wave reflections with high velocity and low capacity; hard driving results in compressive wave reflections with a low velocity and high capacity. Pile penetration occurs when the driving force is greater than the total resistive forces of the soil.

The Smith model analyzes the travelling wave and estimates the penetration resistance and the corresponding Q_{ult} by way of a discrete closed-form numerical model consisting of a series of masses, springs, and dashpots, as shown in Figure 2.8. The hammer and pile components of each segment consist of a mass and a weightless linear-elastic spring of a constant stiffness. The static and dynamic soil components are represented by weightless linear-elastic-perfectly-plastic springs and linear-viscous dashpots, respectively, connected to each embedded pile mass. The soil components along the shaft represent shaft friction resistance; the soil components at the toe of the pile represent the end bearing resistance.

2.5.3.1 Smith Model Simulation

The hammer, pile, and soil properties, and the Q_{ult} and are predetermined to initiate the Smith model analysis. Wave simulation begins on the first mass segment by an assigning an impact velocity to the ram, which is calculated from the drop height (stroke) and hammer efficiency. The motion of the mass results in displacement of the connected spring creating a stress and a force. The displacement of a pile spring activates the soil resistance of an embedded segment producing static and dynamic reactionary forces resulting in a net force which accelerates (or decelerates) the pile mass. The result is a new velocity that is transferred to the next segment in the series. The process is repeated for all the segments in the model for one time step. Pile penetration occurs if the magnitude of the stress wave is sufficient to strain the soil beyond its elastic limits, resulting in plastic deformation and a permanent set.

The displacement (u), stress (C), and force (F) in the spring, soil resistance (R), and velocity (v) of the mass are calculated at each segment for each time interval using d´Alembert's general solution to the one-dimensional partial differential wave equations.

$$u(m,t) = u(m,t-\Delta t) + \Delta t\, v(m,t-\Delta t) \tag{2.11.1}$$

$$C(m,t) = u(m,t) - u(m+1,t) \tag{2.11.2}$$

$$F(m,t) = C(m,t)K(m) \tag{2.11.3}$$

$$R(m,t) = [u(m,t) - u'(m,t)]\, K_s(m)[1 + J_s(m)v(m,t-\Delta t)] \tag{2.11.4}$$

$$v(m,t) = v(m,t-\Delta t) + \frac{\Delta t}{M(m)}[F(m-1,t) + M(m)g - F(m,t)] \tag{2.11.5}$$

Where, m is the mass number, t is the time interval, Δt is the size of the time interval, K is the stiffness of the pile spring, J_s is the Smith damping factor, K_s is the stiffness of the soil spring, g is the gravitational acceleration, and M is the weight of pile mass.

The mass and spring stiffness of each segment are based on the pile dimensions and material properties. The permanent set is calculated by subtracting the weighted averages of the elastic displacement (quake) along the shaft and toe from the maximum toe displacement. A permanent set is the plastic deformation of the soil that remains after elastic rebound of the pile and soil has occurred.

2.5.3.2 Static and Dynamic Soil Models

Wave equation analysis includes a soil model which is absent from dynamic formulas. The soil model represents the total soil resistance to driving and consists of static and dynamic resistance. Figure 2.9 shows the two components. The static resistance of each segment in the model is represented by the linear-elastic-perfectly-plastic spring, which describes the relationship between capacity and displacement, and is expressed as,

$$R_S = k_s u \tag{2.12}$$

Where, R_S is the static resistance, k_s is the stiffness of the spring (soil), and u is the displacement of the spring (soil). The quake is the maximum elastic strain (displacement) that the soil will undergo before yielding, or before plastic strain occurs. The quake value must be reached during driving to mobilize the ultimate static capacity which is considered the Q_{ult} of the pile. Loading beyond the ultimate static capacity is resisted dynamically.

The dynamic resistance of each segment in the model is represented by linear-viscous dashpots. The behavior of the dashpots is described by the relationship between Smith damping (J_s) and pile velocity. Smith damping represents the energy absorption and the response of the soil to the rapid pile movement and is a function of soil grain size; fine grained soils absorb more energy and therefore have greater J_s values than coarse-grained soils. The static and dynamic resistance at each segment are related by the following equation,

$$R_D = R_S J_s v \tag{2.13}$$

Where, R_D is the dynamic resistance and v is the velocity of the mass. Table 2.1 provides J_s and quake values recommended by *Pile Dynamics, Inc*. (2011). The laborious calculations that are required to perform WEA are only practical with a computer. George Goble and Frank Rausche of *Pile Dynamics, Inc.* developed the first WEA program in 1976, called GRLWEAP, which made the rigorous calculations possible within a practical time period.

2.5.3.3 GRLWEAP

The GRL Wave Equation Analysis Program is a computer software program that performs WEA to evaluate the Q_{ult}, blow count and driveability of a pile. The program requires hammer, pile, and soil input variables to perform the analysis. A library consisting of recommended hammer efficiency values is built into the program. The soil model complexity is dictated by the user and its reliability is based on the knowledge gained by the geotechnical investigation or o by the engineer. The soil model input properties determine the J_s and quake values that represent the dynamic and static resistance. The GRL Wave Equation Analysis Program has shown good correlation of the estimated Q_{ult} to the measured Q_{ult} as determined by SLTs. Rausche et al. (2000) state that a 0.92 correlation with a COV equal to 0.31 was achieved in a correlation study based on effective and total stress analysis methods.

2.5.3.4 Shortcomings of WEA

Although WEA is an improvement to dynamic formulas, it has limitations. Some of the main shortcomings of WEA are listed below:

- WEA does not use actual force, velocity or energy measurements; reliability depends on the accuracy of the assumed input variables
- Soil is assumed to be fixed in space and does not move with pile
- WEA cannot evaluate long-term pile capacity, creep behavior, or settlement

The quality of soils information and judgment are vital in achieving meaningful WEA results. Local experience with hammer performance, soil properties and driving behavior will dictate the worth of a WEA in a particular locale. The PDA test and Case method eliminates the assumptions of WEA by integrating actual field measurements of the hammer performance and driving behavior into the dynamic analysis of piles.

2.5.4 PDA and the Case Method

The PDA and the Case method are described below as interpreted from the readings of Goble et al. (1980), Gravare et al. (1980), Likins (1981), Likins and Rausche (1981), Likins (1983), Rausche et al. (1985), Likins and Hussein (1988), Hussein and Likins

(1991), Hussein and Rausche (1991), Rausche et al. (1994), Goble and Hussein (1995), Goble and Likins (1996), Rausche (2000), PDI (2004), and Rausche et al. (2010).

Pile Dynamics, Inc. made the first PDA commercially available in 1972 after being developed in the 1960s and 1970s by a research team headed by George Goble at the Case Institute of Technology. The PDA test is an inexpensive, accurate, and non-destructive high strain dynamic load test that can be performed in a timely fashion (Rausche et al. 1994). The PDA uses the Case method to evaluate the efficiency of the driving system, monitor driving stresses, assess pile integrity, and provide an estimate of the Q_{ult} of a driven pile (Goble and Hussein 1995). Reusable strain gauges and accelerometers, attached to the pile, measure stress waves created from each hammer blow and transmit the information to the PDA, which processes the information in real-time. Figure 2.10 shows a typical blow trace recorded during the PDA test.

2.5.4.1 Case Method Capacity

The Case method uses the stress wave measurements and basic wave mechanics to estimate the Q_{ult} of a pile. The strain gauges measure the axial strain in the pile caused by the hammer impact. The force (F) at the gauges is calculated from the measured strain using Hooke's Law,

$$F = \varepsilon E A \tag{2.14}$$

Where, ε is the axial strain, E is the elastic modulus of the pile material, and A is the cross-sectional area of the pile. The accelerometers measure the acceleration of the generated stress wave, which represents the motion of the pile. The velocity of the wave

(V) is determined by integrating the acceleration of the stress wave over time and is expressed as,

$$V(t) = \int a(t)dt \tag{2.15}$$

Where, a is the acceleration of the stress wave and t is time. The force and velocity waves are related through an impedance factor which was developed from Hooke's law and particle velocity theory. The relationship between the force and velocity waves is expressed as,

$$F = vZ = v(\frac{EA}{c}) \tag{2.16}$$

Where, v is the pile particle velocity, Z is the impedance factor, E is the elastic modulus of the pile material, A is the cross-sectional area of the pile, and c is the wave speed. In the case of no soil resistance, the force and velocity waves are proportional; how the waves behave in the presence of soil relates to the $Q_{s\text{-}ult}$ and $Q_{b\text{-}ult}$ of the pile.

Stress wave reflections occur when soil resistance is encountered along the shaft. These reflections alter the velocity and force waves and disrupt their proportionality. The velocity wave decreases in magnitude and the force wave increases, causing separation between the two waves, as shown in Figures 2.11a to 2.11c. This separation defines the shaft frictional resistance. When the force and velocity waves reach the toe, their separation relates to the end bearing resistance. During easy driving conditions (low end bearing capacity), the force wave at the pile toe is low and the velocity of the wave is relatively high (Figure 2.11a). The velocity wave decreases and the force wave increases as the driving conditions become hard (Figure 2.11b-c) and the end bearing resistance increases. Figure 2.11a and 2.11b show small separation between the force and velocity waves, indicating low shaft friction resistance. Figure 2.11c illustrates a large separation between the waves, indicating high shaft resistance. Theoretically, the Q_{ult} is achieved when the magnitude of the velocity wave at the toe is zero.

The total resistance to driving (R_T) is the net sum of all resistances at the gauges as determined by superposition of the downward and upward travelling waves. The downward travelling force (F_d) and velocity (v_d) waves represent the input of the hammer; the upward travelling force (F_u) and velocity (v_u) waves represent the pile and soil response. The basic Case method equation is therefore expressed as,

$$R_T = \frac{1}{2}\left(F_d + Zv_d + F_u - Zv_u\right) \qquad (2.17)$$

To calculate the static resistance of the pile, the dynamic resistance (R_D) must be determined, which is expressed as,

$$R_D = J_c Z v_{toe} \qquad (2.18)$$

Where, J_c is the dimensionless Case damping factor (Table 2.2 provides recommended J_c values for different soil types), Z is the impedance factor, and v_{toe} is the pile velocity at the toe. The static resistance (R_S) is then calculated by combining Equations 2.17 and 2.18 to form the standard Case-Goble static resistance equation expressed as,

$$R_S = \frac{[(1-J_c)(F_d+Zv_d)]+[(1+J_c)(F_u-Zv_u)]}{2} \qquad (2.19)$$

The maximum static resistance of the pile determined during driving is considered the Q_{ult}.

The PDA and Case method provide reasonable estimates of the Q_{ult} considering they are performed in real-time. Measured pile capacities are generally within about 20 percent of the capacity measured by SLTs when values of J_c range between 0 and 2 (Rausche et al. 1985). The accuracy of the PDA test may be unbecoming with respect to WEA; however, the main benefit of the PDA is not its ability to assess the Q_{ult} but to assess pile integrity, driving stresses, and hammer performance.

2.5.4.2 Pile Integrity, Driving Stresses, and Hammer Performance

The greatest benefit of the PDA test is its ability to evaluate pile integrity, driving stresses, and hammer performance during pile installation. Understanding these elements of the piling system allows the contractor and the engineer to optimize construction and to solve problems based on facts, not assumptions. The PDA can determine if the hammer is transferring sufficient energy to penetrate the pile to the required depth without inducing stresses that may compromise the integrity of the pile.

The amount of transferred energy required must be sufficient enough to fully mobilize the soil resistance and penetrate the pile to the desired bearing strata. The energy transferred through the pile during driving is a function of the measured force and velocity waves. Transferred energy (E) with respect to time is related to the work performed during a blow and can be calculated by integrating the product of the force and velocity waves, and is expressed as,

$$E(t) = \int_0^t F(t)V(t)dt \tag{2.20}$$

Where, F is the measured force wave, V is the measured velocity wave, and t is the time.

Pile stresses are a function of the force in the pile and the cross-sectional area of the pile. Stress (σ) is calculated by the basic expression,

$$\sigma = \frac{F}{A} \tag{2.21}$$

Where, F is the force in the pile and A is the cross-sectional area of the pile. The magnitude of the stresses dictates the integrity of a pile. If the yield stress of the pile material is exceeded the pile will likely become damaged.

Pile integrity is determined by a change in Z (Equation 2.16) in a uniform pile, suggesting that the cross-sectional area has been compromised (reduced) by damage. The damage can be a result of inadequate splices, low quality pile material, or excessive stresses. The development of high stresses may become critical during cases of hard driving. The ability of a pile to resist high stresses is dependent on the size of the pile and the strength of the pile material.

2.5.4.3 Shortcomings of the Case Method

The Case method static capacity is measured using a predicted value of constant J_c. Accurate determination of Q_{ult} can result if local experience is available to select accurate J_c values. Otherwise, computed capacities are fruitless. The Case method lacks a detailed soil model that accurately encompasses all the variables associated with soil resistance; soil resistance cannot be defined by a single constant soil damping parameter. The Case method represents mobilized capacity, not necessarily the Q_{ult}. The Case method also cannot assess the long term pile capacity, creep effects, or settlement. The weaknesses of the PDA test and Case method are handled by further analysis of the data using CAPWAP.

2.5.5 CAPWAP

The following description of CAPWAP analysis is based on papers written by Goble et al. (1980), Likins (1981), Hussein and Likins (1991), Hussein and Rausche (1991), Rausche (1983), Rausche (1991), Rausche et al. (1994), Goble and Hussein (1995), and Rausche et al. (2010).

The Case Pile Wave Analysis Program was developed in the late 1960s at the Case Institute of Technology by the same Case Research Team that developed the Case method and the PDA. It is an iterative signal matching software program used to determine the ultimate capacities (*i.e.* Q_{ult}, $Q_{s\text{-}ult}$, and $Q_{b\text{-}ult}$), and load-displacement behavior of a pile at the time of driving. The Case Pile Wave Analysis Program avoids grand assumptions and provides an accurate measure of the static resistance to driving. It uses modified versions of the Smith model and measured force and velocity wave records generated during hammer impact, and collected by the PDA, to determine the pile and soil response.

The Case Pile Wave Analysis Program addresses the weaknesses of the Smith model and Case method. The Smith model assumptions for transferred energy, force, and velocity are eliminated by using actual measured readings of theses parameters by way of the accelerometers and strain gauges attached to the pile. The Case method is improved upon by integrating a sophisticated soil model into the analysis.

The CAPWAP signal-matching process includes harmonizing the measured stress waves to computed stress waves by adjusting and refining the soil model and its components. The total resistance, shaft resistance distribution, toe resistance, damping, quake, toe gap, and unloading are the main components that are adjusted to refine the soil model. The reliability of the soil model depends on the net differences between the computed and measured stress waves; the smaller the differences, the better the match quality, and the more accurate the calculated pile capacity. Experience and local knowledge of real static and dynamic soil properties increases the dependability the soil model.

2.5.5.1 Pile and Soil Models

The CAPWAP pile model (Figure 2.12) consists of a series of continuous linear-elastic segments of uniform length, cross-sectional area, specific weight, and elastic modulus. Figure 2.13 shows the soil model used in CAPWAP. The CAPWAP soil model is an extended version of the Smith mass-spring-dashpot model. The linear-elastic-perfectly-plastic springs, represent the static soil resistance and pile displacement and the linear-viscous dashpots represent the dynamic soil resistance. The CAPWAP soil model improves upon the Smith model with the inclusion of radiation damping for soils of high penetration resistance (high quake), toe gap, and soil plugging along the shaft. The static resistance can be determined by Equation 2.12 (Section 2.5.3.2). CAPWAP uses three damping soil models; The Smith model, a purely viscous model, and a combination of the two. The traditional Smith model (Equation 2.13 in Section 2.5.3.2) is commonly used for soils of relatively high static resistance. The purely viscous damping model is used for shaft resistance, because the Smith model often under dampens. The purely viscous model replaces the R_S term (static resistance) in Equation 2.13 with a constant ultimate static resistance; this assumes that changes in the total driving resistance are a result of changes in damping. The model that combines both the Smith model and the purely viscous damping model better simulates the resistance at the pile toe. The Smith model is used until the static resistance reaches the ultimate static resistance. At that point, the purely viscous model is used and the static resistance is replaced with a constant ultimate static resistance. The recommended values of J_s are the same for CAPWAP analysis as they are for WEA and should be greater than 0.1 sec/m and less than 1.3 sec/m along the pile shaft and at the pile toe.

2.5.5.2 Signal-Matching Process and CAPWAP Pile Capacity

Wave-up-matching is the most precise signal-matching procedure and is preferred over force and velocity wave-matching. The wave-up signal-matching consists of calculating a wave-up curve to match a measured wave-up curve. The measured wave-up (W_{UM}) curve is attained from a force and velocity measurement recorded during a PDA test as determined by basic wave mechanics and is expressed as,

$$W_{UM} = \frac{1}{2}[F_M - Zv_M] \tag{2.22}$$

Where, F_M is the measured force, Z is the impedance factor, and v_M is the measured velocity.

The CAPWAP signal-matching process is iterative. It begins by setting up the pile and soil models. The pile model is set-up by inputting the size, material properties, and embedment depth. The soil model is set up by inputting values of soil resistance and distribution, unloading effects, soil plug, toe gap, J_s, and quake. A complementary wave-up curve is calculated based on the pile and soil models. The soil resistance model is iteratively adjusted until the match quality between calculated wave-up curve and measured wave-up curve is satisfactory. The match quality can be referred to as the sum of the absolute values between computed and measured wave-up curves. After the match quality has been optimized, the calculated force (F_C) is determined by superposition of the measured wave-down curve and the computed wave-up curve values as expressed by the following equation,

$$F_C = W_{DM} + W_{UC} \tag{2.23}$$

Where, W_{UC} is the calculated wave-up curve and W_{DM} is the measured wave down curve which is calculated from a force and velocity measurement recorded during a PDA test and is expressed as,

$$W_{DM} = \frac{1}{2}[F_M + Zv_M] \tag{2.24}$$

Where, F_M is the measured force, Z is the impedance factor, and v_M is the measured velocity. Wave-up signal-matching provides the best resolution to the soil resistance, since it includes only upward travelling waves due to soil resistance and changes in pile impedance.

2.5.5.3 Load-Displacement Curve

Simulated top-of-pile and bottom-of-pile load-displacement curves are produced in CAPWAP based on the static soil resistance, quake, and elastic compression of the pile. Where, elastic compression is based on the elastic modulus, cross-sectional area, and length of the pile. Empirically based vertical shear versus vertical displacement curves (t-z curves) and vertical force versus vertical toe displacement curves (q-z curves) are used to represent the load-displacement behavior along the pile shaft and at the pile toe, respectively. These curves are defined by the soil resistance and quake value at each soil segment (PDI 2006). By setting the displacement as a boundary condition at the pile toe, the resistance and displacement at each pile segment are calculated. The displacement (d) is calculated by the following equation,

$$d(t) = \int \mathrm{V(t)}\ dt \tag{2.25}$$

Where, V is the velocity of the wave and t is the time. The CAPWAP-calculated load-displacement curve can then be simulated on the basis of the t-z (and q-z) curves, the quake value, and magnitude of the calculated displacement and resistances. The Q_{ult} represents the total applied failure load at the pile head. The $Q_{b\text{-}ult}$ represents the failure load at the pile toe.

2.5.5.3.1 Elastic Compression of a Pile

Many theoretical and empirical methods exist to calculate settlement under applied loads, or in contrast, the amount of load that can be resisted within a certain amount of settlement. One method for such calculations of pile settlement was developed by Vesic in 1977. Vesic's method (1977) indicates that the elastic compression of a pile is a function of the applied load and the pile properties of length, cross-sectional area, and the elastic modulus. The amount of elastic compression of long piles is important in settlement analysis as it can be significant.

2.5.5.4 Correlation of CAPWAP to Static Load Tests

Pile capacity based on CAPWAP analyses has shown excellent correlation to SLTs as reported by Likins and Rausche (2004). On the basis of 303 BOR case histories using a standard quality testing criteria that required: the elapsed time for testing after the EOID must be comparable between the PDA test and SLT, the energy must also be sufficient enough to produce a permanent set of 2.5 mm or greater to fully mobilize the Q_{ult}, and the SLT capacity analysis is based on the Davisson failure criterion. The correlation results showed an average ratio of CAPWAP-calculated Q_{ult} to the Q_{ult} measured by SLTs of 0.98 with a COV of 0.169. The data suggests that CAPWAP analysis provides reliable and slightly conservative BOR results.

The Case Pile Wave Analysis Program is the gold-standard technology for pile capacity testing in Winnipeg and is superior to the SLT in the opinion of the author. The correlation to SLTs described by Rausche et al. (2004) proves its reliability. The cost and time required to perform a PDA test with CAPWAP analysis is miniscule when compared to SLTs. The PDA test is non-destructive and can be conducted to design the piled foundation or to confirm pile capacity on production piles. Analyses conducted using CAPWAP is undervalued by the 2010 version of the NBCC and should be reconsidered using higher recommended RFs. Engineers should consider PDA testing and CAPWAP analysis in the design phase as it will result in more cost-effective and reliable pile foundations.

2.5.5.5 Shortcomings of CAPWAP

Like the aforementioned dynamic RAMs, CAPWAP has deficiencies. The principal limitations of CAPWAP analysis are that it cannot evaluate long-term pile capacity or creep behavior; and CAPWAP merely represents mobilized capacity, not necessarily the Q_{ult}. It is important to realize that the CAPWAP analysis will be conservative if the soil resistance is not fully mobilized due to a high blow count or quake value. High penetration resistances may require adjustments to the production piling criteria to ensure that Q_{ult} is fully mobilized. In some cases, it may not be possible to fully mobilize the soil resistance if the appropriate hammer is not used. Piling experience and understanding of pile and soil behavior under dynamic loading is important to achieve a soil model that is representative of the actual soil conditions present at the site of the tested pile.

2.6 Chapter 2 Discussion

Application of H-piles is increasing in Winnipeg and the understanding of how they behave is critical. The information provided in this chapter is important to understand for proper assessment of the load-displacement behavior and capacity of H-piles. The RAMs presented in this chapter are practical methods for estimating the Q_{ult} of an H-pile for the purpose of ULS design. The static RAMs provide the basis for evaluation of Q_{ult}; whilst the dynamic RAMs also provide an estimate of Q_{ult} but with the added benefit of estimating the penetration resistance required to achieve the estimated Q_{ult}. The load-displacement curves produced by the CAPWAP allows for settlement analysis for the purpose of SLS design.

(Tomlinson and Woodward 2008), suggest that to achieve the maximum potential bearing capacity it is desirable to drive the H-pile in conjunction with a PDA. All projects that include the use of driven steel H-piles should incorporate PDA testing with CAPWAP analysis due to the current lack of knowledge of this pile type. Although this thesis aims to reduce uncertainty in the bearing capacity and behavior of H-piles, variability in ground conditions is always present; therefore, capacity should be confirmed with PDA and CAPWAP technology, when possible.

Table 2.1: Recommended Smith damping and quake values (PDI 2011).

Soil Type	Smith Damping (J_s) (sec/m)		Quake (mm)	
	Shaft	Toe	Shaft	Toe
Clay	0.65	0.5	2.5	5.0
Silt	$0.16 < J_s < 0.65$	0.5	2.5	2.5
Sand	0.16	0.5	2.5	2.5
Rock	0.5	0.5	1.0	1.0

Table 2.2: Recommended Case damping values (PDI 1998).

Soil Type	Case Damping (J_c) (unit-less)
Clay	0.7 – 1.0
Silt	0.5 – 0.7
Sand	0.4 – 0.5
Gravel	0.3 – 0.4

Table 2.3: Recommended values of CAPWAP-based soil model variables (Rausche et al. 2010).

Name	Symbol	Unit	Shaft, min	Shaft, max	Toe, min	Toe, max
Quake	q	mm	1	7.5	1	Max.displacemt-g
Unloading Quake	q_u/q	1	0.3	1.0	0.3	1.0
Neg. ult. Resistance	R_{nu}/R_u	1	0.0	1.0	0.0	0.0
Gap	g	mm	0.0	0.0	0.0	Max.displacemt-q
Reloading	R_L/R_u	1	-1.0	1.0	0.0	1.0
Damping Option		1	0	2	0	2
Damping Factor	j	s/m	0.04	1.4	0.04	1.4

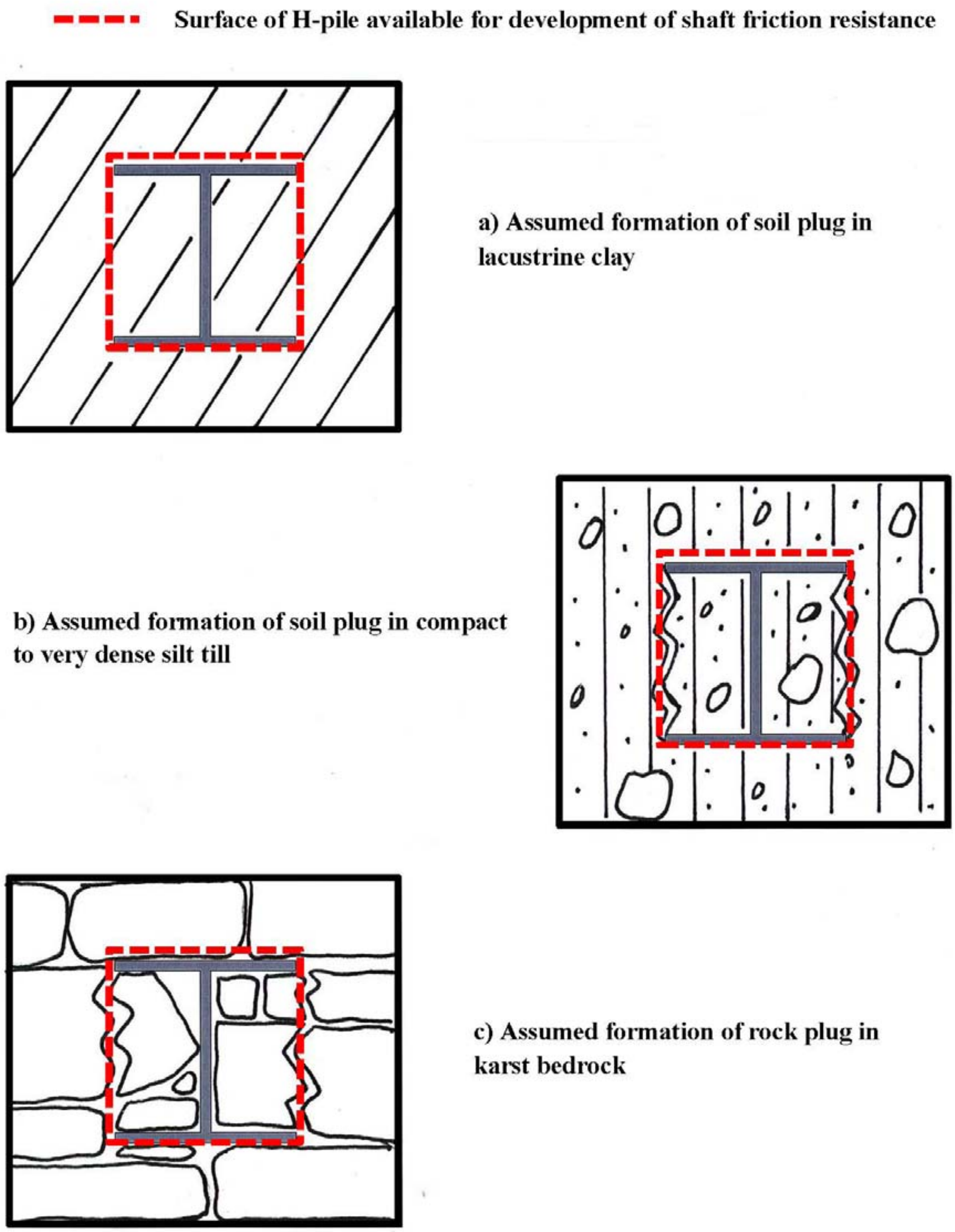


Figure 2.1: Plugging in lacustrine clay, till and karst bedrock.

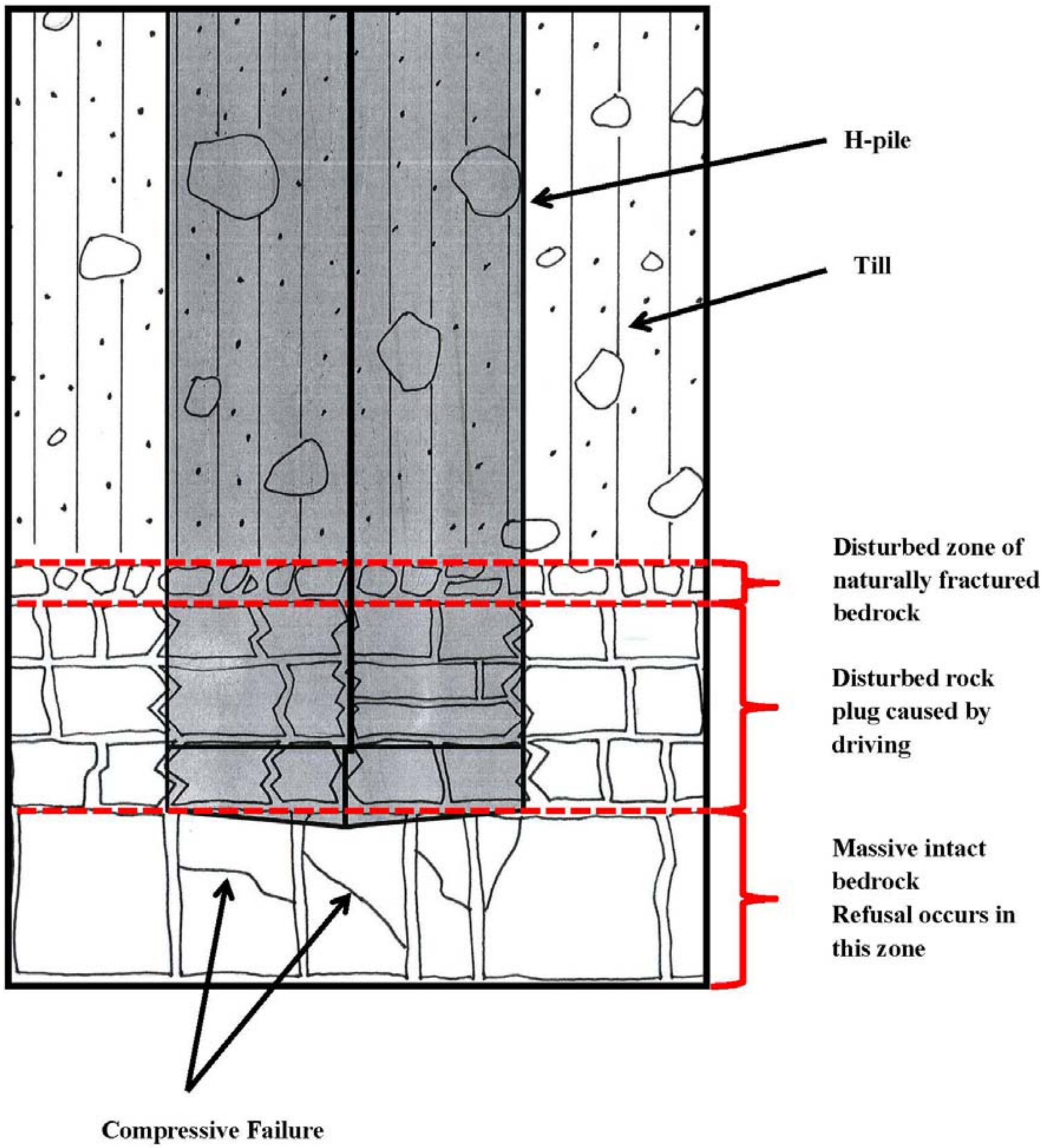


Figure 2.2: Formation of a rock plug at the pile toe at refusal in massive bedrock. Compressive failure of the rock at the pile toe.

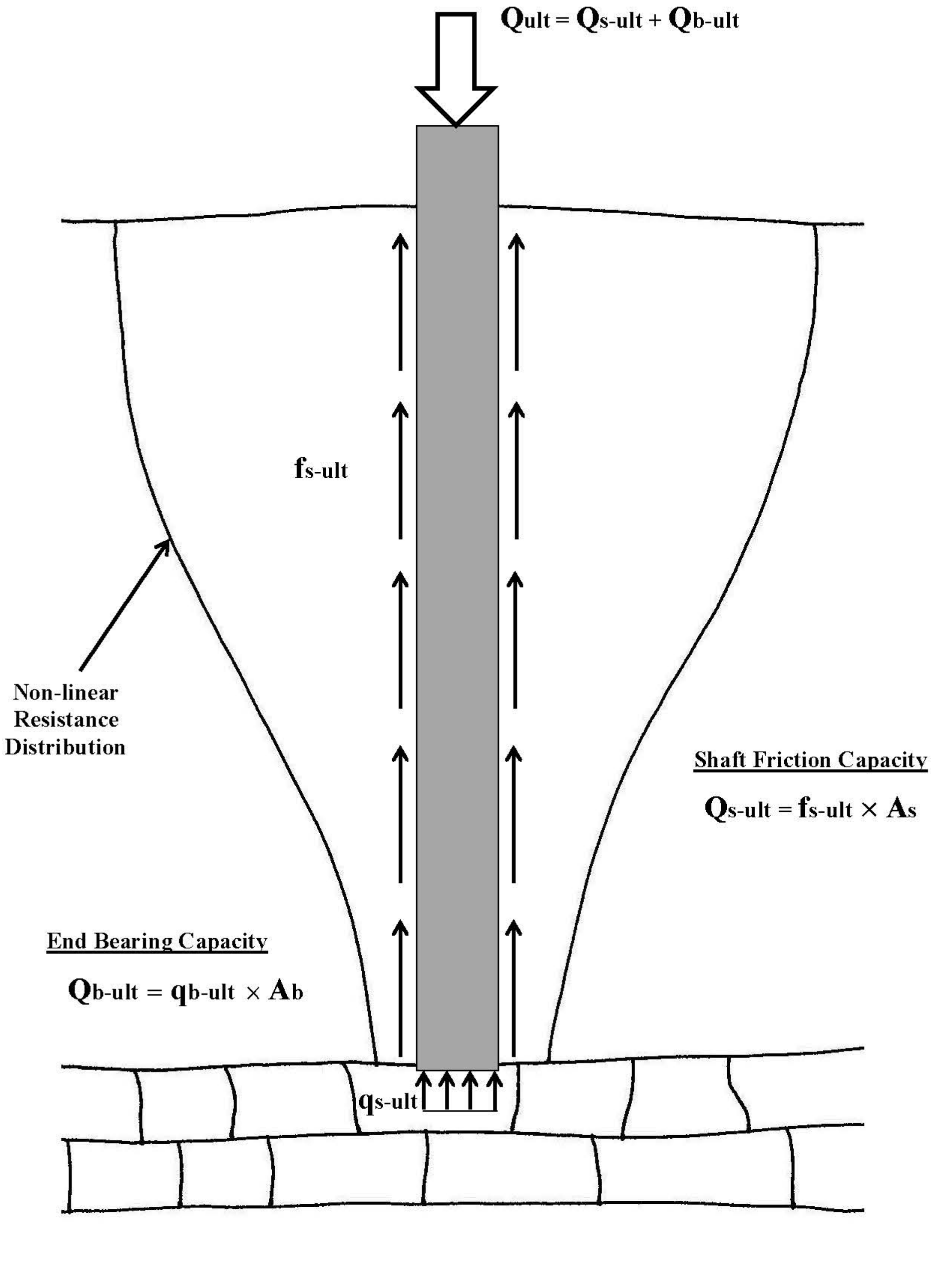


Figure 2.3: Resistance distribution of a pile an under an applied axial-compressive load. Based on Fellenius (2012).

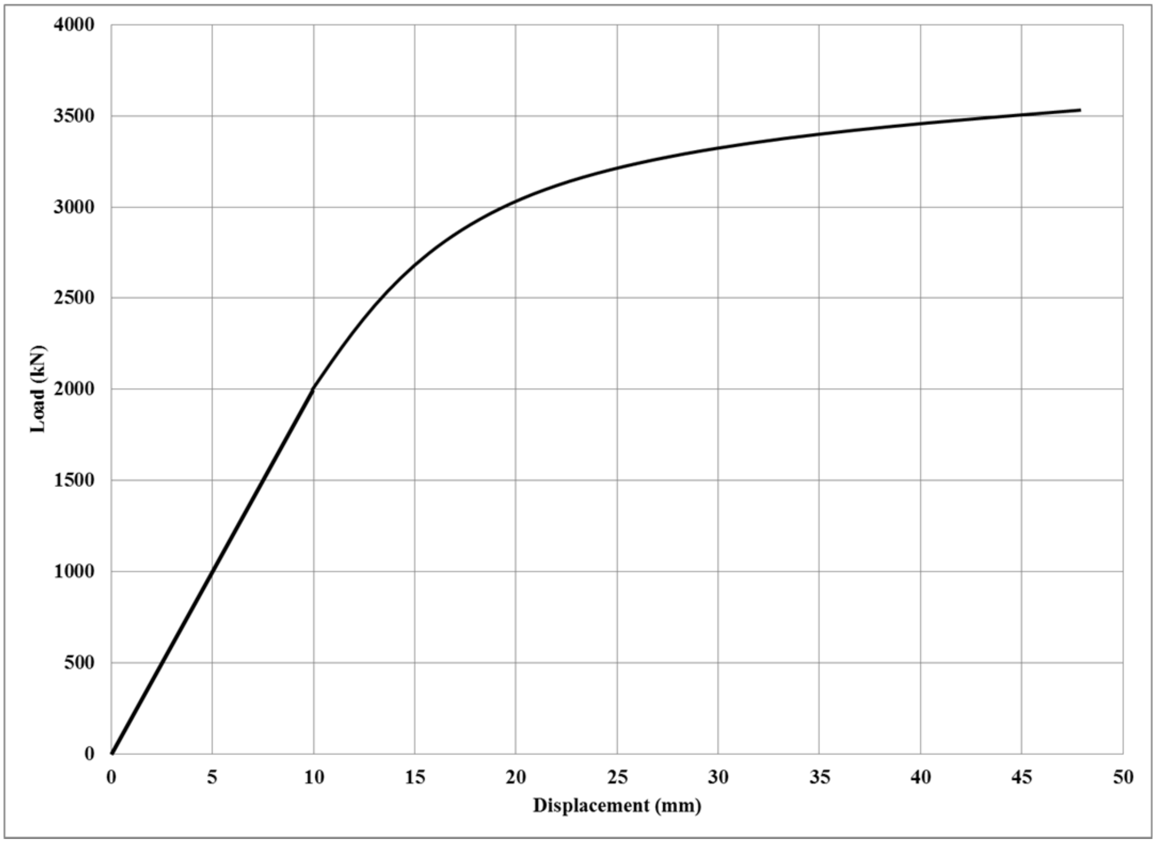


Figure 2.4: Typical load-displacement curve.

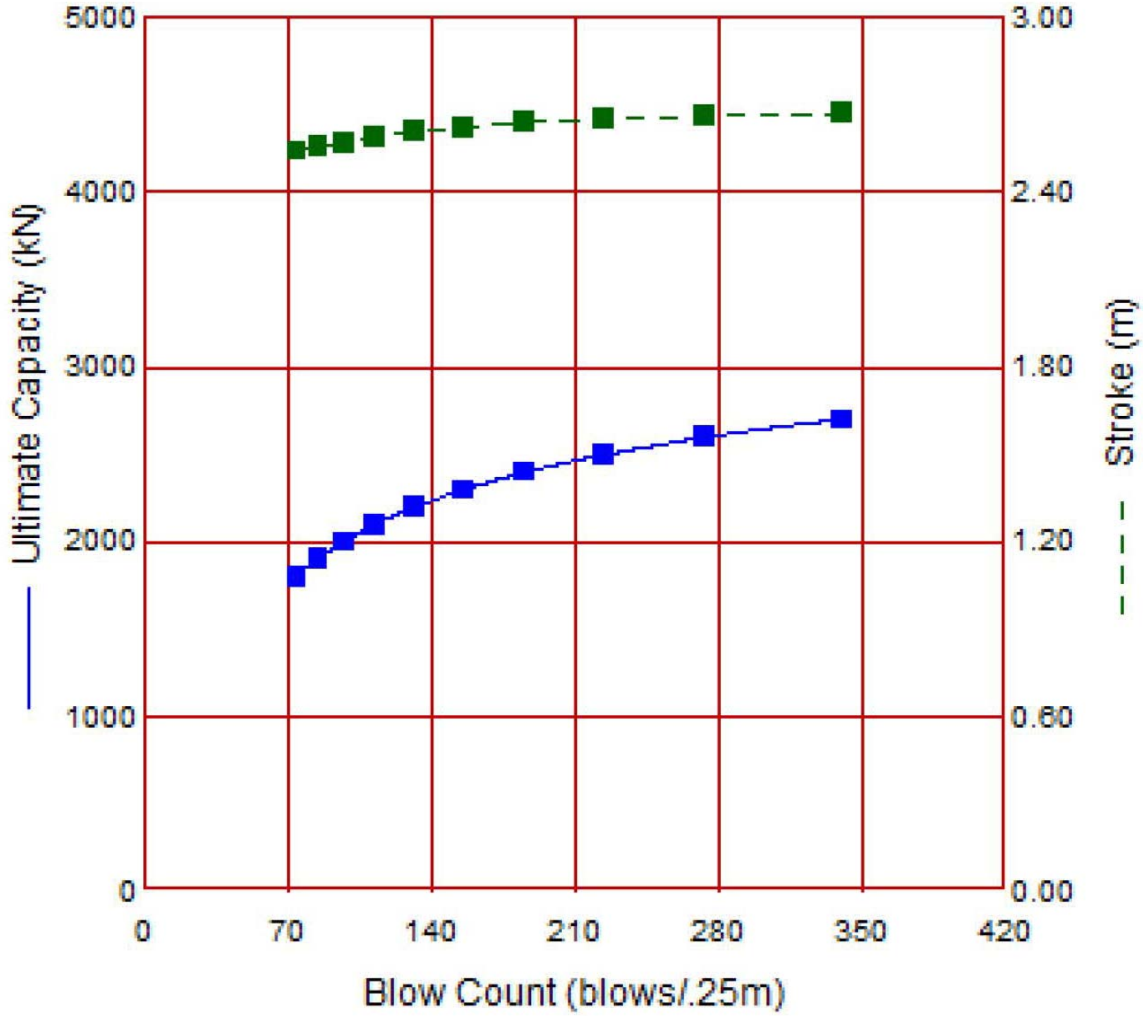


Figure 2.5: Typical bearing graph created in GRLWEAP (2010).

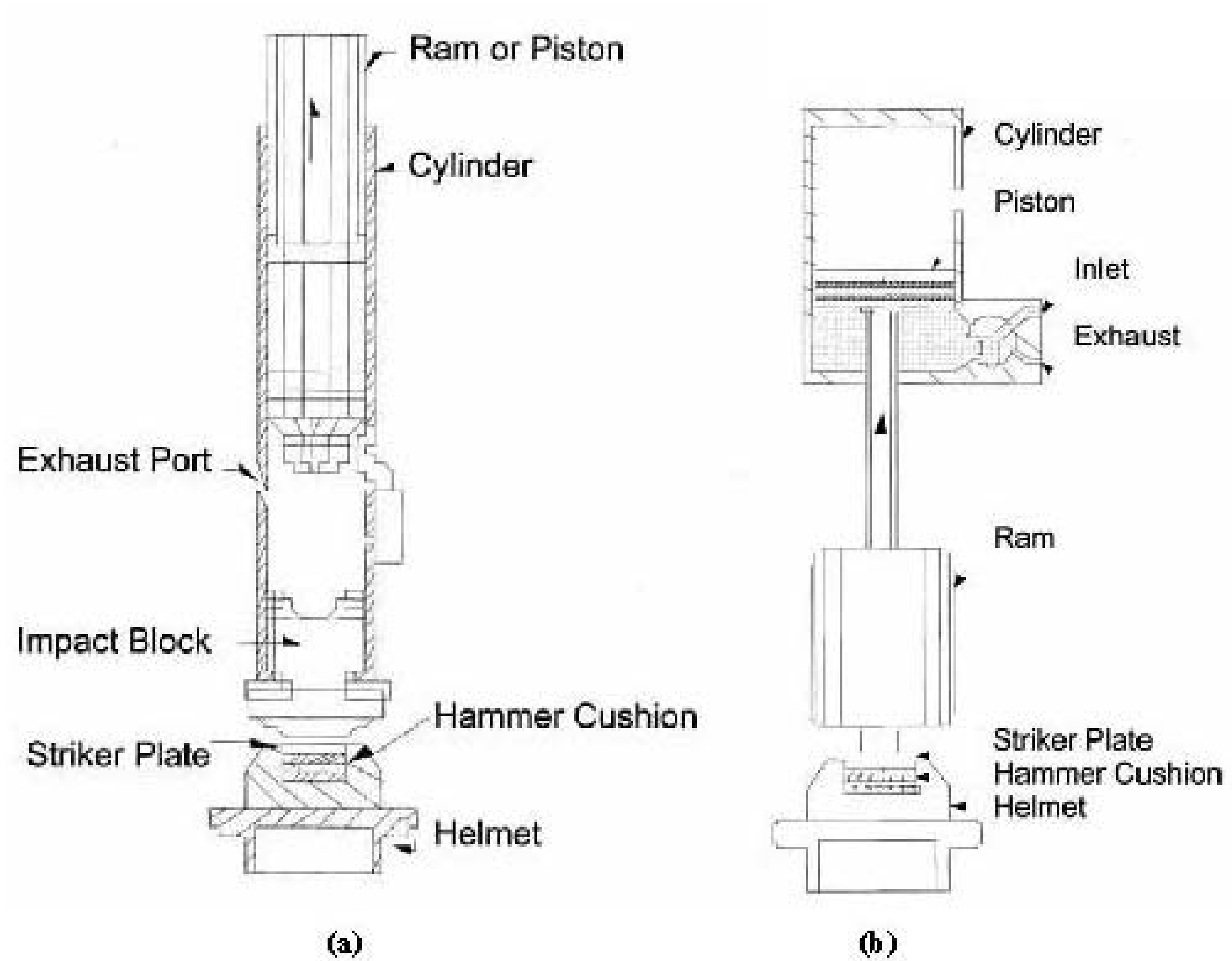


Figure 2.6: (a) Single-acting open-ended diesel internal-combustion hammer, (b) External-combustion hammer (Rausche and Klesney 2007). Reprinted with permission from Dr. Frank Rausche (PDI).

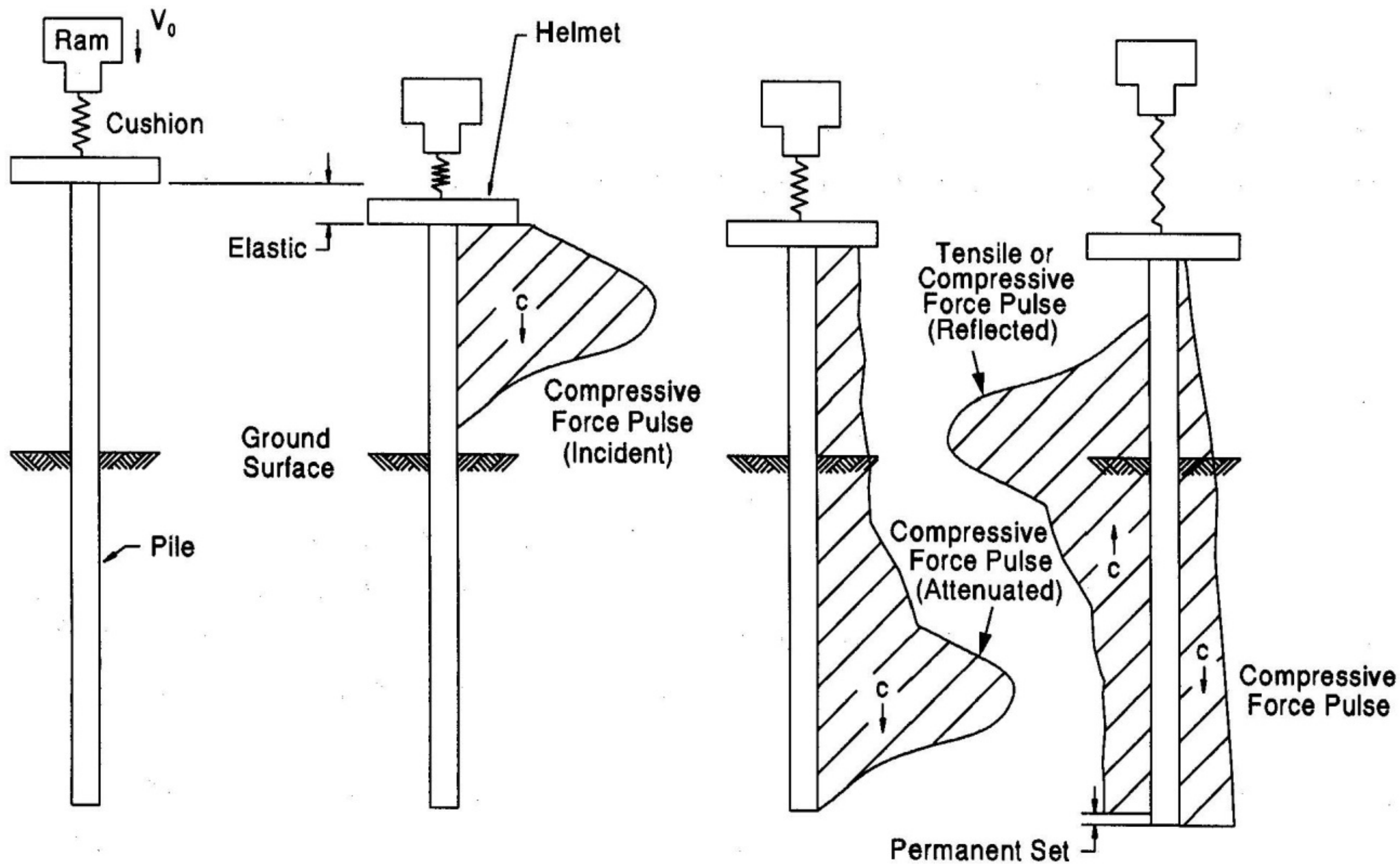


Figure 2.7: Wave propagation along a driven pile (FHWA 2006). Reprinted with permission of the U.S. Department of Transportation - Federal Highway Administration.

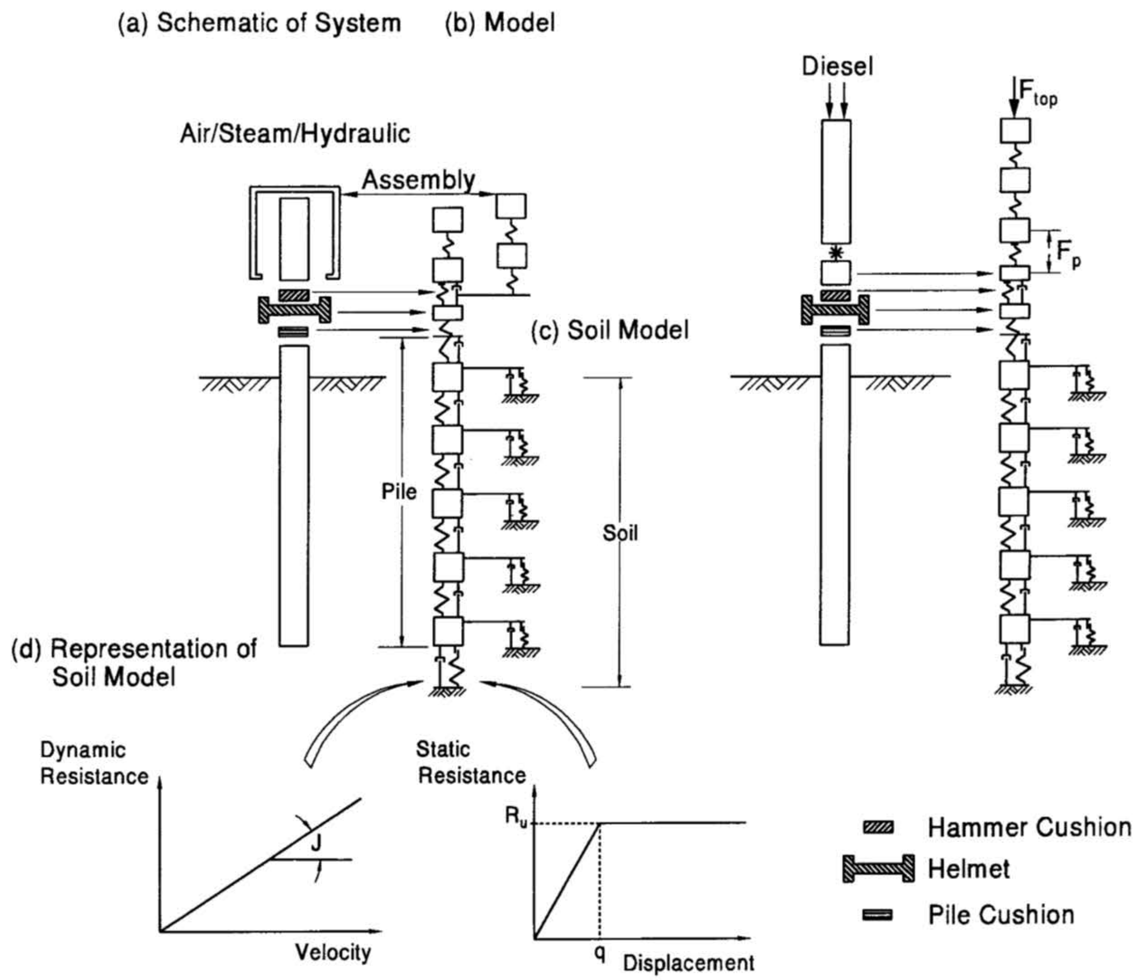


Figure 2.8: Smith mass-spring-dashpot Model (Rausche et al. 2004). Reprinted with permission from Dr. Frank Rausche (PDI) and the U.S. Department of Transportation - Federal Highway Administration.

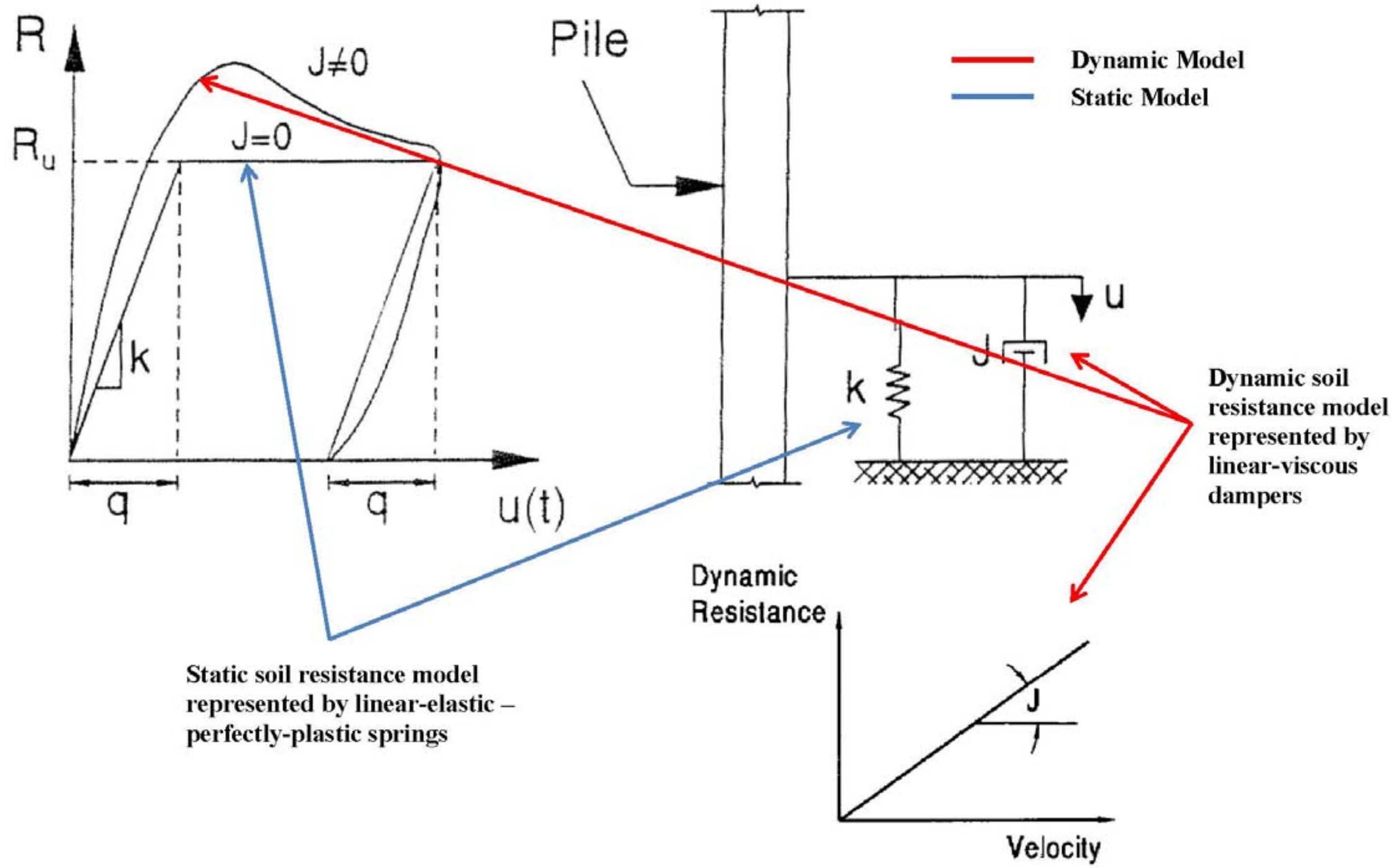


Figure 2.9: Smith Soil Model (Abou-matar et al. 1997). Reprinted with permission from Dr. Frank Rausche (PDI).

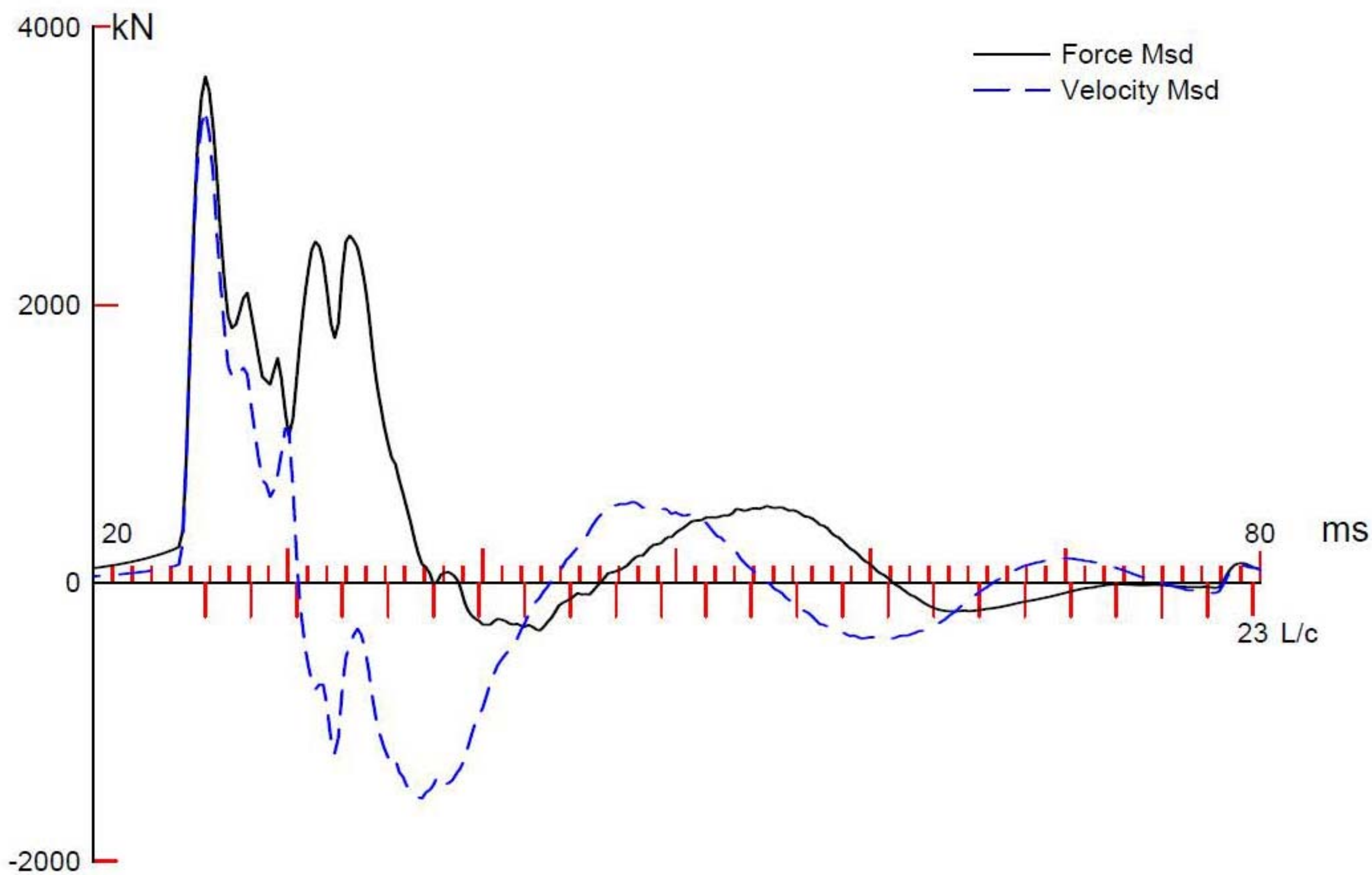
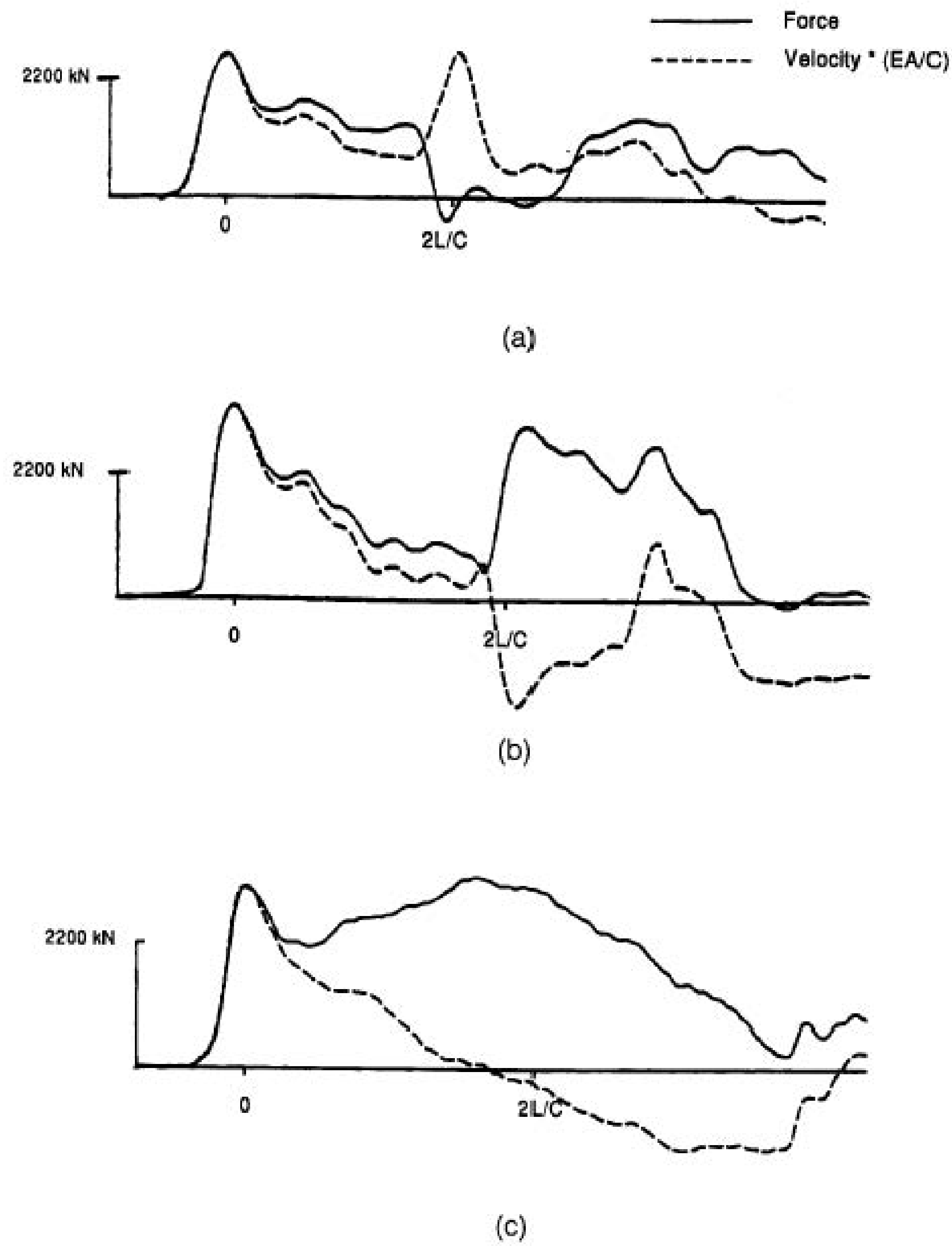


Figure 2.10: Typical force and velocity blow trace from a PDA test.



Figures 2.11a-c: Varying cases of shaft and end bearing soil resistance. (a) Low shaft and end bearing resistance (easy driving conditions). (b) Low shaft resistance, high end bearing resistance (hard driving conditions). (c) High shaft and end bearing resistance (hard driving conditions) (FHWA 2006). Reprinted with permission of the U.S. Department of Transportation - Federal Highway Administration.

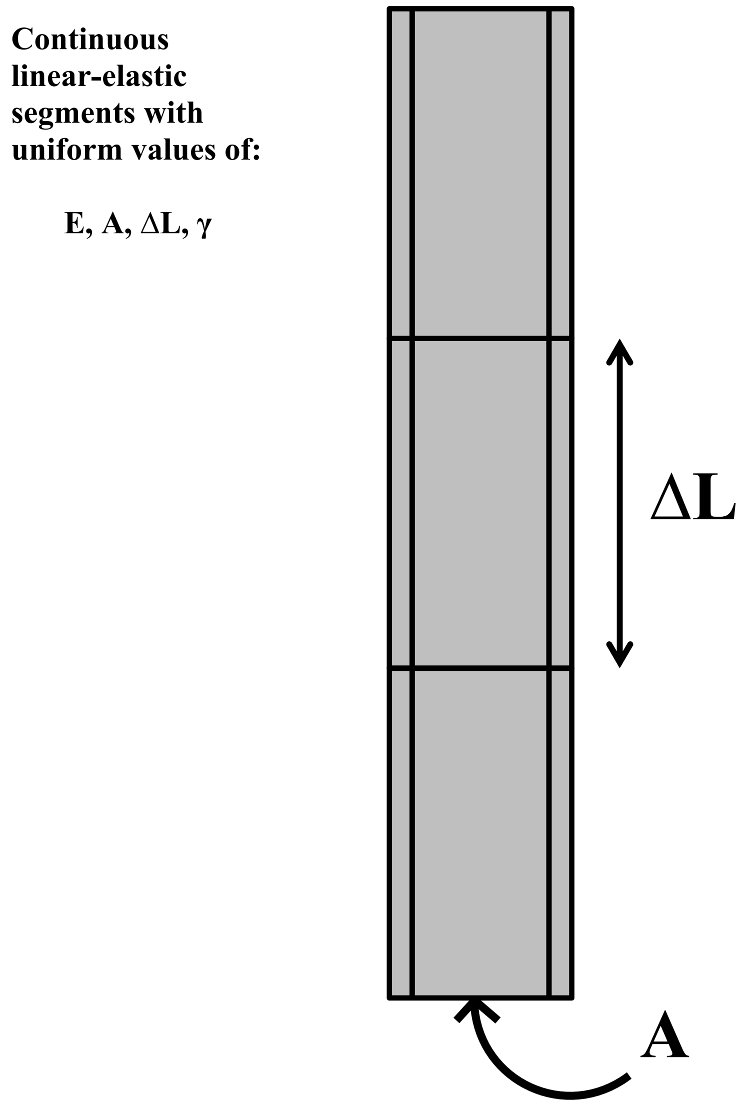


Figure 2.12: The CAPWAP pile model. Based on Rausche et al. (2010).

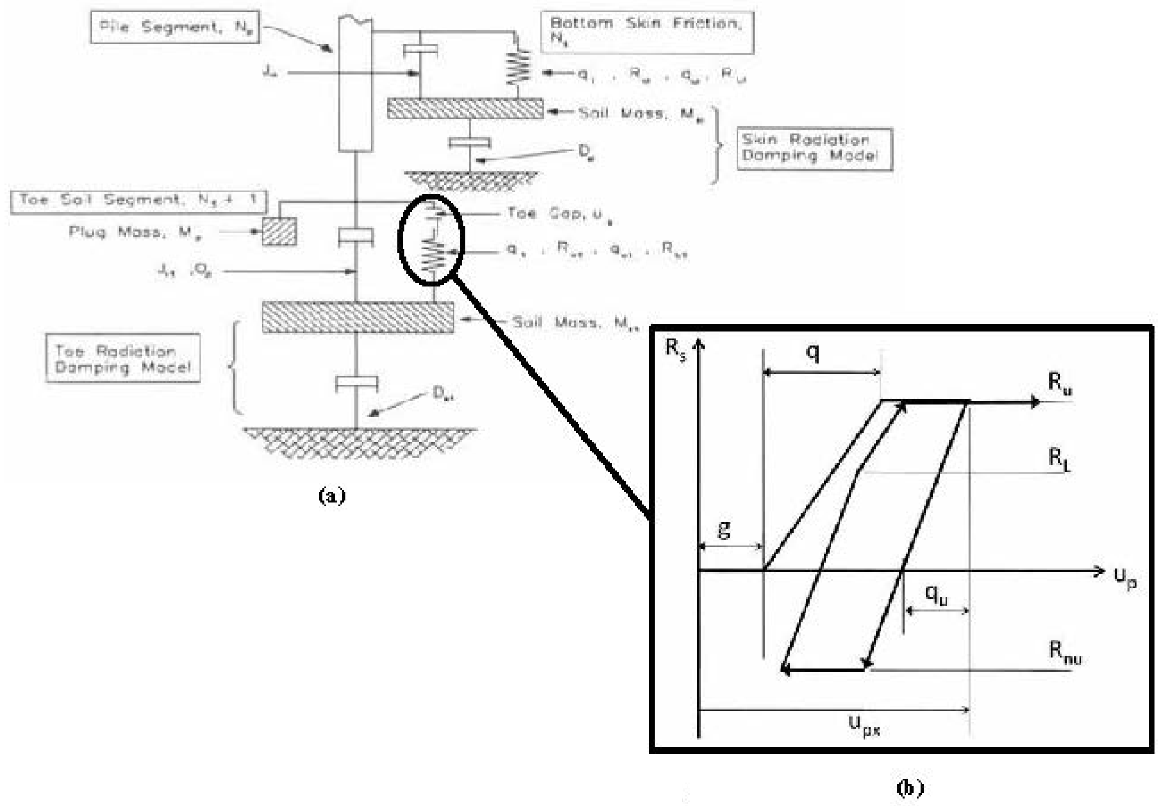


Figure 2.13: The CAPWAP soil model. (a) Extended Smith mass-spring-dashpot model (Rausche et al. 1994). (b) Static resistance and displacement curve that represents the linear-elastic-perfectly-plastic spring (Rausche et al. 2010). Reprinted with permission from Dr. Frank Rausche (PDI).

Chapter 3: Background Information

3.0 Introduction

The first section of Chapter 3 describes the geomorphology and geology of the lacustrine clay, till and limestone bedrock underlying Winnipeg. Soil and rock properties relevant to the static and dynamic analyses covered in this thesis are provided. The second section of Chapter 3 discusses uncertainty, variability, judgment, and the basic principles of LSD as they apply to deep foundations. The last section of Chapter 3 briefly describes soil stratigraphy, pile design, and the construction at twelve sites, where H-piles were installed and tested. No further mathematical expressions are given here; however, the background information provided in this chapter offers guidance for appropriate design.

3.1 Winnipeg Strata

The description of the Winnipeg geomorphology and geology in this section are based on Teller (1976), Teller and Fenton (1980), Baracos et al. (1983a), Baracos and Kingerski (1998), and Kehew (2005).

Winnipeg is located in the Red River Lowlands which occupies relatively the same region as pro-Lake Agassiz. The area is gently rolling to flat and composed of glacio-lacustrine clay, organic sediments and till. Underlying the Red River Lowland is Paleozoic carbonate sedimentary bedrock. A silt layer and a complex zone consisting of interlayering silt and clay are commonly encountered in the upper 4.6 m below the ground surface. Intermittent sand layers are encountered but are not common.

3.1.1 Winnipeg Geology – Bedrock

The bedrock underlying the Winnipeg area consists of four main Ordovician (second period of the Paleozoic era) sedimentary formations; the Stony Mountain, Winnipeg, Red River and Jurassic Amaranth formations. The bedrock is carbonate in nature and consists of primarily dolomite and dolomitic limestone. Shale and sandstone are found in some formation members but are not as prevalent. The surface of the bedrock is irregular and fractured. The pre- surface of the bedrock underwent periods of erosion and weathering, resulting in voids and fractures in the upper 0.3 to 0.9 m. The fractures generally contain infill consisting of clay to boulder particle sizes and till or mixtures of all. The upper 15 to 30 m of the bedrock contains discontinuities; with the frequency of these discontinuities usually decreasing with depth. Below the carbonate bedrock lays Precambrian basement rock consisting primarily of granite and gneiss.

3.1.1.1 Karst Features & Discontinuities

The bedrock in Winnipeg is karstic and contains discontinuities in the form of solution cavities, fissures, void, underground streams and sometimes sinkholes, which can all vary significantly in size. These features were created by pre- erosion and acidic surface and ground water, which chemically dissolved the rock. Glacial thrusting and groundwater weathered, transported and deposited pieces of the carbonate rock and other soils such as till within the discontinuities of the karst formations.

Joints and fractures are types of non-karstic discontinuities in bedrock. Joints and fractures that have not undergone any displacement can vary from hairline thickness to meters thick and can extend up to 3 meters long; they are often in-filled with silt and clay seams up to 50 mm thick. Bedding planes are also discontinuities that are non-karstic; they are layers of rock that had been deposited at different times on the geological time scale and are typically less than 25 mm thick.

3.1.1.2 Engineering Properties

The bedrock underlying Winnipeg has variable strength properties. The q_u of the intact dolomitic limestone ranges from 34 to 37 MPa, in weaker formations, to between 60 and 100 MPa in the stronger formations according to Baracos et al. (1983a). Seventeen UC tests conducted for the Richardson Complex Tower project yielded strengths ranging between 50 and 195 MPa. Das (2004) provides a typical range of 105 to 210 MPa for q_u of limestone; Bowles (1997) gives a range of 35 to 170 MPa. The geotechnical investigations conducted for the bridges of the Floodway Expansion Project (Manitoba Infrastructure and Transportation) yielded q_u values between 19 and 84 MPa. The RQD is variable from site to site and can range from 0 to 100 percent.

3.1.2 Winnipeg Geomorphology – Overburden Soils

Periods of glaciation formed the landscape of the Red River Lowlands. Massive glaciers advanced over the pre- bedrock formation and scoured the rock surface transporting and depositing till. The glacio-lacustrine clay settled out from Lake Agassiz after the last retreat, resulting in the overburden soils underlying Winnipeg.

The Late Wisconsinan Glaciation period began about 22 to 24 thousand years ago (ka) and ended with the final glacial retreat about 11 ka. The Laurentide Ice Sheet advanced from the northeast and deposited Precambrian-rich sandy till in the Red River Lowlands about 22 to 24 thousand years ago. The Keewatin ice sheet advanced into the Red River Lowlands from the northwest where it encountered the Laurentide Ice Sheet about 21 ka, depositing calcareous till. The Keewatin ice sheet continued to advance completely covering the Red River Lowlands between 14 and 20 ka. The Keewatin Ice Sheet deposited clayey till about 13 ka, then retreated only to re-advance, depositing sandy till. The glaciers retreated from southern Manitoba for the last time about 11 ka leaving pro-glacial Lake Agassiz in their place.

Pro-glacial Lake Agassiz extended east from the Manitoba Escarpment to the high rocky elevations of north western Ontario, covering the entire Winnipeg area. Lacustrine sediments from Lake Agassiz covered most of the till formations in the Red River Lowlands and particularly in the Winnipeg and South and North Dakota areas. The lacustrine sediment filled in the irregular surface of the till deposit in the Lake Agassiz basin, resulting in the extremely flat region that is seen today.

3.1.2.1 Till

Till is commonly known in the Winnipeg Area as "glacial till". Till is a heterogeneous mixture of clay, sand, gravel, cobbles and boulders within a predominately silt matrix. The clay and silt fractions are the main constituents of the till and can range between 40 to 60 percent of the soil deposit with silt fraction dominating. The till layer is typically about 3 to 6 meters thick, but can be as thick as 20 meters or more, or non-existent. The irregular till surface varies in depth from the ground surface; it may be exposed at the ground surface or encountered at a depth of more than 20 m.

There are three main types of till, in terms of deposition, that underlie Winnipeg; 1) water-laid till that settled out of pro-glacial Lake Agassiz, 2) ablation till, which was laid down by melting glaciers, and 3) basal till, which was transported in the base of glaciers and deposited underneath. Water-laid and ablation tills were not compressed under the massive loads of glaciers and are generally loose, water bearing, and have low bearing resistance. The clay fraction is often greater in these till segments than the basal till, which is more granular in nature, resulting in higher plasticity and moisture content. The basal tills were deposited by advancing and retreating glaciers. The massive loads imposed by the glaciers consolidated the basal till, squeezing much of the water out, resulting in an over-consolidated soil with relatively low moisture content (< 10 percent). Basal till is generally compact to very dense and moist to dry, having moderate to high bearing resistance. Artesian conditions in the underlying bedrock has caused softening of the bottom 0.3 to 0.9 m of the basal till layer; fractures present in the basal till may allow the artesian water to travel through the till creating intermittent soft spots.

The bearing resistance of the very dense basal till is substantial. The basal till layer is often recommended as the end bearing stratum for end bearing caissons, driven high-displacement piles and belled piles. Thick layers of this material can provide substantial frictional resistance for long driven steel H-piles. The bearing resistance of basal till is substantial enough to support heavy loads and cemented enough to remain stable in pile bores; however, it is not so dense as to restrict most construction drilling activities, or restrict driveability of driven low-displacement piles. Power auger refusal during geotechnical drilling often occurs in the dense to very dense basal till.

3.1.2.1.1 Engineering Properties

The geotechnical soil properties of tills as they pertain to the analysis performed in this thesis are variable. Moisture contents and the bearing pressures between the loose and wet water-laid tills, ablation tills and the very dense and dry basal tills vary significantly. The moisture contents in the loose and wet ablation till are in excess of 10 percent. This layer has poor bearing resistance with SPT N-values generally lower than 10 blows per 300 mm of penetration. Foundations are not designed to be seated in this weak layer.

The basal tills are suited for foundations. The compact and moist basal tills have moisture contents in the order of 10 percent. These tills have SPT N-values typically between 10 and 30 blows per 300 mm of penetration. The dense to very dense and dry basal tills have moisture contents ranging between 3.5 and 8 percent. These tills have SPT N-values typically greater than 30 blows per 300 mm of penetration. The very dense and dry basal tills are best suited to support heavy foundation loads. The moisture contents range between 3.5 to 8 percent and the SPT N-values are usually greater than 50 blows per 300 mm of penetration. Unconfined compressive strength tests that have been performed in the very dense basal tills have yielded strengths between 3.4 and 3.6 MPa.

3.1.2.2 Lacustrine Clay

The lacustrine clay is generally mottled brown and grey, stiff, highly plastic and weathered in the upper 1.5 to 4.6 m. Fractures and fissures are present in this portion of the clay layer. The clay generally becomes grey, firm to stiff, and intermediate to highly plastic below 1.5 to 4.6 m. The clay is comprised of varying amounts silt. Traces of sand, gravel, silt, and gypsum inclusions are common in the clay and cobbles and boulders can be often found near the till surface. The thickness of the clay layer ranges between about 0 and 21 m with a typical thickness in the order of 9 to 12 m.

3.1.2.2.1 Engineering Properties

The undrained shears strength of Winnipeg clay varies between about 35 and 85 kPa. The upper mottled brown and grey clays have variable s_u due to the fractures, fissures and weathering. The grey clays have lower measured s_u due to the higher moisture content and presence of till inclusions. The moisture contents range between about 40 to 60 percent and tend to increase with depth, then decrease closer to the till surface due to the presence of till inclusions.

3.2 Uncertainty, Variability, and Judgment

Uncertainty, variability, and risk exist with every foundation project no matter the size or complexity. A.R. Daneshkhah (2004) describes two types of uncertainty; 1) aleatory uncertainty and 2) epistemic uncertainty. Aleatory uncertainty refers to the variation of an event or a physical property, such as encountering boulders in till or discontinuities in bedrock. The aleatory uncertainty cannot reasonably be reduced by increased information or knowledge and is difficult to discretize. Epistemic uncertainty refers to resolvable uncertainty that can be reduced by gaining information and knowledge, such as increasing the quality and quantity of soil strength testing.

Judgment is important when handling uncertainty in foundation design. Judgment enables an engineer to interpret the aleatory uncertainty and apply appropriate RFs in LSD to decrease the probability of failure. Judgment can also prove to be essential in reducing the controllable epistemic uncertainty of foundation parameters by optimizing geotechnical investigations and the quantity and quality of soil strength testing.

3.2.1 Codes of Practice

Uncertainty in foundation design is handled by codes of practice. Becker (1996, p. 960) states that *"a code of practice describes recommended good engineering design practice by defining a set of requirements or provisions that are aimed at reaching a reasonable technical level of quality and the desired or specified level of safety."* The 2010 versions of the NBCC and the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications are examples of codes of practice. Embedded in these codes of practice are LSD guidelines for deep foundations.

3.2.1.1 Limit States Design

A full description of the application of LSD to foundation design is discussed by Becker (1996). The purpose of LSD is to produce a foundation that will tolerate all loading events that may occur during its life span with an appropriate level of reliability against failure and excessive settlement at a reasonable cost. Limit states design has been integrated into foundation engineering over the past few decades. The City of Winnipeg requires that LSD criteria be incorporated into all foundation reports to acquire building permits. Incorporating LSD into geotechnical engineering provides compatibility and consistency with structural LSD, in contrast to the working stress design method. There are two main conditions in LSD; serviceability limit state and ultimate limit state.

Serviceability limit state refers to the performance of a structure or its components under service loads. Although the consequence of failing to meet the SLS criteria is not as devastating as total failure, it can lead to substantial economic loss. Settlement and the corresponding Q_{SLS} are the main SLS components associated with foundations loaded in axial-compression.

Ultimate limit state refers to the safety of a structure against partial or total failure. The consequence of failure of a pile can be devastating in terms of human life and economic loss. The Q_{ult} is the main ULS component associated with the safety of foundations loaded in axial-compression. The basic ULS equation is expressed as,

$$\phi R = \alpha S \tag{3.1}$$

Where, Φ is the RF, α is the load factor, S is the un-factored load and R is the nominal resistance, which represents the Q_{ult}. The ϕR term represents the Q_{ULS}. Resistance is generally handled by the geotechnical engineer and the loading is generally handled by the structural engineer.

The RF represents the probability that the actual resistance will be less than the calculated characteristic resistance (Becker 1996). Resistance factors have a magnitude of 1 or less and take into account the probability that the Q_{ult} may be less than maximum imposed loads. The RFs and load factors are often developed and calibrated using statistical probabilistic and reliability based design methods, but are also calibrated to the factor of safety associated with the working stress design method. The NBCC (2010) explains that the RFs presented in the code *"…were derived by direct calibration to traditional working (allowable) stress design."* (NBCC User's Guide 2010, pg. K-2). The NBCC (2010) and the AASHTO LRFD Bridge Design Specifications (2010) provide recommendations for RFs for different RAMs that are used to calculate the Q_{ult} of deep foundations.

The numerical value of a RF is based on the quality the RAM used to calculate the Q_{ult}. A RF is assigned to the RAM based on its reliability; the more reliable the RAM the less uncertainty and therefore the greater the RF. Table 3.1 provides RAMs and the corresponding RFs recommended by the NBCC (2010) and the AASHTO LRFD Bridge Design Specifications (2010). It should be noted from reviewing Table 3.1 that the AASHTO LRFD Bridge Design Specifications (2010) is more extensive than the NBCC (2010), as it includes WEA and dynamic formulas, and provides combinations of different RAMs with varying RFs. Also, the NBCC (2010) is significantly more conservative than the AASHTO LRFD Bridge Design Specifications (2010).

Neither PDA testing nor CAPWAP analysis are mentioned in either code. Dynamic monitoring or testing is the referred term. Whether or not dynamic monitoring or testing refers to PDA testing alone or both PDA testing and CAPWAP analysis is unknown. This needs to be addressed, especially since the statistical evidence shows the reliability of CAPWAP analysis is superior to PDA testing by itself.

To finish this section on LSD, it should be noted from the NBCC (2010) and the AASHTO LRFD Bridge Design Specifications (2010), that using dynamic monitoring (or testing) justifies the use of higher recommended RFs in comparison to empirical and theoretical methods. This may not carry much weight for small structures, but for large structures the increased RF may lead to significant cost savings, by reducing the number and (or) size of the piles required in the foundation system. Improved confidence in foundation performance is also realized by using more reliable RAMs.

3.3 Site Descriptions

Sixty-four PDA tests and CAPWAP analyses were conducted over twelve sites throughout Winnipeg. This section provides a brief description of the soil stratigraphy and H-pile and construction details at each site.

3.3.1 Site 1

Six PDA tests were conducted at Site 1, including two under EOID conditions and four under BOR conditions. The tests were conducted on three H-pile sizes, including: HP310x132, HP310x110 and HP310x94 sections cast from 350W steel. The piles were driven using an I.C.E. Model I-19 single-acting open-ended diesel hammer set at fuel setting number 2 at a rated energy 51.8 kJ. The piles were driven to practical refusal in bedrock using a design refusal criterion defined by three consecutive sets of 15 blows per 25 mm. Pile penetration lengths ranged between 12.4 and 15.8 m below ground surface.

The soil stratigraphy consists of clay fill overlying lacustrine clay, silt till and limestone bedrock. A thin sand layer was encountered sporadically between the clay and the till. The lacustrine clay layer is generally stiff and ranges in thickness from 10.4 to 12.8 m. The silt till is loose and wet near the surface of the layer and becomes dense and dry with depth. The till ranges in thickness between 1.1 and 1.8 m. The condition of the bedrock is highly variable. It is intact and massive at some locations within the site and highly fractured in others.

3.3.2 Site 2

Two H-pile sizes (HP200x54 and HP360x132) cast from 350W steel, were tested at site 2. Four piles were tested under EOID conditions and two under BOR conditions. The piles were driven using a Junttan HHK 5A hydraulic hammer equipped with a 5000 kg ram. A 0.5 m stroke having a rated energy of 24.5 kJ was used to install the HP200x54 pile. A 1.1 m stroke having a rated energy of 53.9 kJ was used to install the HP360x132 pile. The H-piles were driven to practical refusal in bedrock using a refusal criterion of three consecutive sets of 12 blows per 25 mm. Pre-bore holes were drilled to minimize vibrations in the existing structures and ranged in depth between 5 and 7 m below ground surface. Pile penetration depths below the bottom of the pre-bore holes ranged between 10.5 and 17.8 m.

The soil stratigraphy consists of clay fill overlying lacustrine clay, silt till and limestone bedrock. The lacustrine clay and silt till layers are 11.9 and 8.7 m thick, respectively. The clay is stiff and the till is dense to very dense and dry. The bedrock is highly fractured in the upper 3 m and massive below.

3.3.3 Site 3

One pile was tested under BOR conditions at Site 3. The pile was an HP310x110 section cast from 350W steel. It was driven to bedrock using a Junttan HHK 5A hydraulic hammer equipped with a 5000 kg ram and using a stroke of 0.7 m having a rated energy of 34.3 kJ. The refusal criterion for this H-pile is unknown. The pile penetration depth below ground surface was 12.8 m.

The soil stratigraphy consists of lacustrine clay and silt till overlying limestone bedrock. The lacustrine clay is stiff, becoming firm with depth and ranged in thickness between 9.2 and 10.1 m. The silt till is compact to dense, moist to dry and ranged in thickness between 5.1 and 6.4 m.

3.3.4 Site 4

Four HP310x110 H-piles rolled from 350W steel were tested at this site under BOR conditions using a Junttan HHK 5S hydraulic hammer equipped with a 5000 kg ram. A

0.5 m stroke was used for driving the piles equalling a rated energy of 24.5 kJ. The design refusal criterion followed was three consecutive sets of 15 blows per 25 mm of pile penetration. Pre-bore holes, 9 m in depth, were drilled through an existing tunnel to minimize ground vibrations and facilitate driving. Penetration depths below the bottom of the pre-bore holes ranged between 5.3 and 6.9 m. All H-piles were driven to bedrock.

The soil stratigraphy consists of lacustrine clay and silt till underlying the tunnel and overlying the limestone bedrock. The lacustrine clay is stiff and ranged in thickness between 3 and 6.5 m. The silt till layer is loose and wet and thin with thicknesses between about 0.2 and 1.3 m. The limestone bedrock is slightly fractured in the upper 0.5 m and massive below.

3.3.5 Site 5

Four HP310x110 H-piles rolled from 350W steel were tested at this site. Two of the H-piles were tested under EOID conditions and three piles were tested under BOR conditions. The piles were driven to bedrock using a 2268 kg drop hammer with drop heights ranging between 1.5 and 3 m which corresponds to a rated energy fluctuating between 33 and 67 kJ. 4.6 m pre-bore holes were drilled to minimize vibrations in the existing structures. Penetration depths below the bottom of the pre-bore holes ranged between 10.8 and 12.5 m. The design refusal criterion consisted of driving the piles to practical refusal defined by 3 consecutive sets of 12 blows per 25 mm of pile penetration.

The soil stratigraphy cannot be described at this site due to denial by the client to use the test hole logs. Therefore, no soil testing data from this site is used in the thesis analyses.

3.3.6 Site 6

Four HP360x132 H-piles cast from 350W steel were tested at site 6. Two of the H-piles were tested under EOID conditions and three piles were tested under BOR conditions. One of the piles was tested under both EOID and BOR conditions. Two piles were driven using a Delmag D19-32 single-acting open-ended diesel hammer set at fuel setting number 4 having a rated energy of 57.6 kJ. Four piles were driven using a Pileco D19-42 single-acting open-ended diesel hammer set at fuel setting number 3, which correlates to a rated energy of 47.8 kJ. All piles were driven to practical refusal in bedrock following the design refusal criterion of three consecutive sets of 13 blows per 25 mm. Pre-bore holes were drilled for four of the piles which ranged between 3 and 5.5 m. Pile penetration depths below the bottom of the pre-bore holes ranged between 4 and 6.6 m.

The soil stratigraphy consists of gravel and clay fills overlying lacustrine clay, silt till and calcareous mudstone bedrock with argillaceous partings. The lacustrine clay is about 3.1 to 4.4 m thick. It is firm to stiff, becoming softer with depth. The till is about 1.6 to 3.9 m thick. It is loose and wet. The bedrock is of the weak Gunn Member and is highly variable in strength and composition. The author found that quantities of this rock can be broken by hand and dissolved in water; other parts of the bedrock are hard like limestone.

3.3.7 Site 7

Eight HP310x110 H-piles rolled from 350W steel were PDA tested under BOR conditions at Site 7. The piles were driven using a Junttan HHK 5A hydraulic hammer equipped with a 5000 kg ram. A 0.8 m stroke, correlating to a rated energy of 39.2 kJ, was used to drive the piles. Pre-bore holes ranging in depth between 3.8 and 5.6 m were drilled to facilitate installing the pile caps. Penetration depths below the bottom of the pre-bore holes ranged between 17.9 and 32.2 m. The design refusal criterion consisted of driving the piles to practical refusal as defined by three consecutive sets of 12 blows per 25 mm. Seven of the piles refused in silt till and one pile was driven to bedrock.

The soil stratigraphy consists of alluvial clay overlying silt till and limestone bedrock. The alluvial clay contained intermittent sand and silt layering; which is common of alluvial clay. The silt till layer is about 26 m thick and highly variable. It is abnormal for till layers to be that thick. It is generally very dense and dry. Irregular occurrences of gravel, cobbles and boulders are observed in the upper half of the layer. In the bottom half of the till, the cobble and boulder frequency increases significantly with depth. The limestone bedrock is slightly fractured in the upper 1.5 m and massive below.

3.3.8 Site 8

Four HP310x110 piles rolled from 400W grade steel were PDA tested a Site 8 using a Pileco D19-42 single-acting open-ended diesel hammer set at fuel setting number 3 having a rated energy of 47.8 kJ. One pile was tested under EOID conditions and three piles were tested under BOR conditions. The piles were driven to bedrock with penetration depths of 11.2 to 12.4 m below ground surface. The design criteria required that the piles be driven to practical refusal defined by 12 blows per 25 mm.

The stratum consists of clay fill overlying lacustrine clay, silt till and limestone bedrock. The lacustrine clay is 3.9 to 5.8 m thick. It is firm to stiff changing to firm with depth. The silt till is compact, moist and 3.1 to 5 m thick. The limestone bedrock is highly fractured in the upper 1.5 m and becomes massive with depth.

3.3.9 Site 9

PDA tests were completed on four HP310x125 H-piles rolled from 350W steel at three different structures at Site 9. One pile was tested under EOID conditions and three piles were tested under BOR conditions. The piles were driven to bedrock using a Pileco D19-42 single-acting open-ended diesel hammer set at fuel setting number 3 with a rated energy of 47.8 kJ. The final pile penetration lengths were between 14.3 to 16.1 m. The design refusal criterion consisted of driving the piles to practical refusal as defined by 3 consecutive sets of 15 blows per 25 mm of pile penetration.

The soil stratigraphy consists of lacustrine clay and silt till overlying limestone bedrock. The lacustrine clay is stiff becoming firm with depth and ranged in thickness between 4.6 and 10.6 m. The silt till layer is generally dense to very dense and fluctuates in thickness between 2.9 and 9 m. The limestone bedrock is generally massive with fractures in the upper 1 to 1.5 m.

3.3.10 Site 10

One H-pile rolled from 350W steel was PDA tested at the EOID at Site 10. The HP310x132 H-pile was driven using a Pileco D19-42 single-acting open-ended diesel hammer set at fuel setting number 3 having a rated energy of 47.8 kJ. The pile refused in bedrock at a final penetration of 19 m. The refusal criterion at this site is unknown.

The soil stratigraphy consists of lacustrine clay overlying silt till and what is assumed to be limestone bedrock. The test holes drilled at this site were not taken into the bedrock; therefore, contact depths could not be determined. The lacustrine clay is stiff turning firm with depth. The thickness of the clay layer was about 15.6 m. The till is typically loose and moist, becoming dense to very dense with depth.

3.3.11 Site 11

Two HP310x132 H-piles rolled from 350W steel were PDA tested under EOID conditions at this site. The piles were installed using a Pileco D19-42 single-acting open-ended diesel hammer set at fuel setting number 3 with a rated energy of 47.8 kJ. The final pile penetration depths were 20.3 m below ground surface. Both piles are seated in bedrock. The refusal criterion at this site is unknown.

The soil stratum consists of clay fill, lacustrine clay and silt till covering limestone bedrock. A silt layer was intermittently encountered at this site in the upper 3 m below ground surface. The lacustrine clay is stiff and becomes firm with depth and is approximately 13 to 14.4 m thick. The till is about 2.4 to 4.6 m thick. It is moist and compact near the surface and turns very dense and dry with depth. The limestone bedrock is fractured and contains sporadic clay seams in the upper 0.5 to 3.4 m and is relatively massive below.

3.3.12 Site 12

Eighteen HPx125 H-piles rolled from 350W steel were PDA tested at this site. Ten of the piles were tested under EOID conditions the remaining piles were tested under BOR conditions. The piles were driven using two different single-acting open-ended diesel hammers; a Delmag D19-32 set at fuel setting number 3 and 4 having rated energies of 47.1 and 57.6 kJ, respectively; and a Pileco D19-42 set at fuel setting numbers 3 and 4 with rated energies of 47.8 and 57.6 kJ, respectively. The piles were driven to practical refusal at 15 blows per 25 mm. The piles ranged in penetration depth from 19.9 to 31.6 m. Seven of the H-piles refused in the sand till and twelve were carried down to bedrock.

The soil stratigraphy consists of lacustrine clay and till covering limestone bedrock. The lacustrine clay varies significantly due to varying ground elevations and ranges in thickness from 14.2 to 21 m. The clay is generally stiff and becomes firm to stiff with depth. The till is highly irregular and continuously changes from a dense and moist to wet sand till to silt till that varies from compact and moist to very dense and dry. The entire till layer ranges in thickness from 7.2 to 9.6 m. The limestone bedrock is fractured at some of the site locations in the upper 1.5 m and turns massive below. In other locations the rock is entirely massive with no fractured zones.

3.4 Chapter 3 Discussion

The Winnipeg Geological setting is generally friendly for installation of driven steel H-piles. The piles shear through the clay with ease and usually through the till and fractured bedrock without too much difficulty, becoming seated in the massive bedrock as depicted in Figure 3.1. Some difficulty in driving H-piles may occur in till deposits with frequent boulders or till deposits that are substantially thick. However, it is relatively rare for an H-pile to refuse in the till.

The remainder of this thesis integrates the static and dynamic RAMs discussed in Chapter 2 with the soil and rock properties discussed in this chapter to better understand the behavior of driven steel H-piles and improve the conventional design criteria in Winnipeg. The RAMs and corresponding RFs provided in the NBCC (2010) and the AASHTO LRFD Bridge Design Specifications (2010) and discussed in this chapter are investigated with the purpose of making improvements to the LSD criteria specific to driven steel H-piles in Winnipeg.

Table 3.1: Resistance factors provided by NBCC (2010) and AASHTO (2010).

Code/Guideline	**Resistance Analysis Method**	**ϕ**
NBCC (2010)	Semi-empirical – insitu testing & lab data	0.4
	Dynamic Monitoring (PDA)	0.5
	Static Load Tests	0.6
AASHTO LRFD Bridge Design Specifications (2010)	Driving criteria established by successful static load test of at least 1 pile per site condition and dynamic testing of at least 2 piles per site condition, but no more than 2% of the production piles	0.8
	Driving criteria established by successful static load test of least 1 pile per site condition without dynamic testing.	0.75
	Driving criteria established by dynamic testing conducted on 100% of production piles.	0.75
	Driving criteria established by dynamic test with signal matching at beginning of redrive (BOR) conditions only of at least 1 production pile per pile pier, but no less than the number of tests per site provided in Table shown below	0.65
	Wave equation analysis, without dynamic measurements of load test, at end of drive conditions only.	0.4
	FHWA-modified Gates dynamic pile formula, end of drive conditions only.	0.4
	Engineering News Record dynamic pile formula, end of drive condition only.	0.1

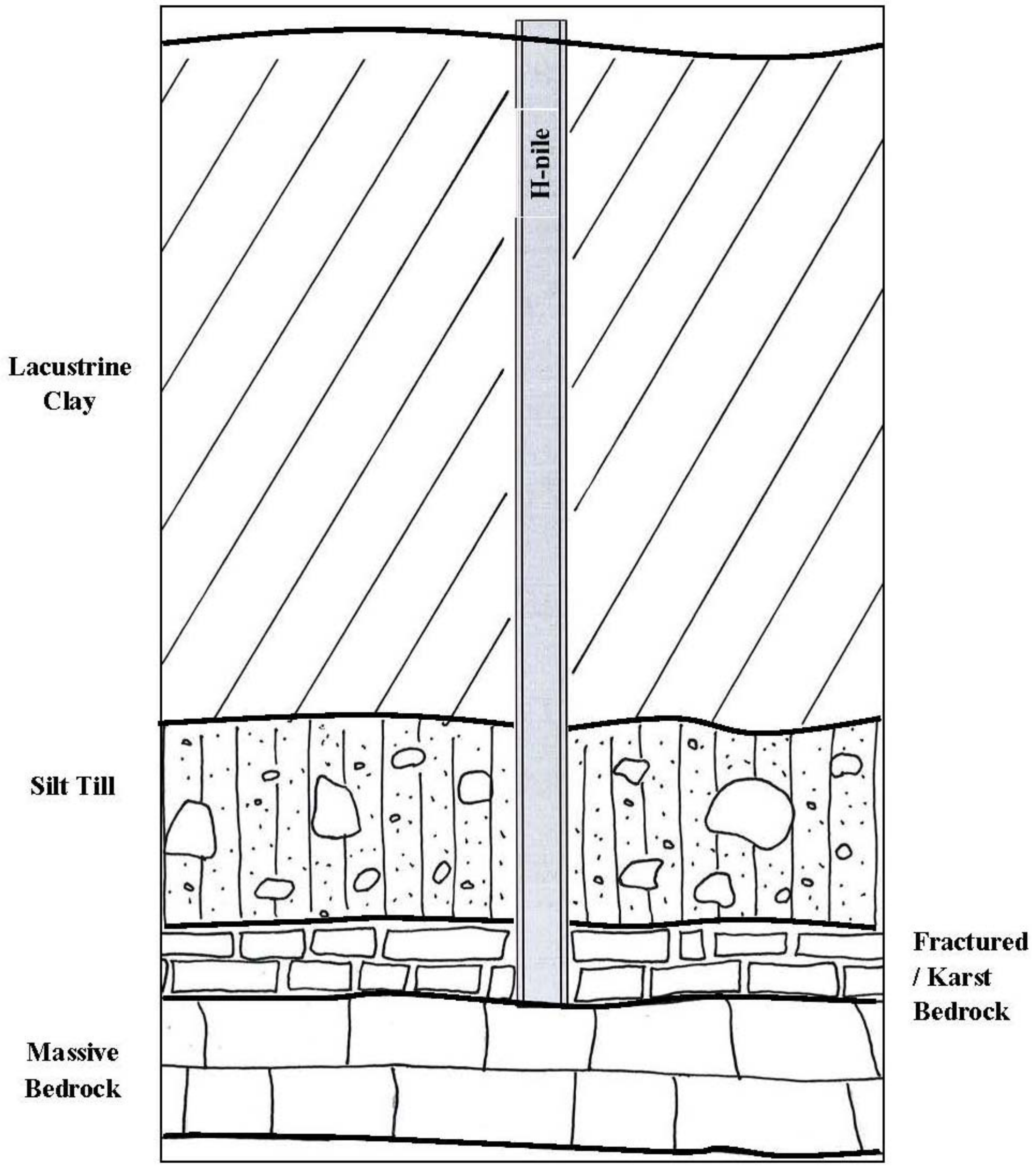


Figure 3.1: An H-pile seated in typical Winnipeg soil and rock stratigraphy.

Chapter 4: Static Analysis of Driven Steel H-Piles in Winnipeg

4.0 Introduction

Chapter 4 presents the results of the static analyses. The CAPWAP analyses of sixty-four driven steel H-piles was compiled, reviewed, and examined to compare the measured ultimate capacities (*i.e.* Q_{ult}, Q_{s-ult}, and Q_{b-ult}) to the ultimate capacities predicted by the conventional ULS design criteria. A detailed analysis, consisting of correlating the CAPWAP-measured values of q_{b-ult} and f_{s-ult} of the H-piles to measured soil and rock strength properties of the lacustrine clay, till, and bedrock, was also undertaken. The results of the correlations made were compared to the static RAMs presented in Chapter 2, to assess the accuracy of the RAMs. Judgement and experience played an integral role in correlating the *in-situ* strength testing and laboratory testing data to the CAPWAP data to assess, refine, and deliver usable engineering approaches for estimating H-pile capacity. Table 4.1 provides a summary of all pertinent CAPWAP data necessary to perform the analyses. The soil and rock strength properties are shown on the test holes logs attached in Appendix A and the CAPWAP outputs are attached in Appendix B.

4.1 Assumptions & Conditions

Some assumptions were necessary to evaluate the Q_{ult} of the H-piles; they are as follows:

- The Q_{ult} is assumed to be fully mobilized during PDA testing.
- A soil plug is assumed to form along the H-pile shaft in the lacustrine clay and till, as discussed in Chapter 2, and shaft frictional resistance develops along the outside (box) perimeter of the H-pile.

- The formation of a rock plug at the toe is assumed to be disturbed and does not contribute to end bearing resistance, for the reasons provided in Chapter 2 regarding soil and rock plugging. The end bearing resistance is concentrated to the cross-sectional area at the H-pile toe.
- A pile is driven under BOR testing conditions if the following criteria are met:
    - the pile is re-struck a minimum of one day after EOID and
    - the pile is driven no more than 25 blows
        - if the pile is driven more than 25 blows, it is considered to be driven under EOID conditions

It should be noted that all of the soil and rock testing and pile driving information was gathered after the completion of the foundation construction at the twelve different sites. The author had no involvement in the design of the geotechnical drilling and testing program.

4.2 Analytical Procedure

An involved procedure was undertaken to evaluate, develop, and refine the static RAMs. The first and most difficult task began by comparing the soil profiles to the penetration records of the H-piles and the CAPWAP results to correlate the soil properties to the calculated soil resistances. This was imperative to ensure that the correct H-pile resistances matched the correct bearing material and its soil properties; otherwise, the values of the CAPWAP analysis would not be representative of the documented soils.

The *in-situ* and laboratory test results for different soil layers (*i.e.* clay and till) were matched to the average CAPWAP-calculated soil resistance values at the corresponding depths. At each site, the average s_u values in the lacustrine clay layer, and the corrected SPT N-values and moisture contents of the till layer were compared to the average CAPWAP-calculated resistances for those layers. The CAPWAP-calculated resistance values of a layer were separated and analysed as distinct layers if there was a drastic and discernible change in the consistency of that layer, and if there was an obvious and extractable difference in the resistance values. For example, a clay layer that changes from stiff to soft would be treated as two separate layers.

The goal of quantifying the resistance of the till was based on isolating the typical silt matrix-dominant till from sandy tills; therefore, layers of till were handled with caution. Till layers that exhibited extensive intermittent layering of sandy or gravelly beds were excluded from the analysis as these layers were difficult to quantify. Till layers demonstrating significant deviations from the typical silt matrix were ignored as they caused high variability in the soil properties and the $f_{s\text{-}ult}$ values determined by CAPWAP analysis. Thin layers of till were also ignored, to ensure the CAPWAP penetration depths were correlated to the appropriate soils. Sand till was analysed on the basis of corrected SPT N-values. Moisture content correlation was not attempted in the sand till.

4.3 Ultimate Bearing Capacities

The Q_{ult} and $Q_{b\text{-}ult}$ of the sixty-four driven steel H-piles were reviewed in regard to pile size and bearing strata. Six different H-pile sizes rolled from 350W and 400W grade steel were evaluated. The pile sizes include, HP200x54, HP310x94, HP310x110, HP310x125, HP310x132, and HP360x132 sections. H-piles seated in karst bedrock, silt till, and sand till were assessed. The CAPWAP-calculated Q_{ult} values were compared to the Q_{ult} values as determined by the Winnipeg conventional ULS design criteria (discussed in Chapter 1) to determine their validity.

To review, the conventional design criteria in their entirety (Equations 1.1 to 1.5) makes two important assumptions, 1) that a driven steel H-pile driven to bedrock is an end bearing pile and shaft friction is considered negligible and 2) the Q_{ult} is directly related to the structural strength of the steel. Analysis of the data presented in Table 4.1 and a comparison of the ratio of Q_{ult} to Q_{b-ult}, as illustrated in Figures 4.1 to 4.3, reveals that the Q_{b-ult} constitutes on average only 75 percent of the Q_{ult}. This proves that the first assumption of the conventional design criteria is incorrect. The Q_{b-ult} ranges between 38.5 and 94.3 percent of the Q_{ult} with a COV equal to 0.16. The ratio of Q_{ult} to Q_{s-ult} ranges between 1.6 and 18.4, with an average value of 5.2 and a COV equal to 0.59. The ratio of Q_{b-ult} to Q_{s-ult} ranges between 0.6 and 17.4, with an average value of 4.2 and a COV equal to 0.73. It is discovered, upon viewing Figures 4.1 to 4.3, that significant variability of the Q_{ult} and Q_{b-ult} exists. This demonstrates that the second assumption of the conventional design criteria is also incorrect. Considering the strength properties of steel do not vary significantly, the structural strength of steel does not govern capacity. A more detailed discussion of these two broad conclusions follows.

4.3.1 CAPWAP-Calculated Ultimate Capacity versus the Ultimate Capacity Estimated by the Conventional Design Criteria

Review of Figure 4.1, shows a slight increasing trend of the CAPWAP-calculated Q_{ult} with respect to pile size. This is expected, because larger pile sizes have more cross-sectional area to activate more of the bearing resistance available from the soil or bedrock. Larger piles also have more surface area to activate more shaft resistance from the overburden soil. Simply put, the Q_{ult} of a driven steel H-pile is a function of pile size; the larger the pile, the greater the capacity. The $Q_{b\text{-}ult}$ was evaluated in the same basic manner. Figure 4.2 shows the same increasing trend of $Q_{b\text{-}ult}$ with pile size. Figure 4.3 presents the Q_{ult} and $Q_{b\text{-}ult}$ of all the H-piles together for comparison. This figure provides a visual depiction of the significant difference between the two components of capacity and the variability of the capacities.

The CAPWAP-measured values of Q_{ult} of the different H-pile sizes were compared to the predicted values as determined by the conventional LSD design criteria defined by $0.6f_y'A_p$ and $0.75f_y'A_p$. Figures 4.4 to 4.8 illustrate the comparison. An observation that one should make from these figures is, again, the variability of the measured Q_{ult} values; there is no consistent correlation to the conventional design criteria values. Another observation that should be made from Figures 4.4 to 4.8 is that the conventional design criteria generally overestimate the actual Q_{ult}.

The following sections of statistical analysis, deliver detailed comparisons of the measured CAPWAP ultimate capacities to the Q_{ult} as predicted by the conventional design criteria.

4.3.1.1 Statistical Comparison of the CAPWAP-Calculated Ultimate Capacity to the Ultimate Capacity Estimated by the Conventional Design Criteria

Calculations of the Q_{ult} using ULS design criterion provided in Equation 1.1, showed a major over-prediction of actual Q_{ult}. Therefore, this particular criterion was not investigated any further. Comparison of the Q_{ult} as defined by the conventional design criterion of $0.6f_y'A_p$ (Equation 1.2) shows that the average absolute difference between the measured and the predicted Q_{ult} for all pile sections is 20 percent with a standard deviation of 12 percent and a COV equal to 0.58. The Q_{ult} of 85.9 percent of the H-piles (55 piles) was over-predicted using this criterion. Comparison of the Q_{ult} as defined by the conventional design criterion of $0.75f_y'A_p$ (Equation 1.3) shows that the average absolute difference between the measured and predicted capacities is 33 percent with a standard deviation of 13 percent and a COV equal to 0.39. The Q_{ult} of all the H-piles was over-predicted using this criterion. This amount of inaccuracy and variability is a firm indication of the unreliability of the conventional ULS design criteria given by Equations 1.2 and 1.3. Little variability would be expected in the case where the capacity of piles driven to bedrock is governed by the structural strength of steel, since the strength characteristics of steel do not exhibit much variability. This is a clear indication that the strength of the steel does not govern ultimate H-pile bearing capacity.

4.3.1.2 Statistical Comparison of the CAPWAP-Calculated Ultimate End Bearing Capacity to the Ultimate Capacity Estimated by the Conventional Design Criteria

Comparison of the Q_{ult} as defined by the conventional design criterion of $0.6f_y'A_p$ reveals that 98.4 percent of the H-piles (63 piles) had a measured $Q_{b\text{-}ult}$ on average 38 percent less than the predicted value. All the H-piles had a $Q_{b\text{-}ult}$ less than predicted by the conventional design criterion as defined by $0.75f_y'A_p$, with an average difference of 50 percent less. These data are further evidence that the conventional ULS design criteria is flawed as it grossly over-predicts the $Q_{b\text{-}ult}$. The statistical evaluation of the $Q_{b\text{-}ult}$ proves that $Q_{s\text{-}ult}$ needs to be included in the analysis of the Q_{ult}.

4.3.1.3 Statistical Comparison of the CAPWAP-Calculated Ultimate Capacities in Till and Bedrock

This section gives evidence that the Q_{ult} and $Q_{b\text{-}ult}$ are not only dependent on the H-pile size but also the soil or rock in which it is seated. According to the conventional design criteria, the H-pile capacity is not influenced by the soil or rock strata. Fortunately, it was possible to investigate this assumption, since fourteen of the sixty-four tested piles refused in till. Seven of the fourteen piles were seated in cobbly and bouldery silt till at Site 7 and seven piles were seated in sand till at Site 12. The remaining piles were seated in bedrock.

The seven piles seated in cobbly, bouldery silt till at Site 7 had measured Q_{ult} and $Q_{b\text{-}ult}$ values less than predicted by the conventional design criteria. The Q_{ult} was on average 26 and 41 percent less than the Q_{ult} as defined by the design criteria of $0.6f_y'A_p$ and

$0.75f_y'A_p$, respectively; the measured $Q_{b\text{-}ult}$ was 56 and 65 percent less than estimated by the same criteria.

All seven piles seated in sand till at Site 12 had measured Q_{ult} and $Q_{b\text{-}ult}$ values less than the predicted amount determined from the conventional design criteria. The seven piles seated in the sand till had averages of measured Q_{ult} of 36 and 49 percent less than the predicted values calculated by the conventional design criteria as defined by $0.6f_y'A_p$ and $0.75f_y'A_p$, respectively. The measured $Q_{b\text{-}ult}$ was 54 and 63 percent less than predicted for the same design criteria. Several other piles, in addition to the seven seated in the sand till, were driven to bedrock at the same site. These particular piles exhibited a measured $Q_{b\text{-}ult}$ of 32 and 46 percent less than predicted by the conventional design criteria as defined by $0.6f_y'A_p$ and $0.75f_y'A_p$, respectively, which are noticeably greater end bearing resistances than the piles seated in the sand till.

Piles seated in bedrock showed greater Q_{ult} than those seated in till, as expected by the author. The piles seated in bedrock exhibited measured Q_{ult} on average of 17 and 30 percent lower than predicted by the conventional design criteria as defined by $0.6f_y'A_p$ and $0.75f_y'A_p$, respectively. The $Q_{b\text{-}ult}$ was measured on average to be 33 and 46 percent less than estimated by the same criteria, respectively. Discernible variability was evident in the measured Q_{ult} with a COV equal to 0.61 and 0.41 based on the conventional design criteria as defined by $0.6f_y'A_p$ and $0.75f_y'A_p$, respectively. The COV of the $Q_{b\text{-}ult}$ was 0.46 and 0.27 for the same respective design criteria. One pile seated in bedrock met the conventional design criterion as defined by $0.6f_y'A_p$, in regards to $Q_{b\text{-}ult}$, by exceeding the predicted value by 4 percent.

4.4 Ultimate End Bearing Capacity: Evaluation and Refinement of Static RAMs

Comparing laboratory and *in-situ* testing values to the CAPWAP-measured $q_{b\text{-}ult}$ allowed for assessment and refinement of existing static RAMs for their specific applicability to driven steel H-piles. Corrected SPT N-values and moisture content of the silt till and the q_u of the bedrock were compared to the CAPWAP-calculated $q_{b\text{-}ult}$ of the H-piles seated in the till and rock. The static RAMs used to estimate the $q_{b\text{-}ult}$ of piles, which were discussed in Chapter 2, were reviewed and refined. The static RAMs presented below will be referred to as the refined static RAMs for estimating the $q_{b\text{-}ult}$ of driven steel H-piles in Winnipeg.

4.4.1 q_u-Method

Fifty CAPWAP analyses were conducted on H-piles seated in bedrock. Unconfined compression tests were conducted on six rock core samples at six of the twelve sites. The average measured H-pile capacity at each of the six sites was compared to the q_u results of the UC tests. The $q_{b\text{-}ult}$ of the H-piles seated in the bedrock ranges between 85 and 219 MPa with an average of 142 MPa. The q_u of the rock ranges between 23 and 84 MPa with an average of 38 MPa. The datum points of $q_{b\text{-}ult}$ of the H-piles and the correlating q_u of the bedrock from the six project sites were plotted together on Figure 4.9, along with the relationship presented by Prakoso and Kulhawy (2002). The Prakaso and Kulhawy relationship (2002) is favourable in terms of the relative ease in measuring the input properties (*i.e.* the measured q_u of the rock). It also represents the assumed failure mechanism (*i.e.* compressive failure), which was discussed in detail in Chapter 2. It can be seen in Figure 4.9 that, although the amount of data is limited, stronger rock generally produces greater $q_{b\text{-}ult}$ values. The ratio of the CAPWAP-measured values of $q_{b\text{-}ult}$ to the measured values of q_u, determined by UC tests, ranges between 2.0 and 5.7 with an average of 4.4. The ratios are based on the assumption that the end bearing area is that of the cross-sectional area of the steel HP section.

Relating the $q_{b\text{-}ult}$ of the H-piles to the q_u of the bedrock is attractive because it is the simplest and most practical indicator of the strength of the rock. Although the correlation between $q_{b\text{-}ult}$ and q_u was made based on limited available UC test data, the available data covers a relatively large area, extending from the north to the south ends of Winnipeg. This provides enough distance to eradicate a possible site bias in the data. However, until more correlation data can be collected, it is recommended to use the Prakaso and Kulhawy relationship (2002).

4.4.2 Moisture Content Method

Fourteen CAPWAP analyses were conducted on piles seated in in till. Seven of these piles at Site 7 were seated in gravelly, cobbly, bouldery silt till, containing intermittent wet sand layers. The gravel, cobbles and boulders were a mixture of granite and limestone. This material made it very difficult for drilling and required coring techniques to advance the test holes to bedrock. To give an example of the difficulty, construction of rock socketed caissons at the same site required long periods of chopping and coring techniques to get through this heterogeneous layer and advance the caisson bores. Standard Penetration Tests were not feasible in this soil layer, due to the presence of the boulders, making it impossible to relate *in-situ* testing of the silt matrix to the q_{b-ult} of the H-piles. A few grab samples of the silt matrix were retrieved however for moisture content correlation. The average moisture content of the silt matrix till was 7.2 percent. The q_{b-ult} of the HP310x110 sections seated in the silt till ranged between 73 and 105 MPa with a mean value of 93 MPa and a COV of 0.13.

4.4.3 SPT-Method

Seven of the sixty-four H-piles at Site 12 were seated in a sand till. Trace to some gravel and cobbles were present in the till with infrequent boulders. Standard Penetration Tests were conducted in this layer at the location of the pile tips. About half of the SPTs were terminated with less than 300 mm of penetration. These SPTs had corrected N-values ranging between 55 and 160 with an average corrected N-value, adjusted to 300 mm penetration, of 101 blows. Standard Penetration Test N-values greater than 100 blows per 300 mm penetration are suspicious to the author and are regarded as being unrepresentative of the actual *in-situ* soil conditions. High SPT N-values, such as described above, are often an indicator of the presence of gravel, cobbles and or boulders. According to Weltman and Healy (1978), high N-values that result from encountering these materials can be ignored.

The penetration of the H-piles into the sand till layer ranged between about the top of the layer to 1.3 m into the layer. The sand till was described as dense to very dense at these depths. The $q_{b\text{-}ult}$ of the H-piles seated in the sand till ranged between 65 and 122 MPa with an average value of 97 MPa. The COV was calculated to be 0.21. Figure 4.10 provides the plot of $q_{b\text{-}ult}$ of the H-piles driven in sand till versus the corrected SPT N-value, and includes the Meyerhof method (1976) for comparison. The EOID conditions yielded higher values of $q_{b\text{-}ult}$ than BOR conditions which may indicate dilation at the pile toe during initial driving, resulting in false set-up. If this is the case, the lower BOR values of $q_{b\text{-}ult}$ would indicate soil relaxation over time.

No credibly strong correlations could be made between the corrected SPT N-value and the $q_{b\text{-}ult}$ of the H-piles with the limited amount of data. The reliability of the SPT-method to estimate the $q_{b\text{-}ult}$ of the H-piles is therefore inconclusive and not recommended for design.

4.4.4 Ultimate End Bearing Capacity and H-Pile Size

Each H-pile size was analysed individually to get a sense of the $Q_{b\text{-}ult}$ that can be expected when the piles are driven to bedrock and till. The Q_{ult} was not included here because the depth of penetration varies from site to site. Table 4.2 provides the minimum, maximum, and average measured values of $Q_{b\text{-}ult}$ for each H-pile size, and the $Q_{b\text{-}ult}$ as predicted by the conventional ULS design criteria defined by $0.6f_y'A_p$ and $0.75f_y'A_p$.

An interesting discovery was made upon review of the data presented in Table 4.2. The HP360x132 piles had an average $Q_{b\text{-}ult}$ less than the smaller HP310x132 piles. A reason for the lower average Q_{b-ult} may be due to inadequate hammer energy transferred during driving, resulting in the inability of the hammer to fully mobilize the Q_{b-ult}. Evidence of this event occurred at Site 6. Here, four HP360x132 piles were driven. Two of the piles were driven with an average maximum transferred energy of about 23 kJ resulting in an average Q_{b-ult} equal to 1990 kN. The other two piles were driven with an average maximum energy of about 36 kJ which developed an average Q_{b-ult} of about 3120 kN. Four piles of this size were driven at two other project sites and developed an average Q_{b-ult} of 3249 kN. The rock was particularly hard at one of the sites; and at the other site, the H-piles that exhibited higher capacities were driven with greater energy. It appears that the larger H-pile sizes require more transferred energy to fully mobilize the soil resistance at the pile toe than the smaller pile sizes. The PDA test coupled with CAPWAP analysis provides the ability to determine the energies required to mobilize the Q_{ult} of certain pile sizes. And of course, H-piles seated in weaker rock have lower Q_{b-ult}.

4.5 Load Transfer

The load transfer behavior of driven steel H-piles was investigated to determine the amount of applied load (via the hammer blow) that is resisted in the shaft and how much is transferred to the pile toe. This component was explored because the author believes that steel H-piles driven to bedrock are not entirely end bearing piles since shaft frictional resistance must be mobilized prior to the mobilization of the end bearing resistance. Ignoring the H-pile size and length and the testing condition, it was discovered that the amount of Q_{s-ult} of the sixty-four driven steel H-piles ranged between 9 and 61 percent of the Q_{ult} with an average of 26 percent. These statistics indicate two things; 1) H-piles are not entirely end bearing piles and 2) significant shaft friction resistance may potentially be developed along the shaft of steel H-piles making them partial friction piles. Of course, these values are dependent on many factors. However, this example provides insight into the potential amount of applied load that can be absorbed in the shaft of H-piles. Shaft frictional resistance is investigated in more detail in the following section.

4.6 Ultimate Shaft Friction Capacity: Evaluation and Refinement of Static RAMs

Static analysis of the $Q_{s\text{-}ult}$ of H-piles is a complex task. There are many variables that play a role in the development of frictional resistance along an H-pile shaft, including the size and embedment length of the pile, and the type and strength characteristics of the embedded strata. Pile size, embedment length, and laboratory and *in-situ* testing results were compared to the CAPWAP-calculated $Q_{s\text{-}ult}$ to assess load transfer and existing static RAMs to determine their applicability to driven steel H-piles in Winnipeg. The s_u of the lacustrine clay and the corrected SPT N-values and moisture contents of silt till were compared to the $f_{s\text{-}ult}$ values developed along the pile shafts. The static RAMs used to estimate the $f_{s\text{-}ult}$ of piles, which were discussed in Chapter 2, were reviewed and refined. The static RAMs presented below will be referred to as the refined static RAMs for estimating the $f_{s\text{-}ult}$ of driven steel H-piles in Winnipeg.

4.6.1 Ultimate Shaft Friction Capacity and Shaft Embedment Area

General analysis results of the load transfer along the H-pile shaft with respect to shaft embedment area are illustrated in Figures 4.11 and 4.12. The strength characteristics of the strata are ignored in this case for simplicity. It can be seen from the amassed data in Figure 4.11 that there is a slightly increasing trend of load transferred to the pile shaft with increased embedment area; however, the scatter in the data is large. In Figure 4.12, the data was sorted by H-pile size. It should be anticipated that H-piles with larger embedded area would result in more load transfer to the pile shaft. This was not necessarily the case upon review of Figure 4.12. The reason being is that the soil properties, which vary from site to site, were not factored into this particular analysis.

4.6.2 Ultimate Shaft Friction Capacity and Soil Strength

To bring the load transfer characteristics of soil into play, the values of $f_{s\text{-}ult}$ calculated by CAPWAP were back analysed and correlated to soil properties of lacustrine clay and till. The α-method for H-piles enveloped in clay was investigated. The SPT-method and moisture content method were examined for H-piles embedded in silt till. Piles in sand till were not investigated due to the intermittent layering along the pile shaft, making it difficult to discretize the location of the layers.

The $f_{s\text{-}ult}$ values for the driven steel H-piles were investigated without differentiation of soil type to get a sense of average values that can be expected in the typical Winnipeg soil stratigraphy. Ignoring the shaft frictional resistance developed in zones of fractured rock the $f_{s\text{-}ult}$ values ranged between about 8.3 and 59.9 kPa, with an average of 29.2 kPa, a standard deviation of 13.6 and a COV equal to 0.47. The high amount of variability found in the results is an indication of the varying soil thicknesses and frictional resistive strengths.

4.6.2.1 α-Method

The α-method is commonly used to estimate the $f_{s\text{-}ult}$ of piles in clay. The CAPWAP-calculated values of $f_{s\text{-}ult}$ were back-analysed and correlated to the laboratory-tested s_u, to refine the α-method to specifically estimate the $f_{s\text{-}ult}$ of driven steel H-piles in Winnipeg's lacustrine clays. The s_u values were based on UC, torvane and lab vane tests conducted on undisturbed Shelby tube samples in a laboratory setting. Measuring the s_u of the clay using these testing methods is common practice in Winnipeg. Triaxial shear tests are rarely used in Winnipeg.

Figure 4.13 provides a plot of α versus s_u. Included on the plot is the empirical α-method provided by Semple and Rigden (1984), which was discussed in Chapter 2, and the refined α-method based on the CAPWAP results. It can be seen that the refined α-factors are significantly lower than those determined by Semple and Rigden (1984) for firm to stiff clays. This may be because the CAPWAP results are based on the static resistance to driving and not long-term soil resistance, in which case full pile set-up had not occurred at the time of testing.

Another important observation to make from Figure 4.13 is the α-factor for the H-pile embedded in soft clay. The α-factor is greater than 1, in this case, which indicates that the clay may be quite sensitive to remoulding effects. It seems reasonable that soft sensitive clays conform to the pile surface creating a better contact than stiff clays. Generation of pore water pressure in soft clay during driving may nearly liquefy the soil causing it to behave more plastically and fill-in all the geometric details of the H-pile. If this is the case, then perhaps the shaft frictional resistance is not limited to the outside perimeter of the pile, but rather develops along the entire pile surface. Stiffer clays are not as malleable as soft clays and are assumed not to form the same contact to the inside of the pile during driving. Additionally, recovery of the stiff clays to their original state may take much longer and, if the pile is not re-tested to confirm, the remoulding effects may never be noticed; not to say they will not occur. In reality, the α-factors are not likely greater than 1, but appear to be, due to greater available surface area for frictional resistance and the significant remoulding effects of soft sensitive clays when compared to their *in-situ* state.

The PDA tests under BOR conditions were conducted between 2 and 7 days after EOID. No discernible difference of α-factors between the two testing conditions was evident. This may indicate that pile set-up had not occurred and the true long-term pile capacity had not developed. A second explanation could be that firm to stiff clays are not very sensitive to driving effects in terms of pore water pressure generation and remoulding effects due to the small cross-sectional area of H-piles and their low displacement characteristics. If this were the case, pile set-up may be minor, non-existent, or may take longer periods of time to develop. The second case may not be very reasonable considering set-up in clay is generally considered to be the norm with all piles, with the exception of very stiff clays that experience post-holing or gapping effects during driving. It is recommended to use the refined α-method to estimate the $f_{s\text{-}ult}$ of H-piles driven in lacustrine clay which are based on measured CAPWAP values of frictional resistance.

4.6.2.2 SPT-Method

The SPT is a common *in-situ* testing method conducted during geotechnical investigations for classifying the till and its strength characteristics in Winnipeg. Estimating the $f_{s\text{-}ult}$ for steel H-piles in silt tills using the SPT N-value is pragmatic for this reason. Judgment and experience played an important role in determining what N-values represent the strength of the till matrix. The presence of gravel and cobbles were acknowledged as these common till constituents can distort the SPT resulting in unrepresentative N-values. Standard penetration tests that resulted in split spoon samplers bouncing without penetration were recognized and ignored. Standard penetration tests that were terminated prior to the full 300 mm penetration were neglected from the analysis. N-values that were greater than 100 blows/300 mm or appeared to be unreasonably high when compared to moisture content were questioned. Caution was used with N-values greater than 60 blows/300mm. The N-values that were believed to be representative of the silt matrix without the influence of gravel or cobble sized material were used. Furthermore, the N-values were corrected for effective overburden pressure.

Figure 4.14 shows a plot of $f_{s\text{-}ult}$ versus the SPT N-value. Included on the plot are the refined SPT and Meyerhof (1976) methods. The refined SPT-method is based on CAPWAP-calculated values of $f_{s\text{-}ult}$ of the H-piles and the corrected N-values. The Meyerhof method is based on values of $f_{s\text{-}ult}$ for piles as determined by SLTs on piles seated in non-cohesive soils. The refined SPT-method and the Meyerhof method (1976) are compared. The refined SPT-method shows good correlation to the Meyerhof method. Upon reviewing Figure 4.14, three general conclusions for driven steel H-piles embedded in silt till can be made about the refined SPT-method: 1) higher SPT N-values result in greater $f_{s\text{-}ult}$ values, 2) a progressive linear relationship exists between the SPT N-value and $f_{s\text{-}ult}$, and 3) the Meyerhof method is an accurate RAM for estimating the $f_{s\text{-}ult}$ of H-piles on the basis of the SPT N-value. Based on the findings of the refined SPT-method it, or Meyerhof's method, can be used to estimate the $f_{s\text{-}ult}$ of driven steel H-piles in Winnipeg silt till.

4.6.2.3 Moisture Content Method

Moisture content is an excellent indicator of bearing pressure of silt till and is a design tool for many experienced geotechnical engineers. The dense tills have moisture contents in the range of 8 to 10 percent which often justify design end bearing pressures of 720 kPa for end bearing caissons. Very dense silt tills have lower moisture contents in the range of 3.5 to 8 percent which often justify design bearing pressures in the order of 1440 kPa. The author strived to determine if the same confidence might be developed for estimating the $f_{s\text{-}ult}$ for H-piles embedded in till.

The moisture contents in the silt till were based on split spoon samples and grab samples. Moisture contents that were believed to be unrepresentative of the surrounding silt till or tainted from groundwater seepage into the test hole were ignored. Figure 4.15 shows a plot of the CAPWAP-calculated $f_{s\text{-}ult}$ values of H-piles versus moisture content. The results of the analysis were not as favorable as anticipated. Although there is a trend in the data indicating higher $f_{s\text{-}ult}$ values in silt till of lower moisture contents, the data does not suggest a definitive correlation. The author is a believer in the legitimacy of moisture contents as strength indicators and would be interested, in the future, in correlating more CAPWAP-calculated $f_{s\text{-}ult}$ data of H-piles to moisture contents. The author considers the correlation inconclusive for the time being and would not suggest using the moisture content method to estimate the $f_{s\text{-}ult}$ of driven steel H-piles in Winnipeg silt till until more data is collected and analysed.

4.6.2.4 Ultimate Shaft Friction Capacity in Fractured Bedrock

Frictional resistance in highly fractured rock is difficult to dictate by *in-situ* testing or rock classification methods. Rock quality designation is particularly difficult to denote for a project site, due to the inherent variability and uncertainty of karst rock masses. Additionally, the driving process itself changes the *in-situ* state of the rock as a result of crushing and disturbance. The author believes that the best result one may conclude during a geotechnical investigation is determining the thickness of the fractured zone(s). H-piles driven through relatively thick zones of fractured rock in six of the twelve sites investigated in the Winnipeg case study produced CAPWAP-calculated values of $f_{s\text{-}ult}$. These values ranged between 155 and 300 kPa with an average equal to 228 kPa. Unfortunately, an UC test are not always performed, or cannot be performed, in fractured rock, and the spatial variability of RQD is so considerable that no reasonable correlation of $f_{s\text{-}ult}$ of H-piles can be made to these rock properties.

4.6.3 Shaft Capacity and End Bearing Strata

The end bearing material appears to have an influence on the amount of frictional resistance that develops along the shaft of driven steel H-piles. A comparison of the effect of the end bearing material on the load transfer characteristics was possible at two different sites where some piles had reached practical refusal in till and others reached practical refusal on bedrock.

A number of piles were driven to practical refusal in both till and bedrock at Sites 7 and 12. Eight steel HP310x110 piles were driven at Site 7; seven of the piles reached practical refusal in cobbly bouldery silt till at depths ranging between 17.9 and 21.1 m and one pile was driven to 32.2 m to bedrock. The H-piles that refused in the silt till developed an average of 40 percent of the Q_{ult} from shaft friction. The one H-pile that was driven to bedrock, which is about 11 m deeper than the other seven piles, developed 23 percent of its Q_{ult} from shaft friction.

A similar situation arose at a particular structure at Site 12. Two H-piles were driven to practical refusal in sand till at an average depth of 24 m, and three piles were driven to practical refusal on bedrock at depths ranging between 29.5 to 30 m. The two piles seated in the sand till developed an average of 38 percent of the Q_{ult} from shaft friction, whereas the three piles driven to bedrock developed an average of 28 percent of the Q_{ult} from shaft friction, despite being driven 6 m deeper.

The differences in the Q_{s-ult} (described above) can be observed in the dynamic behavior, which will be discussed in greater detail in Chapter 5. Greater penetration resistance was observed during driving of the H-piles seated in bedrock. The H-pile driven to bedrock at Site 7 had a penetration set of 1 mm compared to an average of 2 mm for the H-piles seated in the till for the same driving energy. The H-piles driven to bedrock at Site 12 had an average penetration set of 1 mm in contrast to 1.3 mm for the H-piles seated in the till for the same hammer fuel setting. The greater penetration resistance (lower penetration set) of the H-piles driven to bedrock may have inhibited full mobilization of the frictional resistance of the soil along the H-pile shaft.

Another possible explanation could be a result of bending or whipping action during driving. When the piles are seated on hard material and are driven to high penetration resistances, they may exhibit more bending than if driven to low penetration resistances in a weaker soil. Bending and whipping causes lateral deflections of the H-pile and disturbance of the surrounding soil which can lead to "soil-gapping" or increased generation of pore water pressure.

A third possible explanation of the higher $f_{s\text{-}ult}$ of H-piles seated in till could be that applied load tends to be transferred to stronger and stiffer material. The load is more evenly distributed if the end bearing material is not significantly stronger than the soil along the shaft. In response to the weaker end bearing material, excess load that is transferred to the pile toe is redistributed along the pile shaft. When relatively strong soil or rock resides at the pile toe, with respect to the material along the shaft, more of the applied load is transferred to the pile toe, as described by Tomlinson (2008).

4.7 Driveability in Till

The driveability of steel H-piles was assessed based on interesting behavior that occurred at Site Number 7. Some of the H-piles at this site reached practical refusal in the silt till at penetration depths ranging between 6.8 and 9.6 m at one structure's location, and 11.2 to 13.9 m at another structure's location. A single pile of the same size using the same hammer and energy located at the second structure was driven 20.2 m through the silt till to bedrock, almost twice the depth as the other piles. It was assumed that the piles setting up in the till had refused on large granite boulders or deposits of frequent cobbles and boulders that were discovered in the geotechnical investigation and confirmed during construction of rock socketed caissons nearby. In many cases, driven steel H-piles have the slender shape and material strength to thread through these types of layers. However, in this particular case great difficulty in doing so was encountered.

It appears that H-piles generally have good penetration proficiency and are capable of penetrating through thick layers of silt till, provided large or frequent boulders are not present. A majority of the steel H-piles investigated fully penetrated the till layers and reached the bedrock. The thickness of the till layers at the project sites ranged between about 0.2 and 20.2 m with an average thickness of about 6.4 m.

Some of the H-piles refused in layers of sand till. The penetration of the H-piles into the sand till layer ranged from the top of the layer to about 1.3 m into the layer. The preamble to this behavior was observed during the geotechnical investigation. Full penetration of the split spoon samplers did not occur for a majority of the SPTs conducted in this layer, resulting is very high and unrepresentative N-values. Thick sand till layers are infrequent in Winnipeg and require special attention when encountered.

The above situation at Site 7 provides a prime example of the importance of the PDA test and CAPWAP analysis. The Q_{ult} was unknown in this case, since the bedrock could not be reached for a majority of the H-piles. Pile Driving Analyzer testing and CAPWAP analysis was performed to measure the actual ultimate capacities (Q_{b-ult} and Q_{s-ult}). It was discovered that the H-piles did not particularly have high Q_{b-ult}; however, enough Q_{s-ult} had developed along the shaft to compensate for the reduced Q_{b-ult}, and the required Q_{ult} was achieved to satisfy the engineers.

4.8 Chapter 4 Discussion

Some key points based on the outcome of the static analyses should be made to end this chapter. First of all, the Q_{ult} of driven steel H-piles is not based on the structural strength of steel. This was determined by the variability in the Q_{ult} and Q_{b-ult}, and the relationships discovered between the measured soil and rock strength properties and the measured ultimate capacities. Second, driven steel H-piles are not entirely end bearing piles. In all cases, shaft friction resistance developed in the H-piles, and in most cases, the contribution of shaft friction resistance to the Q_{ult} was significant. Third, the q_{b-ult} of driven steel H-piles in Winnipeg can be estimated from the measured data presented in Sections 4.3 and 4.4 along with existing RAMs presented that were based on other's research. The refined RAMs in these sections are not recommended to calculate q_{b-ult} of driven steel H-piles, due to the limited amount of soil and rock strength data. The f_{s-ult} of driven steel H-piles in Winnipeg can be estimated using the measured data and refined static RAMs presented in Section 4.6.

Developing empirical relationships between the CAPWAP-calculated $q_{b\text{-}ult}$ and $f_{s\text{-}ult}$ values and the laboratory and *in-situ* test results, for the purpose of estimating the ultimate capacities of driven steel H-piles is practical. The laboratory measurements of s_u, moisture content, and q_u, along with the *in-situ* SPT N-values are commonly and easily determined during geotechnical investigations. Testing methods, such as the Pressuremeter, are time consuming and expensive. Therefore, pragmatic correlations to field and laboratory tests are more attractive methods for estimation of the Q_{ult} of an H-pile.

Rock quality designation was not incorporated in the analyses, because discontinuities of karst bedrock formations are very difficult to map. The frequency and size of voids, joints, and fractures can vary greatly within a short distance. Therefore, spatial variability is impossible to predict from only a few small diameter test holes. Even large diameter test holes cannot guarantee reasonable geological mapping. Complex theoretical methods did not seem practical for estimating the capacity of H-piles. These methods would likely lead to erroneous results. H-piles have a high resistance to driving stress, which allows for the piles to be repeatedly driven, potentially crushing the rock and altering its *in-situ* state. However, compressive failure and hence the q_u of the rock are presumed to appropriately suit the driving characteristics of driven steel H-piles in Winnipeg.

Table 4.1: The CAPWAP analysis data required to perform the static analyses.

Site	Test Condition	Blow No.	Observed Set	Final Pile Penetration	Pile Type	Ultimate Capacity	Ultimate End Bearing	Ultimate Shaft Capacity	End Bearing Strata
			mm	m		kN	kN	kN	
1	EOID	345	1.0	15.8	HP310x110	2853	1900	953	BEDROCK
	EOID	231	1.4	14.0	HP310x110	2976	2202	774	BEDROCK
	BOR	8	0.6	12.6	HP310x132	3501	2958	543	BEDROCK
	BOR	8	1.0	12.4	HP310x132	3300	2593	707	BEDROCK
	BOR	6	1.0	13.3	HP310x132	3302	2932	370	BEDROCK
	BOR	5	1.4	13.3	HP310x94	2871	2278	593	BEDROCK
2	EOID	1559	0.6	14.0	HP200x54	1614	836	778	BEDROCK
	EOID	247	1.0	12.3	HP360x132	2862	1663	1198	BEDROCK
	EOID	312	0.8	17.8	HP360x132	2302	1567	735	BEDROCK
	EOID	33	2.0	14.3	HP360x132	2381	1449	932	BEDROCK
	BOR	14	0.4	10.5	HP360x132	4325	2888	1437	BEDROCK
	BOR	14	0.2	10.5	HP360x132	4037	3684	353	BEDROCK
3	BOR	23	1.4	12.8	HP310x110	2332	1661	671	BEDROCK
4	BOR	11	1.0	5.5	HP310x110	2405	2155	250	BEDROCK
	BOR	27	0.5	6.9	HP310x110	2543	2254	289	BEDROCK
	BOR	24	1.0	5.3	HP310x110	2907	2587	321	BEDROCK
5	EOID	10	2.3	10.8	HP310x110	2243	1685	558	BEDROCK
	EOID	33	1.9	10.8	HP310x110	3104	1195	1909	BEDROCK
	BOR	10	2.0	11.7	HP310x110	2419	1765	653	BEDROCK
	BOR	15	2.0	11.4	HP310x110	2414	1332	1082	BEDROCK
	BOR	16	2.0	12.5	HP310x110	2101	1360	742	BEDROCK
6	EOID	231	1.3	4.0	HP360x132	2537	2200	337	BEDROCK
	EOID	125	1.5	6.6	HP360x132	2348	1776	573	BEDROCK
	BOR	20	2.6	6.6	HP360x132	1964	1587	377	BEDROCK
	BOR	4	1.0	6.4	HP360x132	3482	3192	290	BEDROCK
	BOR	3	2.0	6.4	HP360x132	3419	3233	186	BEDROCK
7	BOR	14	1.6	20.1	HP310x110	2514	1400	1113	SILT TILL
	BOR	15	2.0	19.6	HP310x110	2267	1137	1130	SILT TILL
	BOR	18	2.4	21.1	HP310x110	2254	1395	859	SILT TILL
	BOR	27	6.0	18.3	HP310x110	1860	1023	837	SILT TILL
	BOR	9	1.8	19.7	HP310x110	2217	1476	741	SILT TILL
	BOR	7	2.0	18.2	HP310x110	2094	1355	739	SILT TILL
	BOR	8	2.2	17.9	HP310x110	2095	1390	704	SILT TILL
	BOR	18	1.0	32.2	HP310x110	2384	1835	549	BEDROCK
8	EOID	172	0.9	12.2	HP310x110	3420	2570	851	BEDROCK
	BOR	6	0.7	12.4	HP310x110	3350	2503	847	BEDROCK
	BOR	10	1.0	11.5	HP310x110	2803	2135	668	BEDROCK
	BOR	9	1.0	11.2	HP310x110	3232	2480	752	BEDROCK
9	EOID	575	0.6	15.4	HP310x125	2464	2039	425	BEDROCK
	BOR	22	1.0	15.0	HP310x125	2342	2005	337	BEDROCK
	BOR	5	1.0	14.3	HP310x125	2520	2025	494	BEDROCK
	BOR	6	2.0	16.1	HP310x125	2410	2055	355	BEDROCK

10	EOID	235	2.1	19.0	HP310x132	3001	1901	1100	BEDROCK
11	EOID		1.7	20.3	HP310x132	2901	2541	360	BEDROCK
	EOID		1.0	20.3	HP310x132	3201	2801	400	BEDROCK
12	EOID	122	1.3	19.9	HP310x125	2130	1586	544	SAND TILL
	EOID	137	1.0	20.9	HP310x125	2166	1671	495	BEDROCK
	EOID	1324	1.7	31.6	HP310x125	2783	2363	419	BEDROCK
	EOID	1432	1.3	31.5	HP310x125	2525	2184	341	BEDROCK
	EOID	76	0.9	29.8	HP310x125	2724	1840	883	BEDROCK
	EOID	42	1.0	29.5	HP310x125	2668	2145	523	BEDROCK
	EOID	201	0.8	22.9	HP310x125	2363	1737	626	SAND TILL
	EOID	133	1.4	22.5	HP310x125	2457	1942	515	SAND TILL
	EOID	527	2.1	20.9	HP310x125	3157	2850	307	BEDROCK
	EOID	53	1.0	20.9	HP310x125	2450	2239	211	BEDROCK
	BOR	4	1.0	30.0	HP310x125	2267	1637	629	BEDROCK
	BOR	4	1.2	24.0	HP310x125	1737	1037	700	SAND TILL
	BOR	9	1.4	24.0	HP310x125	1850	1189	662	SAND TILL
	BOR	22	0.8	22.5	HP310x125	2171	1631	540	SAND TILL
	BOR	15	0.3	21.1	HP310x125	3166	2625	541	BEDROCK
	BOR	3	0.6	20.9	HP310x125	3057	2650	407	BEDROCK
	BOR	4	0.6	21.3	HP310x125	2825	2228	597	BEDROCK
	BOR	9	0.7	22.3	HP310x125	2746	2168	578	BEDROCK
	BOR	29	0.9	29.7	HP310x125	2750	2190	560	BEDROCK

Table 4.2: Measured $Q_{b\text{-}ult}$ and values of $Q_{b\text{-}ult}$ predicted by the conventional ULS design criteria for different H-pile sizes.

H-Pile Size	Number of Piles	End Bearing Stratum	$Q_{b\text{-}ult}$			Con. ULS Design Criteria	
			Minimum	Maximum	Average	$0.6f_y'A_p$	$0.75f_y'A_p$
			kN	kN	kN	kN	kN
HP200x54	1	Bedrock	836	836	836	1436	1796
HP310x94	1	Bedrock	2278	2278	2278	2500	3125
HP310x110	16	Bedrock	1195	2587	1976	2962	3702
	7	Silt Till	1023	1476	1311		
HP310x125	16	Bedrock	1637	2850	2203	3340	4175
	7	Sand Till	1037	1942	1542		
HP310x132	6	Bedrock	1901	2958	2621	3508	4385
HP360x132	10	Bedrock	1449	3684	2324	3528	4410

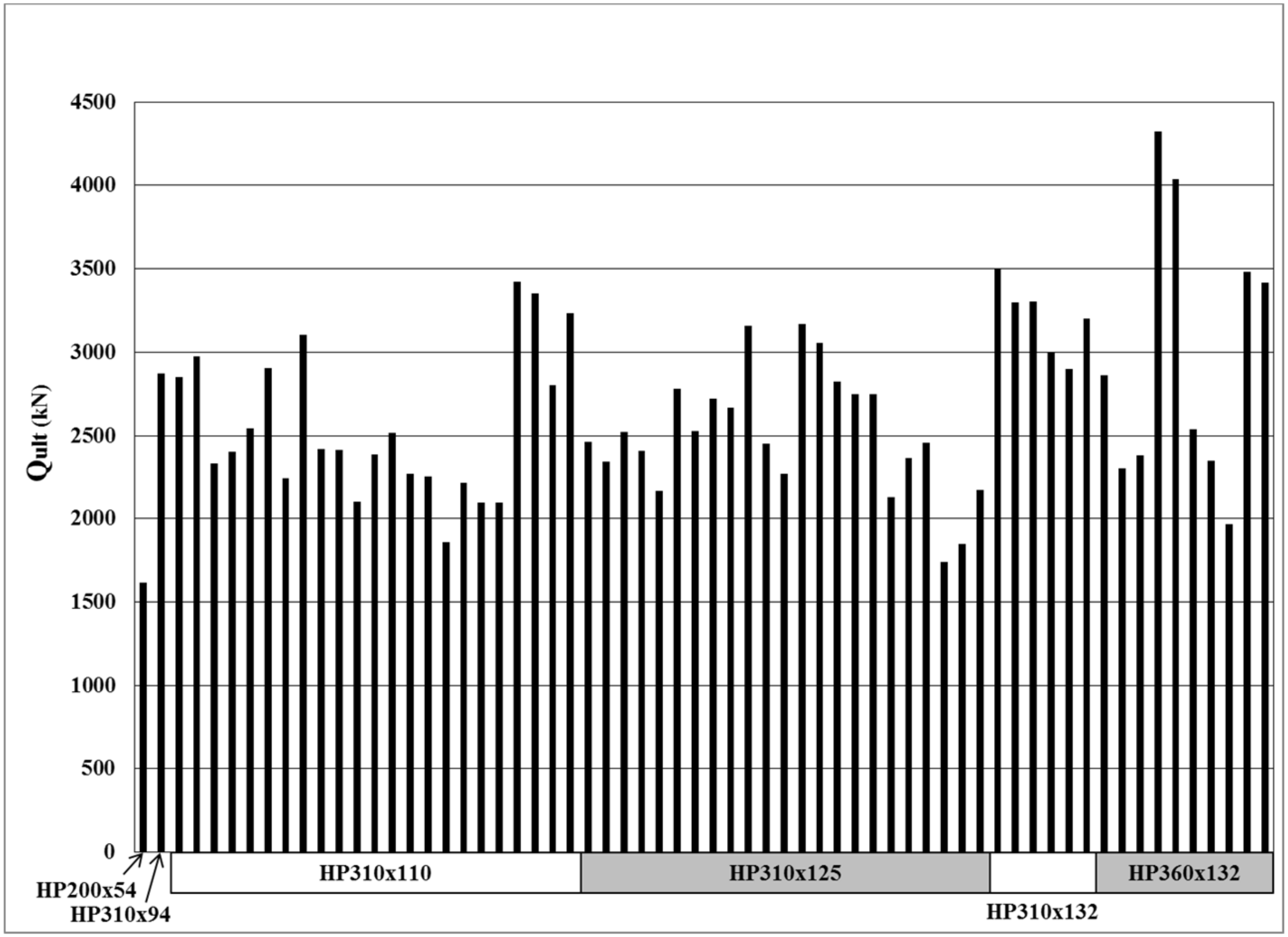


Figure 4.1: Measured Q_{ult} of driven steel H-piles in Winnipeg. Sorted by pile size.

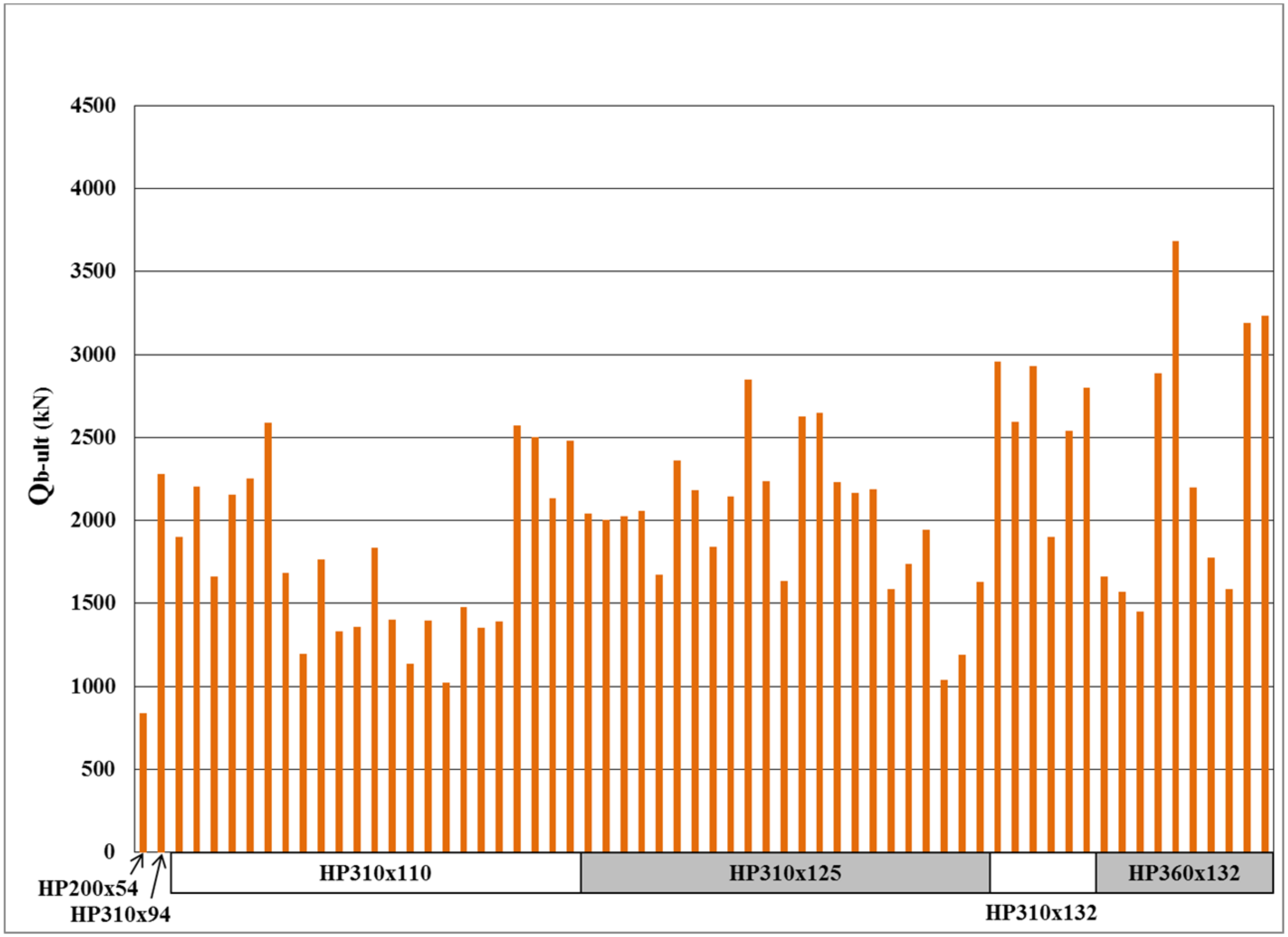


Figure 4.2: Measured $Q_{b\text{-}ult}$ of driven steel H-piles in Winnipeg. Sorted by pile size.

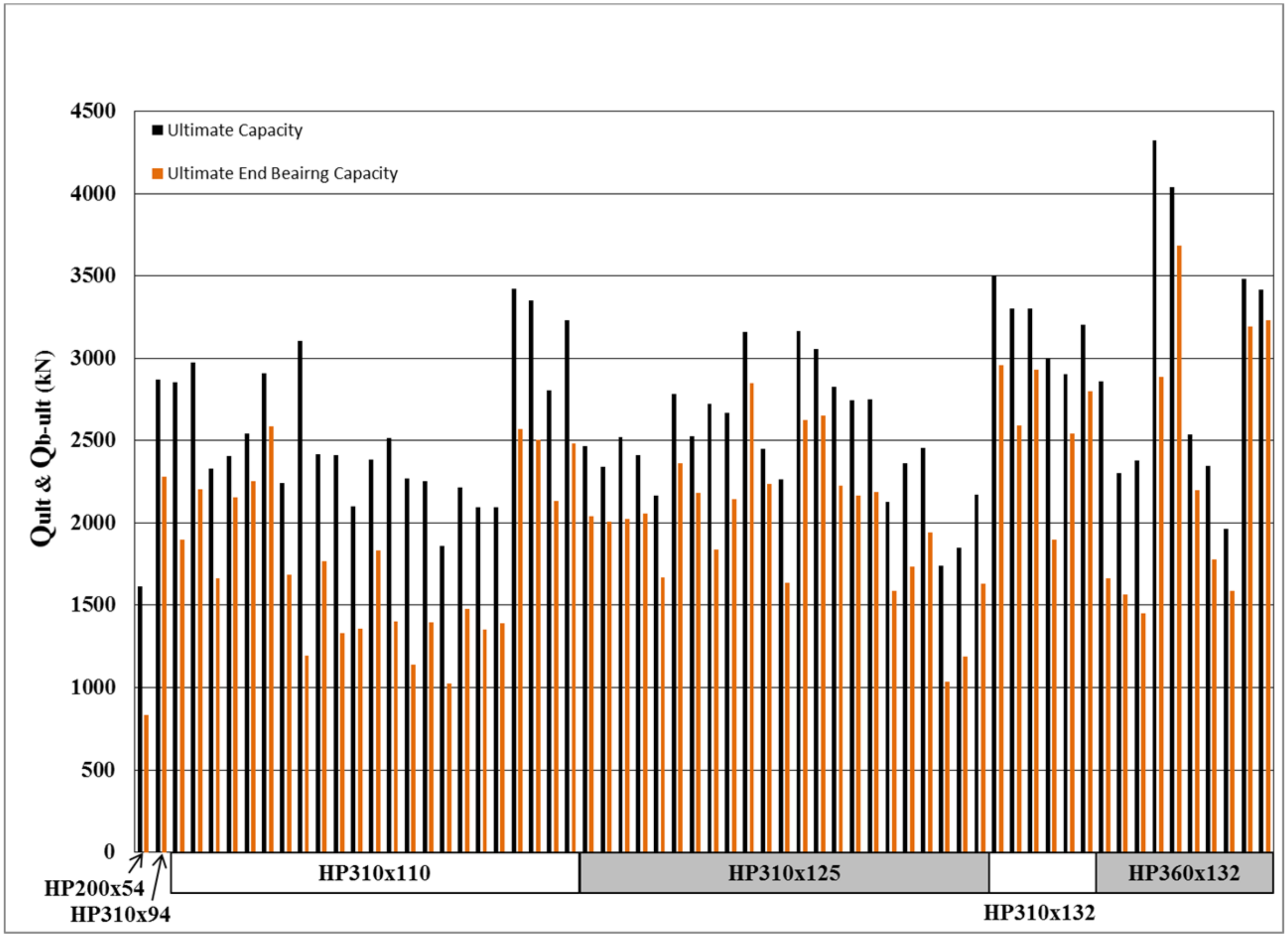


Figure 4.3: Measured Q_{ult} and $Q_{b\text{-}ult}$ of driven steel H-piles in Winnipeg. Sorted by pile size.

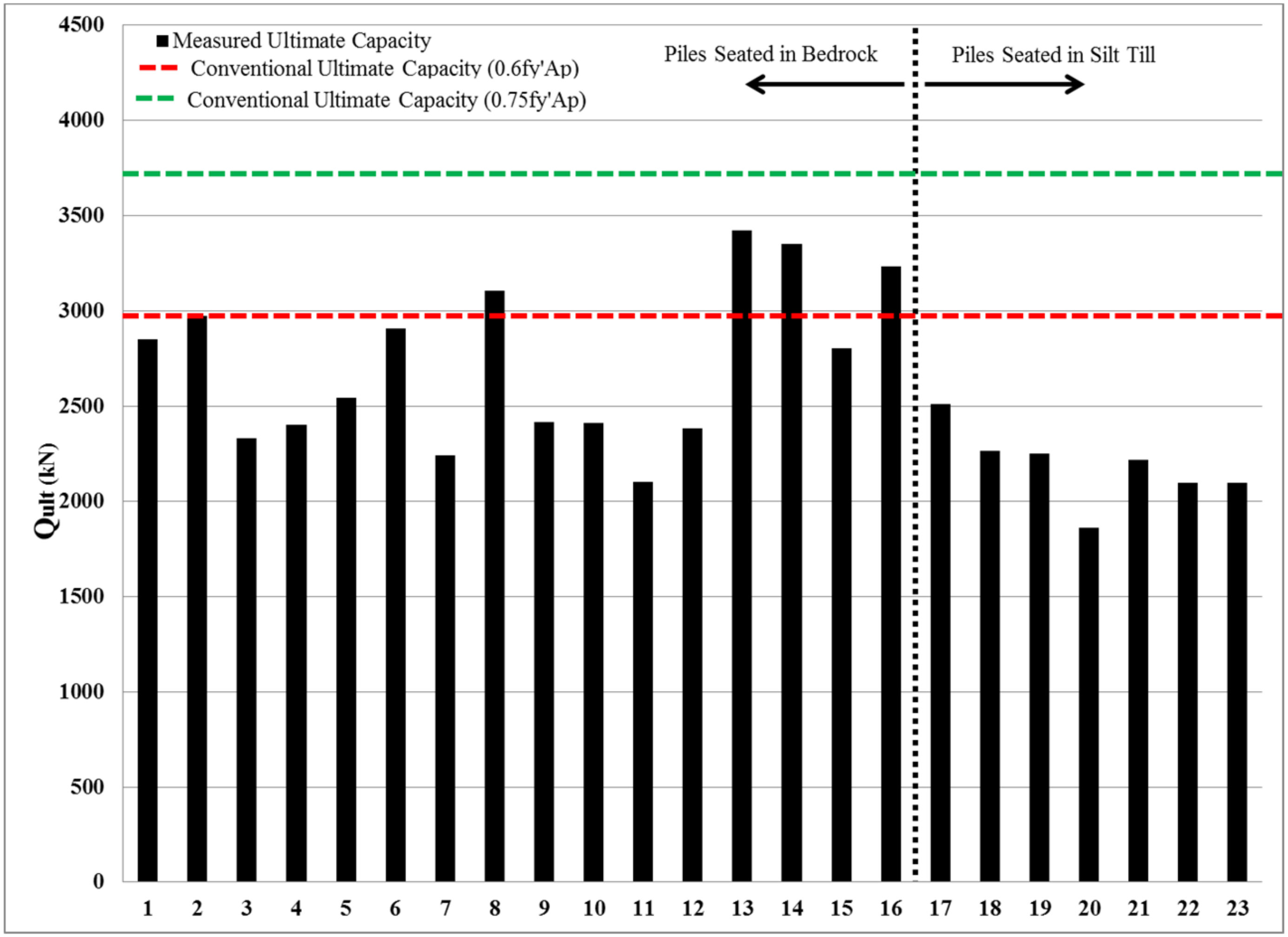


Figure 4.4: The Q_{ult} measured by CAPWAP and the Q_{ult} predicted by the conventional LSD criteria for the HP310x110 sections.

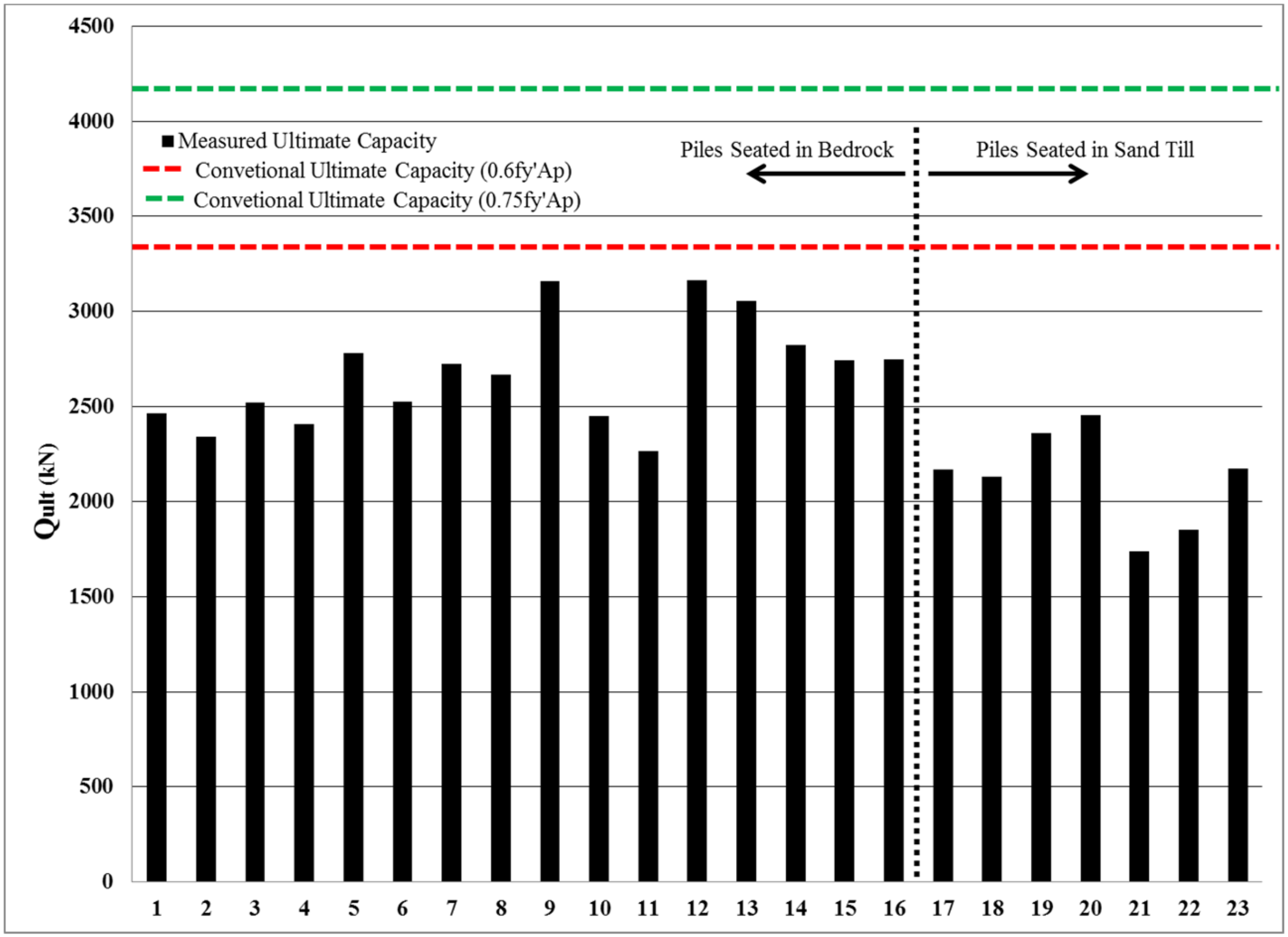


Figure 4.5: The Q_{ult} measured by CAPWAP and the Q_{ult} predicted by the conventional LSD criteria for the HP310x125 sections.

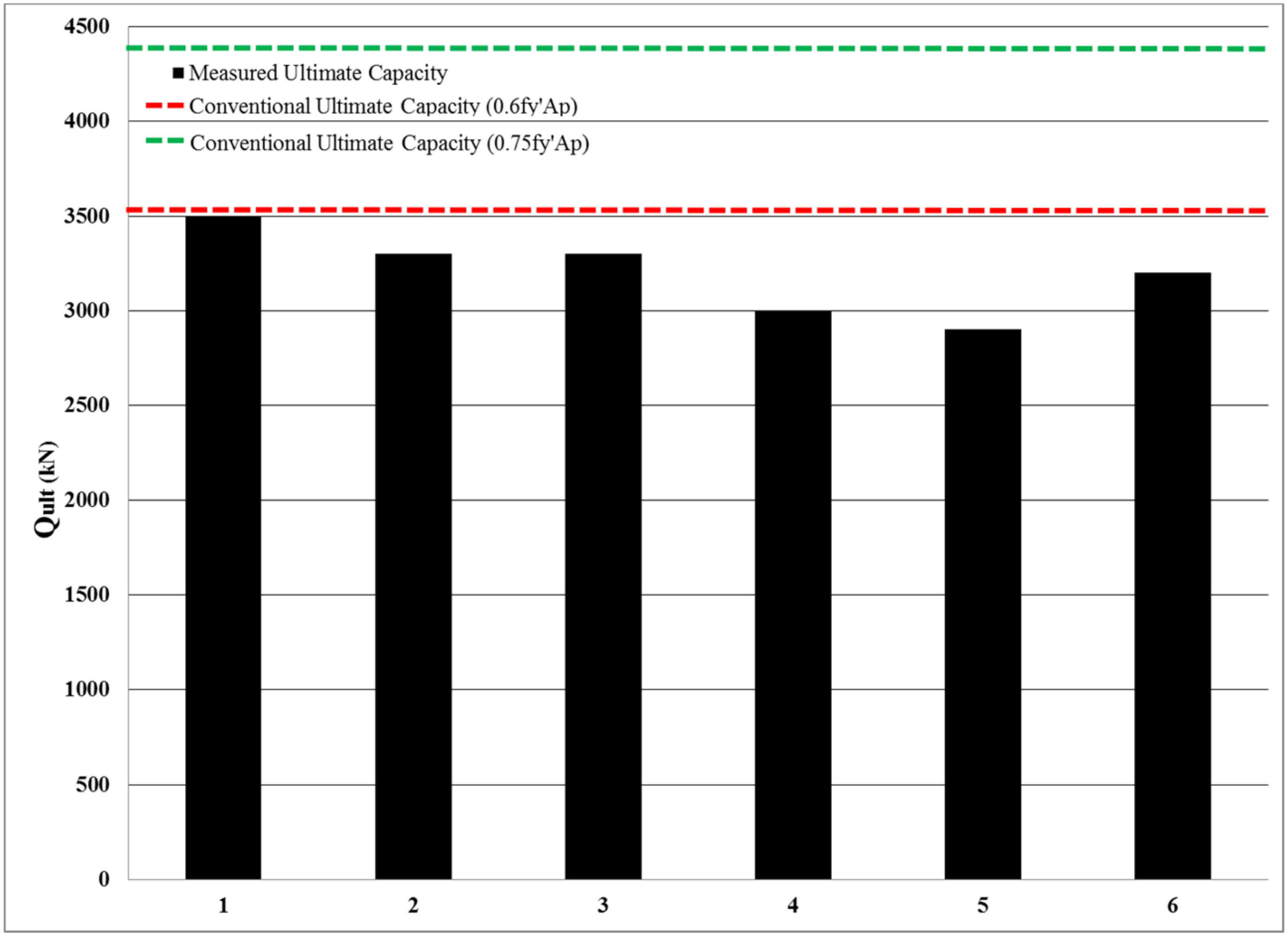


Figure 4.6: The Q_{ult} measured by CAPWAP and the Q_{ult} predicted by the conventional LSD criteria for the HP310x132 sections.

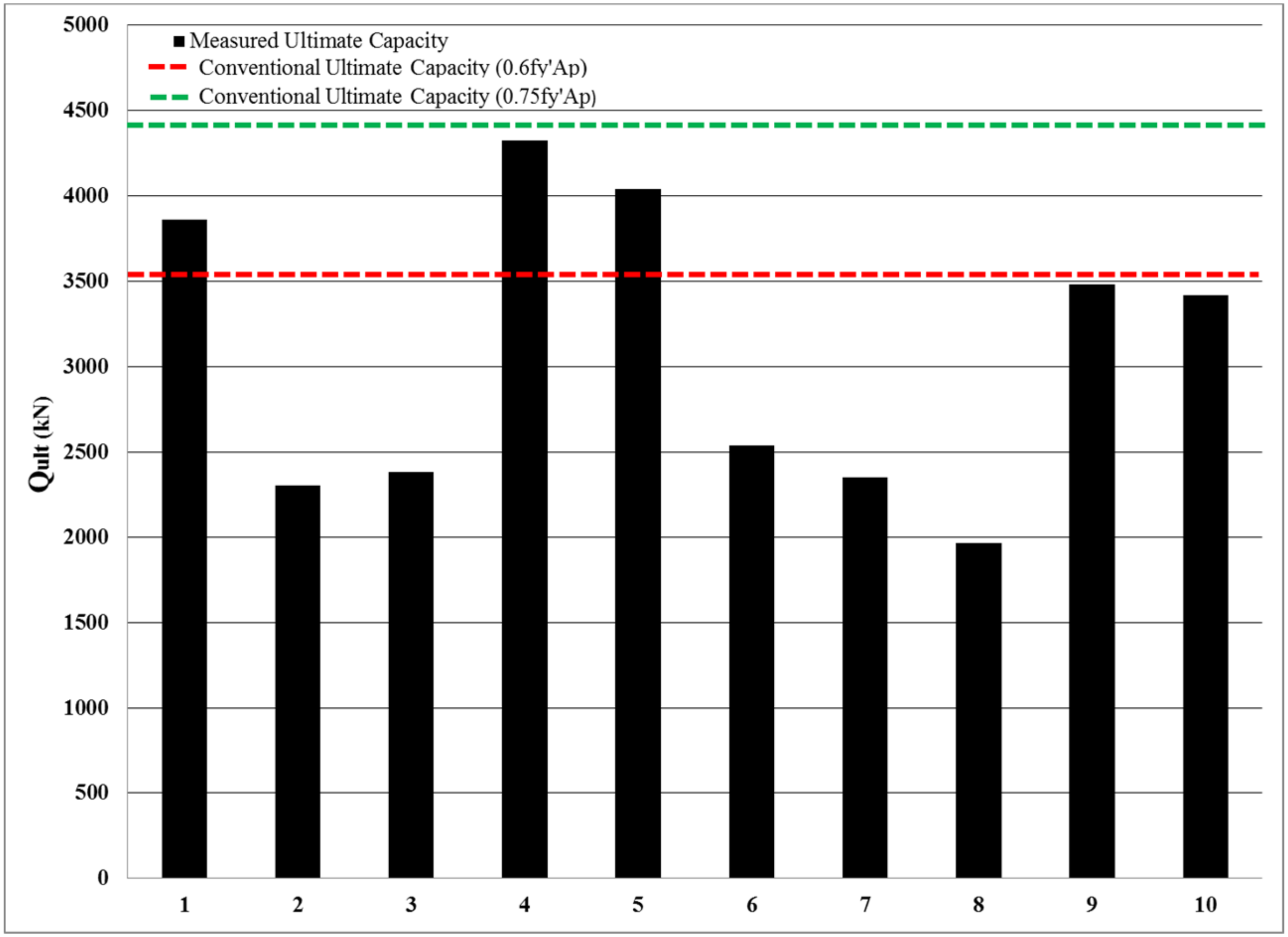


Figure 4.7: The Q_{ult} measured by CAPWAP and the Q_{ult} predicted by the conventional LSD criteria for the HP360x132 sections.

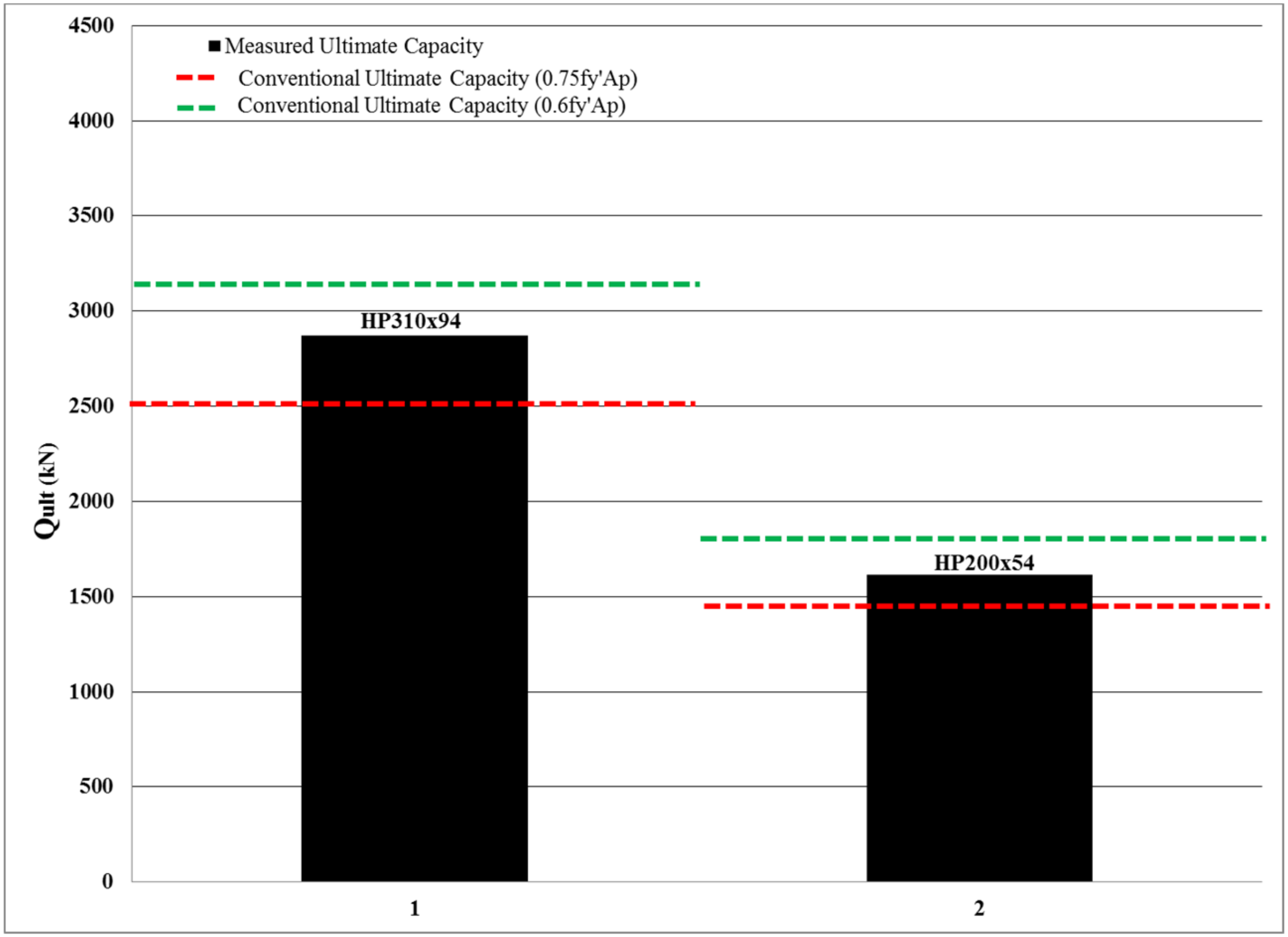


Figure 4.8: The Q_{ult} measured by CAPWAP and the Q_{ult} predicted by the conventional LSD criteria for the HP310x94 and HP200x54 sections.

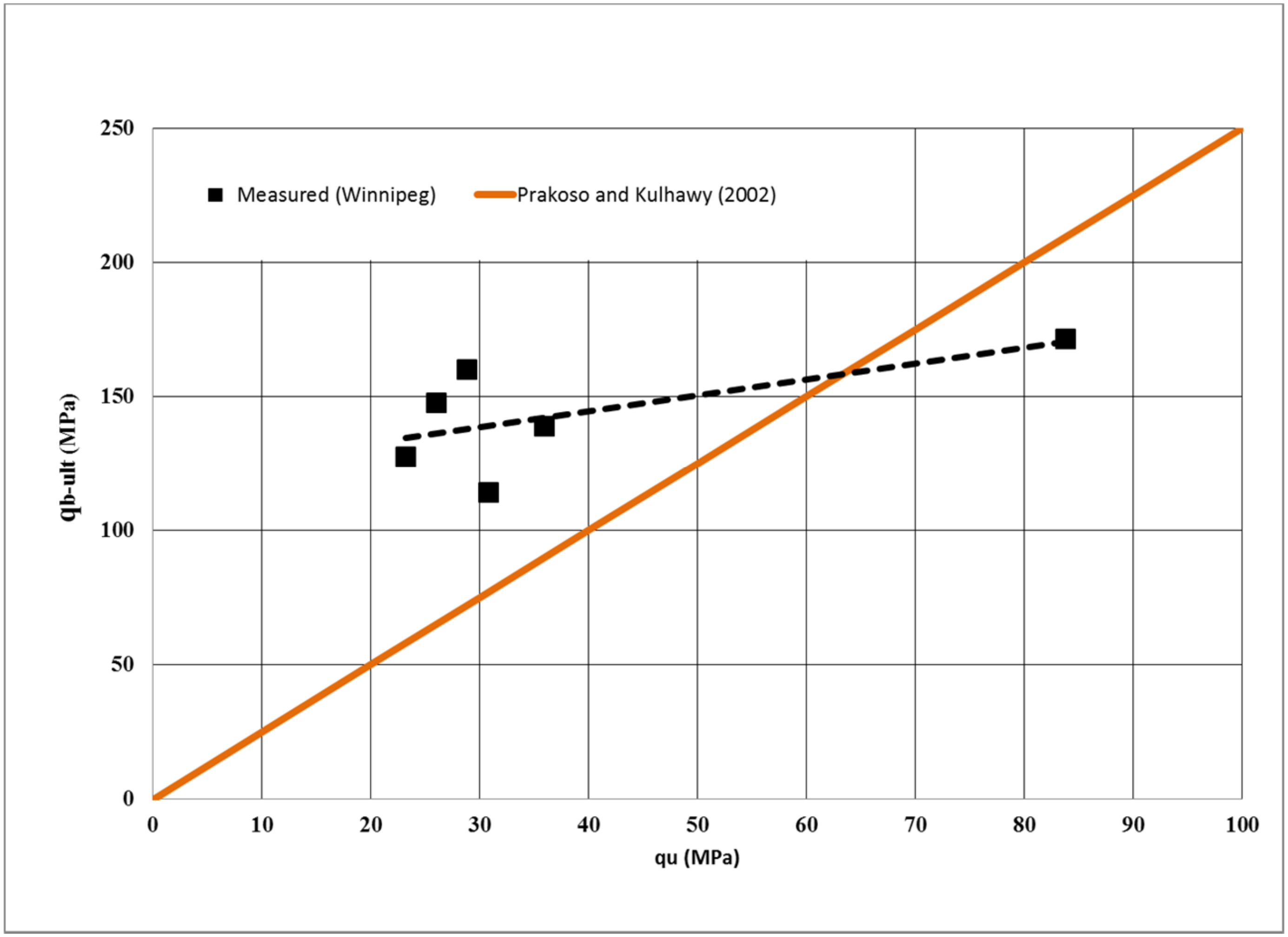


Figure 4.9: The Q_u method. The $q_{b\text{-}ult}$ of the H-piles versus q_u of bedrock.

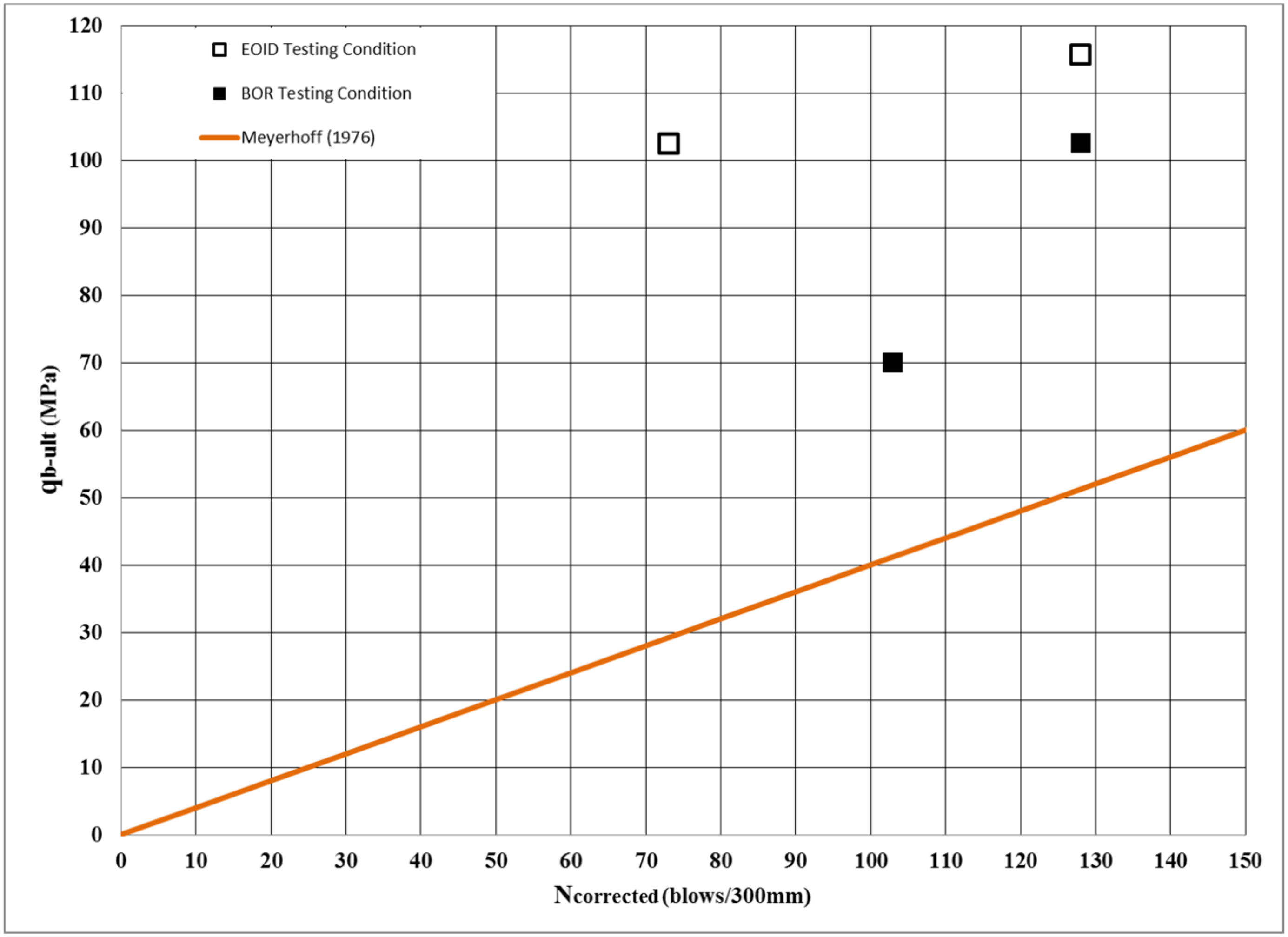


Figure 4.10: The SPT Method. The $q_{b\text{-}ult}$ of H-piles versus SPT corrected N-value in sand till.

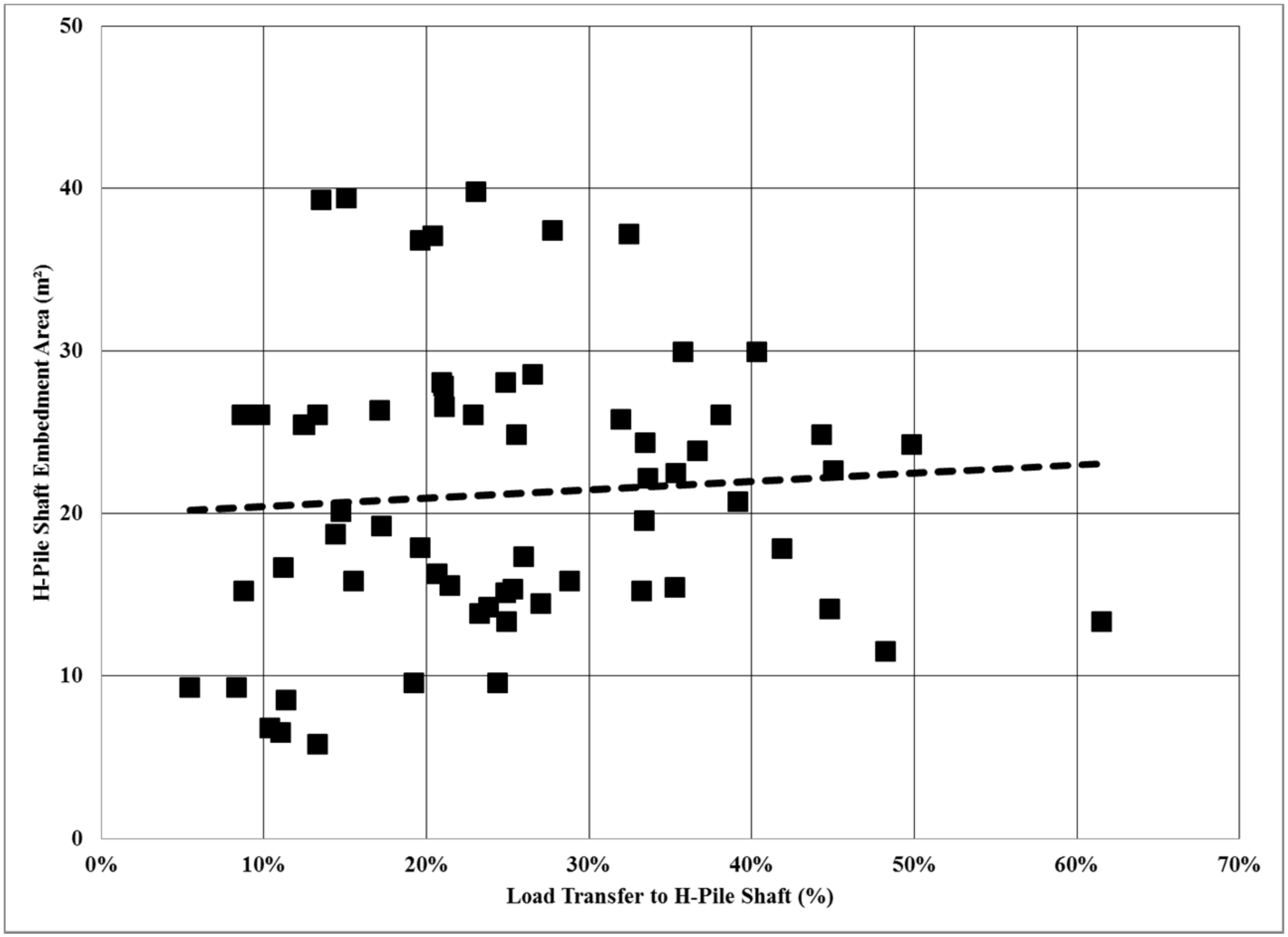


Figure 4.11: Load transfer to H-pile shaft versus H-pile shaft embedment area.

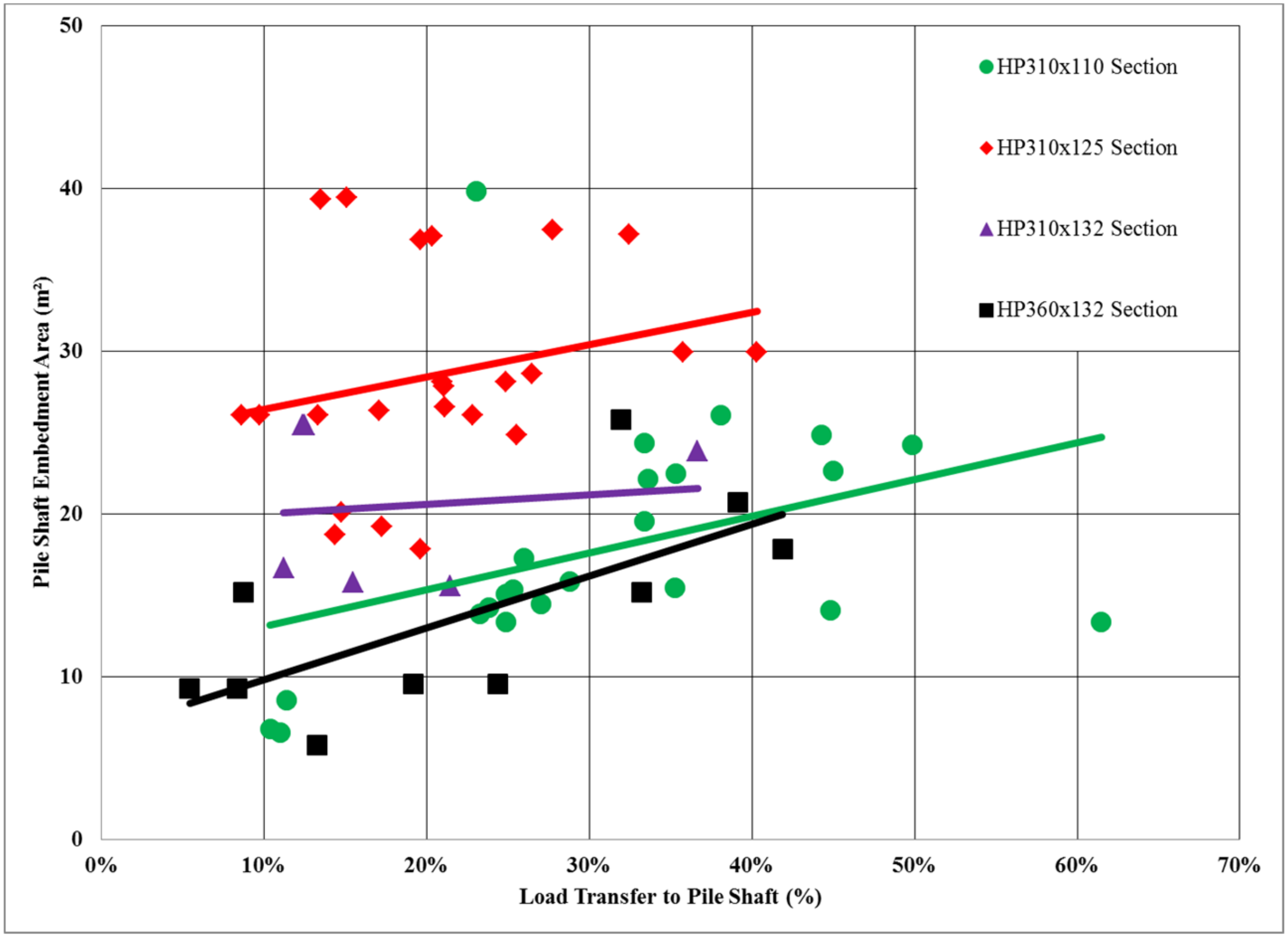


Figure 4.12: Load transfer to H-pile shaft versus embedment. Sorted by pile size.

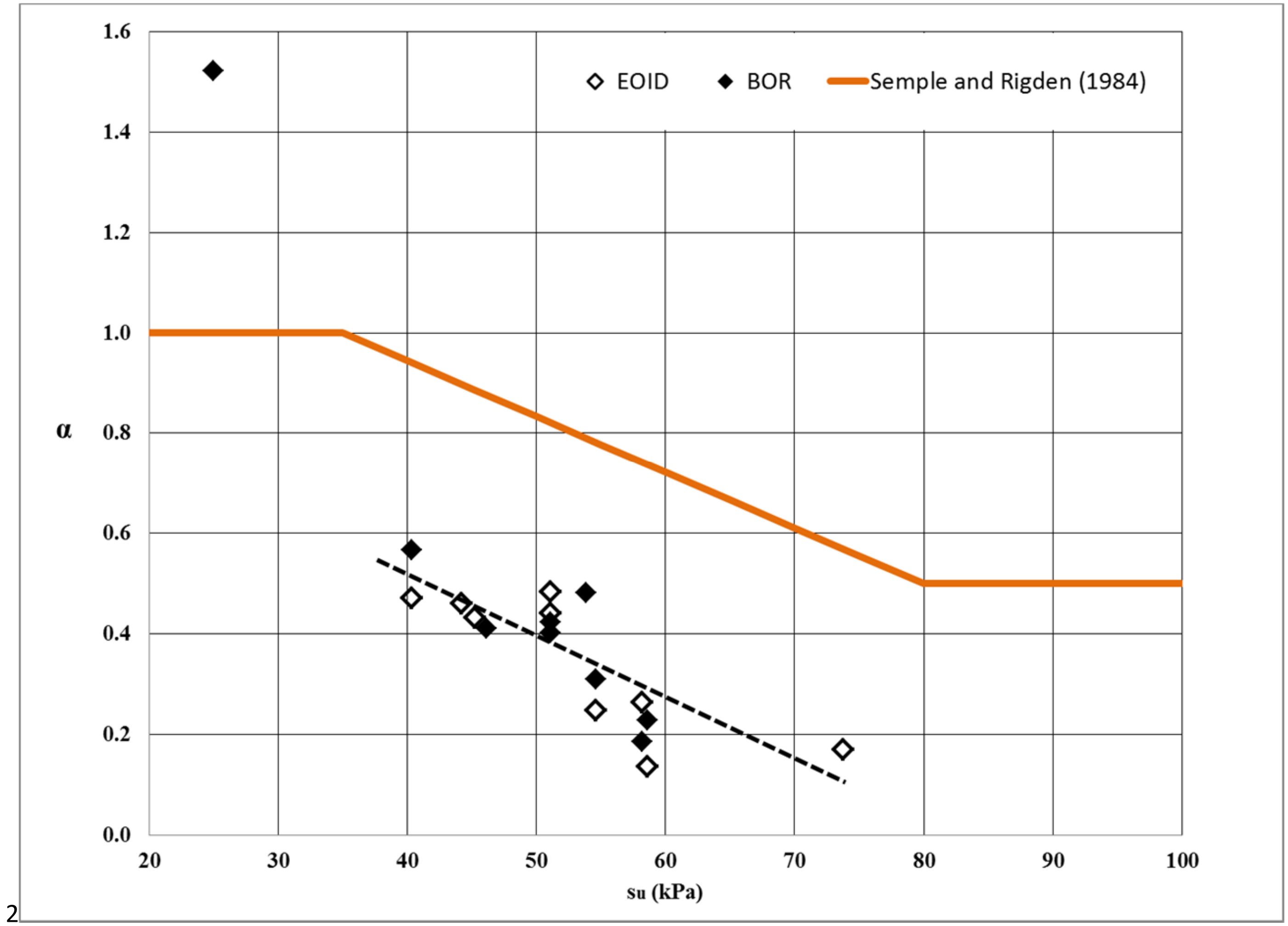


Figure 4.13: The α-method. The α-factor of H-piles versus s_u of lacustrine clay.

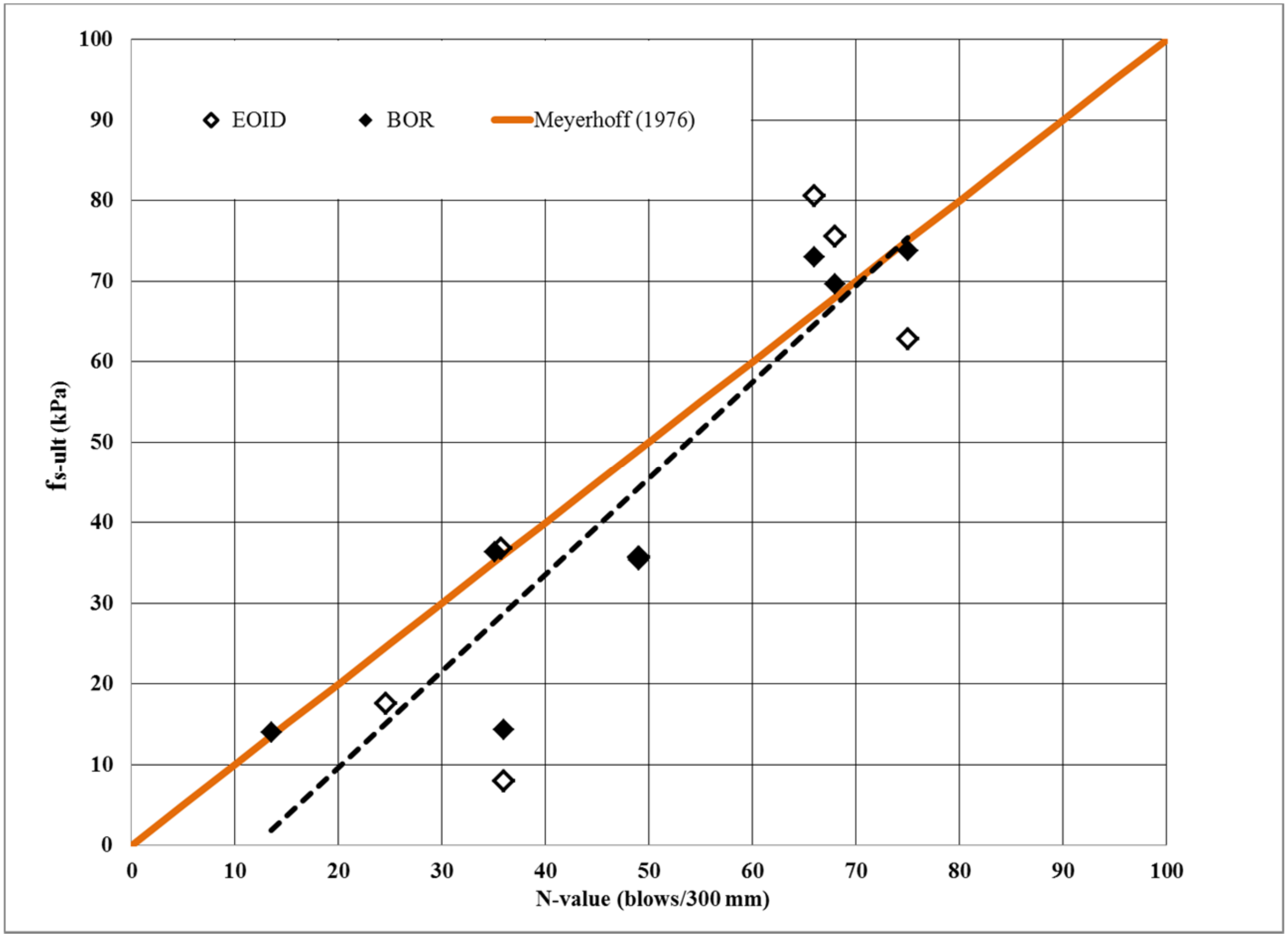


Figure 4.14: The SPT Method. The $f_{s\text{-}ult}$ of H-piles versus corrected SPT N-value in silt till.

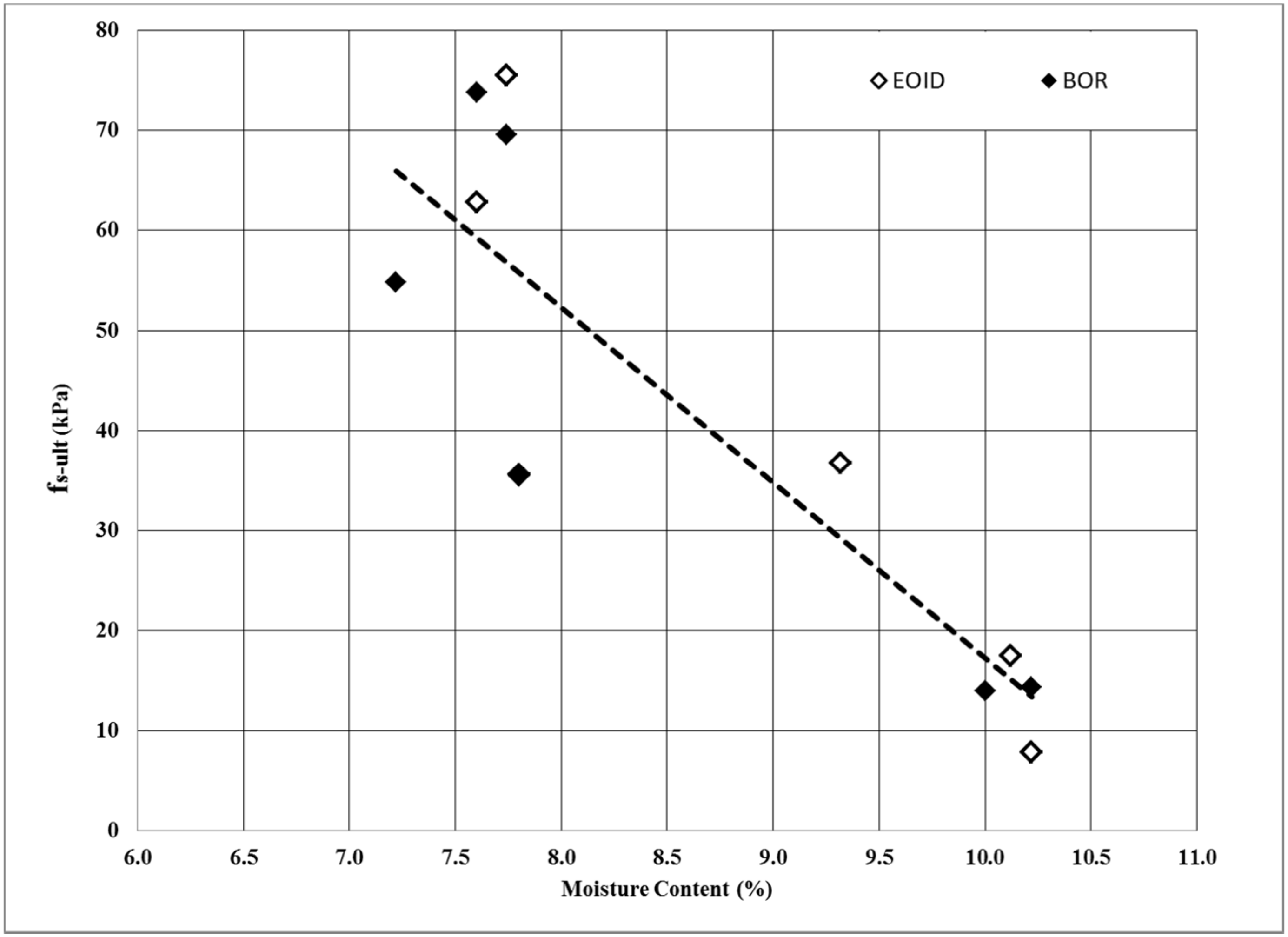


Figure 4.15: The Moisture Content Method. The $f_{s\text{-}ult}$ of H-piles versus moisture content in silt till.

Chapter 5: Dynamic Analysis of Driven Steel H-Piles in Winnipeg

5.0 Introduction

This chapter present the dynamic analyses conducted on the sixty-four test piles. There are two main sections of this chapter. The first section includes a description of the development of general multi-variable bearing graphs that encompass all the CAPWAP-calculated capacities. The bearing graphs show the variables that influence Q_{ult} and Q_{b-ult}, including transferred energy, end bearing strata, testing condition, and pile size.

In the second part of this chapter, the Q_{ult} determined by the dynamic RAMs, including the modified Gates formula, WEA, and the Case method, are compared to the CAPWAP-calculated capacities in a correlation study to assess their accuracy and variability. Dynamic analyses using these RAMs were conducted at each project site using recommended input properties and the results are plotted against the CAPWAP results. Improvement in the accuracy and variability of the dynamic RAMs was completed by using CAPWAP-calculated values of the dynamic input variables used for the RAMs.

5.1 General Bearing Graph

General bearing graphs, shown in Figures 5.1 to 5.8, were constructed using the CAPWAP analysis results and piling records from sixty-four driven steel H-piles. The purposes of the bearing graphs are to understand how different dynamic variables (*e.g.* transferred energy and penetration set) affect driven H-pile capacity and to provide a tool (dynamic RAM) for estimating the refusal criterion for these piles when driven to bedrock and till. Three integral variables were incorporated into the bearing graphs, including the pile penetration per hammer blow (pile set), Q_{ult} (or Q_{b-ult}), and the transferred hammer energy. These three variables are dependent on one another and must all be included in the bearing graph to properly represent pile driving behavior and the related capacity. Other variables that are included in the bearing graphs that influence the Q_{ult} and Q_{b-ult} are the end bearing stratum, testing condition, and H-pile size.

The interdependence of the three main variables is discussed here. At a constant amount of energy, a smaller penetration set (higher blow count) results in greater pile capacity, and the greater the penetration set (lower blow count) the lower the pile capacity. At a fixed capacity, greater transferred energy results in a higher penetration set and lower energy results in a lower penetration set. Higher transferred energy results in greater pile capacity, at a constant penetration set, and lower energy results in lower pile capacity.

Omitting any one of the three main variables (*i.e.* Q_{ult}, set, and energy) produces misleading results. For example, a pile driven to practical refusal using an under-performing hammer may produce a high blow count in the field leading an observer to believe that the pile has met the specifications. However, if a PDA test and CAPWAP analysis were to be performed on that pile, it may be discovered that the capacity is low and the maximum capacity has not been mobilized. In contrast, driving a pile using a hammer with a high stroke may produce a low blow count deceiving the observer to believe the pile has not developed the required capacity. A PDA test coupled with a CAPWAP analysis may determine that the design capacity has been exceeded and the pile has been overstressed. These situations are prime examples of why all three of the main dynamic variables must always be considered during dynamic analysis.

The transferred-energy variable in the bearing graphs is determined during the PDA test from the strain and accelerometer gauges. The force, velocity, and subsequent energy transferred through the piles are determined by means of basic wave mechanics and the Case method as discussed in Chapter 2. The amount of energy transferred to the piles is dependent on the stroke and (or) fuel setting of the piling hammer and the efficiency depends on the hammer type and its performance. For the purpose of the bearing graph, the amount of transferred energy was separated into intervals of 10 kJ, including ranges of less than 20, 20 to 30, 30 to 40, and greater than 40 kJ. These interval sizes were chosen with expectation of finding a trend in the data, as too many intervals will likely end in results that are too complicated to distinguish. Measured transferred energies of the locally available hammers in Winnipeg are provided in Section 5.2.2.2.

Values of the Q_{ult} of the H-piles were computed from the CAPWAP analyses. All piles analysed met the design refusal criterion prior to testing. Comparisons of the Q_{ult} and Q_{b-ult} were made to investigate if stronger correlations to blow count and transferred energy could be found with either. Analyzing the different end bearing stratum was also reviewed to see if any tendencies could be uncovered. Details of the ultimate pile capacities were discussed previously in Chapter 4.

The pile penetration sets were measured in the field during the PDA tests. Sets of 5 blows were typically marked on the piles and recorded instead of the regular refusal criterion sets (*i.e.* every 15 blows). Five blows is low enough to minimize changes that may occur within a 15 blow set and high enough to accurately measure on piles exhibiting high penetration resistance. Taking the average pile penetration over a 5-blow set is most practical. The sets were marked using typical construction techniques. The penetration sets measured during the PDA tests ranged between 0.2 and 6 mm per hammer blow.

The bearing graphs presented in this chapter display the Q_{ult} (or $Q_{b\text{-}ult}$) on the y-axis and the penetration set on the x-axis. The data points are color coordinated to represent the intervals of transferred energy. Unfilled markers represent piles that reached practical refusal in bedrock and solid markers represent the piles seated in till. Different marker shapes are used to separate other variables such as pile size and testing condition. Table 5.1 provides all the data required to build the bearing graphs.

The general bearing graphs shown in Figures 5.1 and 5.2 describe the Q_{ult} and $Q_{b\text{-}ult}$, respectively, as they relate to penetration set, energy, and end bearing stratum. Certain general observations are made upon review of these bearing graphs which are most clearly described in point form:

- Smaller pile penetration sets result in higher Q_{ult} and $Q_{b\text{-}ult}$
- Higher energy results in larger penetration sets
- Higher energy results in higher Q_{ult} and $Q_{b\text{-}ult}$

- Piles seated in bedrock have higher Q_{ult} and $Q_{b\text{-}ult}$ than piles seated in till at comparable energy and penetration sets
- Variability in the data is decreased when Q_{ult} is the focus of the bearing graph as opposed to the $Q_{b\text{-}ult}$

The first three points of the general observations confirm the interdependence of the three main variables discussed above. A fourth variable, end bearing stratum, was merged into the bearing graphs that included piles seated in bedrock, silt till and sand till. A clear indication of reduced Q_{ult} and $Q_{b\text{-}ult}$ values in the tills can be discerned at comparable transferred energies and penetration sets. This is dynamic evidence to the claim provided in Chapter 4 that a pile seated in bedrock develops more capacity than a pile seated in till.

The testing conditions (*i.e.* BOR and EOID) were included into the bearing graph as shown in Figures 5.3 and 5.4. Markers are provided that indicate the EOID and BOR testing conditions. Although the relationship between the three main dynamic variables is not impacted by pile set-up, since pile set-up is a time-dependent static characteristic, the differences in the Q_{ult} and $Q_{b\text{-}ult}$ of different H-pile sizes are more noticeable in a dynamic context such as the bearing graph. The bearing graphs shown in Figures 5.5 and 5.6 replace the end-bearing-stratum variable with an H-pile-size variable. The larger H-piles generally develop more Q_{ult} and $Q_{b\text{-}ult}$ at similar penetration sets and transferred energy, as discussed in Chapter 4 and confirmed here. The bearing graphs shown in Figures 5.7 and 5.8 include all the variables, with the exception of the testing condition.

The bearing graphs provided in Figures 5.1 to 5.8 illustrate the complexity of dynamic analysis of driven piles. When incorporating different end bearing stratum, assorted pile sizes, and multiple hammers, the bearing graphs become much more involved than a typical bearing graph created for a single pile in a set stratigraphy using a particular hammer. Due to all the variables included in the analysis, the bearing graphs presented in this chapter show a fair amount of variability and should therefore not be considered as very reliable RAMs. It remains difficult to provide a refined refusal criterion due to the variability present. However, the bearing graph RAMs are valuable. For example, if the pile size, hammer type, and end bearing stratum are known, a refusal criterion and the Q_{ult} (or $Q_{b\text{-}ult}$) can be estimated with increased confidence using the given bearing graphs when compared to the conventional design criteria. For example, an HP310x110 H-pile driven using a hammer capable of transferring 20 and 30 kJ of energy penetration and a refusal criterion of 1 mm per blow (or 25 blows per 25 mm) can be expected to develop Q_{ult} of about 2800 kN. If a more reasonable (*i.e.* less than 20 blows per 25 mm) refusal criterion is desired, such as 15 blows per 25 mm, then Q_{ult} in the range of 2500 to 2600 kN can be expected.

These bearing graphs show quite a bit of variability, because they are based on observed results and complex analysis. To achieve optimal bearing graphs in regard to the intended purpose, more data points are required. However, the currently presented bearing graphs provide a first step towards improving the understanding of the capacity of driven steel H-piles in Winnipeg, in regard to their dynamic behavior and the main dynamic variables.

Review of the bearing graphs in terms of the Q_{ult} and Q_{b-ult} revealed some surprising secondary results. The author expected that the Q_{b-ult} would yield less variability in the data since the Q_{s-ult} is considerably variable from site to site. It seemed reasonable that removing the shaft component from the bearing graph would result in less variable data distribution; however, based on the appearance of the graphs, the opposite occurred. The author considers that pile penetration during driving is more sensitive to the end bearing resistance than the amount of embedded pile length, based on the above noted findings. The overburden soils, particularly clay, have a much less influence on driving behavior than bedrock and (or) thick deposits of till. The karst limestone bedrock in Winnipeg can exhibit great variability in strength and degree of fracturing, resulting in greater variability in driving behavior. It appears that H-piles with greater end bearing resistance develop proportionally less shaft friction and vice versa. Since the shaft soils have little influence on penetration sets but high influence on the Q_{ult}, the Q_{s-ult} has a tendency to "smooth" the data. The bearing graphs that include the Q_{ult} (in contrast to Q_{b-ult}) may be more reliable for this reason.

A common question that arises in determining a pile's refusal criterion is whether to drive a pile using a high blow count with a lesser amount of energy, or a higher amount of energy at a lower blow count. This question may be easier to answer after reviewing the bearing graphs discussed above and reading the forthcoming section about the driving system performance and driving stresses. If enough capacity can be developed to satisfy the foundation engineer by driving the pile at a high blow count and low transferred energy, then the potentially damaging effects of hard driving can be avoided. The bearing graphs should be used in conjunction with the information regarding driving stresses that is provided in the second part of this chapter.

5.2 Correlation Study: Evaluation and Refinement of Dynamic RAMs

The accuracy and variability of the dynamic RAMs, discussed in Chapter 2, are evaluated in a correlation study which is presented in the second part of this chapter. The evaluation consists of comparing the ultimate capacities of the H-piles as estimated by the modified Gates formula, GRLWEAP and the Case method to the ultimate capacities determined using CAPWAP. Project pile design, pile installation records, hammer specifics, soil stratigraphy, and soil characteristics identified during the geotechnical investigation and construction at eleven of the twelve project sites are used in the correlation analyses. The study does not include Site 5, due to the uncertainty in the drop heights of the drop hammer.

The first step of the correlation study consists of estimating the ultimate capacities at each project site using recommended input properties and comparing them to the CAPWAP-calculated ultimate capacities. The recommended values used in this step are alluded to in the literature cited in Chapter 2. The second step of the correlation study consists of using back-analysed input property values determined from the CAPWAP analyses to refine the dynamic RAMs in an attempt to improve their accuracy and variability. The recommended and refined input properties that are under scrutiny include transferred energy, soil damping, and quake values.

Table 5.2 summarizes the results of the correlation study in tabular form. A site by site comparison of the ultimate capacities as predicted by the dynamic RAMs using the recommended input properties and those calculated using CAPWAP are provided in Appendix C. It should be noted again that the results at Site 5 were excluded from the correlation study. The ultimate capacities as determined by the modified Gates formula and GRLWEAP analyses are plotted on the graphs as simulated non-linear curves. The ultimate capacities calculated using the Case method and CAPWAP are plotted as single points on the graphs because the values are based on single capacity measurements.

5.2.1 Evaluation of Dynamic RAMs using Recommended Input Properties

The accuracy and variability of the dynamic RAMs for predicting the Q_{ult} of piles are based on the quality of the input properties. The evaluation step of the correlation study assesses the validity of the recommended and (or) or estimated input properties. Evaluation of the modified Gates formula, GRLWEAP and the Case method is based on the energy transferred to the pile head from the different hammers and the J_s, quake, and J_c values of the soils surrounding the H-piles.

The key input parameter in the modified Gates formula for predicting the Q_{ult} is the energy transferred through the driving system. The recommended transferred energies selected for the evaluation step of correlation study are based on recommended hammer and cushion properties and efficiencies provided in GRLWEAP. Penetration sets ranging from 0 to 25 mm are used to calculate a corresponding range of Q_{ult} values which are then plotted on the bearing graphs. Therefore, the WEA was performed prior to the modified Gates formula analysis to obtain the recommended transferred energies.

The 2010 version of GRLWEAP was used for the WEA to predict the penetration set for the corresponding Q_{ult}. A recommended hammer efficiency of 0.8 and recommended hammer cushion parameters were used to determine the energy transferred to the pile head. Table 2.1 provides the recommended values of static and dynamic properties of quake and J_s. A J_s value of 0.3 was assumed for the silt till. Pile size and length variables and the soil stratigraphy are incorporated into the more complex and theoretical WEA which are absent from the modified Gates formula.

The accuracy of the prediction of the Q_{ult} using the Case method depends on the J_c dynamic input variable. A single J_c value is selected to represent the soil and rock stratigraphy at each site. It is determined by calculating the weighted average using J_c values assigned to each soil layer. The J_c values selected to characterize each soil layer are based on recommended ranges of values provided in Table 2.2. The J_c value chosen for clay, silt till and sand till were 0.85, 0.6, and 0.45, respectively. The weighted average J_c values were rounded to the nearest decimal place.

5.2.1.1 Statistical Results

Figure 5.9 shows the ultimate capacities determined by the three dynamic RAMs (*i.e.* the modified Gates formula, GRWEAP, and the Case Method) using recommended and estimated input properties, and the CAPWAP-calculated ultimate capacities. Overall, the

Case method generally over-predicts the Q_{ult} of H-piles but provides decent correlation (accuracy) to the CAPWAP determined Q_{ult} and minor variability. The ultimate capacities, as predicted by GRLWEAP, exhibit more variability than those predicted by the Case method but produce similar correlation to the CAPWAP-calculated ultimate capacities. The ultimate capacities predicted by the modified Gates formula shows even greater variability than those predicted by the Case method and GRLWEAP analysis. The modified Gates formula tends to under-predict the Q_{ult} compared to CAPWAP.

5.2.1.1.1 Comparison of the Ultimate Capacities Predicted by the Modified Gates Formula to the CAPWAP-Calculated Ultimate Capacities

Comparison of the ultimate capacities predicted by the modified Gates formula to the CAPWAP-calculated ultimate capacities, as shown in Figure 5.9, yielded surprising results. The modified Gates formula under-predicted the Q_{ult} for 95 percent of the tested H-piles when using the recommended hammer and cushion efficiencies. The ratio of the ultimate capacities as predicted by the modified Gates formula to ultimate capacities calculated by CAPWAP ranged between 0.55 and 1.24 with a mean value of 0.79 and a COV equal to 0.18.

The ratio of the ultimate capacities as predicted by the modified Gates formula to ultimate capacities calculated by CAPWAP was assessed for H-piles seated in till and bedrock. Interestingly, the modified Gates formula yielded more accurate and less variable results in predicting the Q_{ult} of H-piles seated in till than in bedrock. The best correlation was observed with H-piles seated in sand till. These H-piles had a ratio ranging between 0.76 and 1.07 with a mean of 0.93 and a COV of 0.10. H-piles seated in silt till had a ratio ranging between 0.74 and 0.91 with an average of 0.84 and a COV equal to 0.07. The ratio for H-piles seated in bedrock, varied between 0.55 and 1.24 with an average of 0.76 and a COV equal to 0.19.

5.2.1.1.2 Comparison of the Ultimate Capacities Predicted by GRLWEAP to the CAPWAP-Calculated Ultimate Capacities

Overall comparison of the ratio of the ultimate capacities as predicted by GRLWEAP to the CAPWAP-calculated ultimate capacities, illustrated in Figure 5.9, yielded more accurate results than the modified Gates formula. The overall ratio ranged between 0.69 and 1.37 with a mean value of 0.97 and a COV equal to 0.17. H-piles seated in bedrock had a ratio varying between 0.69 and 1.37 with an average of 0.96 and a COV of 0.17. H-piles seated in silt till had a ratio ranging between 0.79 and 0.93 with a mean of 0.86 with a COV equal to 0.06. The ratio of H-piles seated in sand till ranged between 0.90 and 1.27 with an average of 1.12 and a COV of 0.10. The results of the GRLWEAP correlation analysis show that this RAM is more accurate than the modified Gates formula in predicting the Q_{ult} of driven steel H-piles seated in bedrock, but not in predicting the Q_{ult} of H-piles seated in till. It is actually less accurate in predicting the Q_{ult} of H-piles seated in sand till.

The transferred energy is relatively consistent for piles seated in bedrock and till; therefore, the difference in the results is likely attributed to the soil damping and quake properties. Perhaps the variability of the karst bedrock and heterogeneous till are not represented very well by these particular recommended input values.

5.2.1.1.3 Comparison of the Ultimate Capacities Predicted by the Case Method to the CAPWAP-Calculated Ultimate Capacities

A comparison of the ratio of the ultimate capacities as predicted by the Case method to the CAPWAP-calculated ultimate capacities was made. Sixty-six percent of the analyses resulted in the Q_{ult} being over-predicted. The ratio ranged between 0.67 and 1.38 with a mean value of 1.04 and a COV equal to 0.12. There was no discernible difference in the correlation results between H-piles seated in bedrock and H-piles seated in till. The accuracy and variability of predicting the Q_{ult} of H-piles using the Case method is an improvement to the modified Gates formula and GRLWEAP analysis.

Generally, the dynamic RAMs that are based on recommended and estimated input properties evaluated in the correlation study provide reasonable accuracy and variability in estimating the Q_{ult} of driven steel H-piles in Winnipeg. However, there is room for improvement. To improve the correlation results of these dynamic RAMs, the input properties need to be refined to represent local hammer and soil components for pile dynamic analysis. An attempt to refine these input variables and increase the reliability of the dynamic RAMs is forthcoming in the following sections.

5.2.2 Measured Dynamic Input Variables

Review of the PDA test data and CAPWAP analyses presented an opportunity to improve the accuracy and variability of the dynamic RAMs. Measurements of transferred energy and damping and quake quantities were compiled, reduced, and organized for refinement. Driving stresses were also examined to determine if the yield limits of the pile material were being approached during typical local construction practice.

5.2.2.1 Damping and Quake Values

The calculated J_s and quake values were compiled from each CAPWAP analysis. The results showed high variability. As a result, great difficulty was encountered when attempting to correlate the J_s and quake values to specific soil layers and their strength properties, because the CAPWAP-calculated J_s and quake values were constant along the pile shafts. Therefore, the differences in the soil types and their properties were not represented by distinct values. As a result, the correlation of J_s and quake values to specific soil layers and their strength properties could not be performed with any reasonable degree of accuracy. The variability in the till and karst bedrock likely contributed to the high variability of the J_s and quake values of the piles that were calculated by CAPWAP.

Figures 5.10 to 5.13 show the distribution of the average CAPWAP-calculated damping and quake values of the H-piles. Figure 5.10 shows that the shaft quakes range between 1.01 and 7.35 with a mean of 3.60 and a COV equal to 0.44. Figure 5.11 illustrate that the toe quakes range between 1.01 and 12.98 with an average of 4.36 and COV equal to

0.45. The J_s values along the pile shaft, as illustrated in Figure 5.12, range between 0.17 and 1.43 with a mean value of 0.82 and a COV equal to 0.41. The J_s values at the pile toe range between 0.05 and 1.01 with an average of 0.38 and a COV equal to 0.61, as shown in Figure 5.13. Surprisingly, there is no discernible difference in either the J_s or quake values in regard to the end bearing stratum in which the piles are seated. The variability in the J_s and quake results may be attributed to the variability in the condition of karst bedrock and the heterogeneity of the till. Consistent results in these types of formations (deposits) cannot be expected.

The J_c values that produced ultimate capacities that resulted in the best correlation to the ultimate capacities calculated using CAPWAP were isolated from the results. This was completed to get a sense of the J_c values the best represent driven steel H-piles in Winnipeg. Figure 5.14 presents the distribution of the J_c values that produced the best correlation of the Q_{ult} (via the Case method) to the CAPWAP-calculated Q_{ult}. A J_c value equal to 0.9 provided the most accurate capacities 30 percent of the time. Seventy-two percent of the H-piles had their best correlation to the CAPWAP results, when using J_c values equal to 0.8, 0.9, and 0 (independent of damping).

5.2.2.1.1 Mobilization of Ultimate Capacity

The shaft and toe quake values represent the static displacements that must be achieved or exceeded to mobilize the $Q_{s\text{-}ult}$ and $Q_{b\text{-}ult}$, respectively. As illustrated in Figure 5.11, and described in the previous section, mobilization of $Q_{s\text{-}ult}$ and $Q_{b\text{-}ult}$ of driven steel H-piles in Winnipeg occurs on average at about 3.6 and 4.36 mm, respectively. It must be noted that these values are actual measured values and the level of precision of these values should not be expected in practice.

5.2.2.2 Driving System Efficiency and Transferred Energy

The performance of four different hammers were evaluated, including a Junttan HHK 5A hydraulic hammer, a Delmag D19-32 diesel hammer, a Pileco D19-42 diesel hammer, and an I.C.E. I-19 diesel hammer. These four hammer types are commonly used for driving steel H-piles in Winnipeg.

Overall, the Junttan HHK 5A hydraulic hammer considerably out-performed all other hammers, in terms of efficiency. Hammer efficiency is defined as the ratio of the transferred energy to the rated energy. On average it was 66 percent more efficient than the next competitor, the Pileco D19-42 diesel hammer. The efficiency of the Junttan HHK 5A hammer and piling system ranged from 80 to 96.5 percent with an average of 89.9 percent and a COV equal to 0.05. The actual efficiency of the piling system with the Junttan HHK 5A hydraulic hammer is significantly greater than recommended in GRLWEAP.

The diesel hammers proved to be much less efficient than the hydraulic hammer. The Delmag D19-32 produced efficiencies through the driving system in the order of 32.5 to 48.6 percent with an average of 43 percent and a COV equal to 0.17. The Pileco D19-42 generated transferred energy efficiencies in the order of 43.6 to 79.3 percent with a mean value of 54.1 percent and a COV equal to 0.15. The I.C.E. I-19 had similar performance to the Pileco manufacturing efficiencies ranging from 48.7 to 60.2 percent with an average of 53.9 percent and a COV of 0.07.

5.2.2.3 Driving Stresses

Excessive driving stresses are the main contributor to pile damage. When stresses reach or exceed the yield stress of the pile material the elastic limits are reached or surpassed, causing permanent (plastic) deformation and pile damage. The PDA uses strain gauges to monitor compressive and tensile stresses induced during driving. Stress data was collected and analysed to assess the stresses that are being transferred to driven steel H-piles with typical generated hammer energies. Figure 5.15 illustrates the relationship between transferred energy and driving induced stresses. As expected, the driving stresses increase with the amount of energy transferred to the pile. The stresses were measured when the piles were at, or near, practical refusal. Therefore, the compressive stresses are much greater than the tensile stresses. The compressive stresses are in the order of 3 to 37 times greater than the tensile stresses. Figure 5.16 illustrates penetration set versus the compressive stresses measured in the piles. As driving conditions become harder, the generated compressive stresses increase and the sets become smaller. In no case was pile damage indicated by the PDA during driving. All but one of the H-piles tested were rolled from 350W grade steel and the highest stress recorded during all of the sixty-four tests was 307 MPa in compression, which is about 88 percent of the yield stress of the steel.

5.2.3 Refinement of the Dynamic RAMs using Measured Input Properties

The author attempted to improve the accuracy and variability of the dynamic RAMs for predicting the Q_{ult} of driven steel H-piles in Winnipeg. This effort of refinement was undertaken by implementing CAPWAP-calculated input properties into the dynamic RAMs discussed in this thesis. The principal method of refinement is concentrated on the performance of the local piling hammers and their efficiencies. Improvement based on shaft and toe damping and shaft quake values was not attempted due to the variability observed in the CAPWAP-calculated values.

The work invested in improving the accuracy of modified Gates formula included implementing the average PDA recorded transferred energies for the four different hammer types and modifying the empirical factor (*i.e.* 6.7, expressed at the front of the dynamic formula in Equation 2.10). The refined GRLWEAP analysis was comprised of applying the same transferred energies and adjusting the toe quake. The Case method was enhanced by using the J_c values that deliver the best-fit correlation of the predicted Q_{ult} to the CAPWAP-calculated Q_{ult}. The statistical results of the refined analyses are discussed next.

5.2.3.1 Statistical Results

The overall correlations of the Q_{ult} determined by the refined dynamic RAMs to the CAPWAP-calculated ultimate capacities are shown in Figure 5.17. Improvements to the dynamic RAMs were noticed when compared to the correlations based on the recommended input properties. The accuracy and variability of the Case method improves substantially, meanwhile the accuracy of both the modified Gates formula and GRLWEAP moderately improve with no significant change in the variability. These improvements are discussed in following paragraphs.

5.2.3.1.1 Comparison of the Ultimate Capacities Predicted by the Refined Modified Gates Formula to the CAPWAP-Calculated Ultimate Capacities

A key variable in the modified Gates formula is the amount of energy transferred to the pile. It is expected that real, rather than assumed, transferred energy quantities would result in more accurate prediction of the Q_{ult}. The average efficiencies through the driving system delivered by the hammers discussed in Section 5.2.2.2 were implemented into the computations replacing the recommended input values. Comparisons between the results from the estimated and refined predictions of the Q_{ult} are made.

The first trial of refining the modified Gates formula resulted in improvements in the accuracy. The ratio of the ultimate capacities as predicted by the refined modified Gates formula to the ultimate capacities calculated by CAPWAP was assessed for H-piles seated in till and bedrock. The overall ratio using the modified Gates formula ranged between 0.56 and 1.41 with an average of 0.86 and a COV equal to 0.21. This equates to an overall improvement of 7 percent in the accuracy from the predicted capacities based on recommended driving system efficiencies. The most noticeable improvements of the modified Gates formula occurred for piles seated in till. The ratio for piles seated in silt till ranged between 0.86 and 1.05 with an average of 0.97 and a COV equal to 0.07; an improvement of 13 percent from the predicted ultimate capacities that used recommended hammer efficiencies. The ratio of H-piles seated in sand till ranged between 0.81 and 1.19 with an average equal to 1.0 with a COV of 0.12; an improvement of 7 percent in the accuracy from the recommended based results. Piles seated in bedrock had a ratio ranging between 0.56 and 1.41 with an average of 0.81 and a COV equal to 0.23, which equals to an improvement of 5 percent. Again it can be seen that the variability in the results is quite low for H-piles seated in till relative to H-piles seated in bedrock.

The first-trial refinement results indicate that the modified Gates formula still conservatively under-predicts the Q_{ult} of H-piles seated in bedrock by 19 percent. Although H-piles seated in till yielded good results, it was realized that there was room for further improvement in predicting capacity of H-piles seated in bedrock. This led to a second trial refinement which included modifying the empirical factor to better correlate the Q_{ult} to the CAPWAP-calculated ultimate capacities for H-piles seated in bedrock. To perform this task, the empirical factor was adjusted until the correlation value for piles seated in bedrock was equal to 1.0. The best-fit empirical factor was calculated to be 7.9. This modified empirical factor resulted in a ratio of ultimate capacities as calculated by the refined modified Gates formula to the ultimate capacities calculated using CAPWAP that varied from 0.69 to 1.71 with a mean value of 1.0 and a COV equal to 0.22. This equates to an improvement of 24 percent compared to the original prediction based on the recommended input property values. The new factor improved the average accuracy of the modified Gates formula for piles seated in bedrock, but did not improve the variability. The variability found in the results for all the H-piles is probably related to a combination of the variability of the underlying till and karst bedrock formations, along with the incorrect fundamentals of the modified Gates formula.

The modified Gates formula data plotted on the bearing graphs in Appendix C include the analyses based on an empirical factor equal to 6.7 to for H-piles seated in till and 7.9 for H-piles seated in bedrock. The overall correlation plot (Figure 5.17) of the refined analyses also includes the predicted values based on the strata in which the H-piles are seated. On the basis of refined analysis, it is recommended that empirical coefficients, for the modified Gates formula, of 6.7 and 7.9 be used in practice for H-piles seated in till and bedrock, respectively.

5.2.3.1.2 Comparison of Ultimate Capacities Predicted by the Refined GRLWEAP Analysis to the CAPWAP-Calculated Ultimate Capacities

An attempt to refine the GRLWEAP analyses based on CAPWAP-calculated J_s and quake values was unsuccessful, as it resulted in computational errors and extremely poor results. The CAPWAP-calculated J_s and quake values were constant along the pile shafts and therefore could not be correlated to specific soil types and their strength properties. Therefore, the calculated values of J_s and quake cannot properly be used to represent the variability in the soil strata that is usually encountered from site to site. The author knows no method to isolate the shaft J_s and quake values from the existing CAPWAP data. For this reason, the refined WEA was performed on the basis of measured transferred energies alone.

Refinement of the WEA using GRLWEAP was completed by adjusting the hammer efficiencies to achieve transferred energies that matched or nearly matched the average values determined from the PDA tests and discussed in Section 5.2.2.2. Overall, the ratio of the Q_{ult} as determined by the refined GRLWEAP analysis to the CAPWAP-calculated Q_{ult} ranged between 0.67 and 1.45 with an average of 0.99 and a COV equal to 0.15. On the basis of these statistics, the average correlation to CAPWAP improved by 2 percent compared to the predicted ultimate capacities determined by the unrefined GRLWEAP analysis.

The ratio of the ultimate capacities as predicted by the refined GRLWEAP method to the ultimate capacities calculated by CAPWAP was assessed for H-piles seated in till and bedrock. The ratio improved by 3 percent and 11 percent for H-piles seated in bedrock and silt till, respectively, over the original prediction analysis based on recommended input properties. For H-piles seated in sand till, the ratio worsened by 7 percent. The refined ratio for H-piles seated in bedrock ranged from 0.67 to 1.45 with an average of 0.99 and a COV equal to 0.15. For H-piles seated in silt till, the ratio ranged from 0.89 to 1.05 with an average of 0.97 and a COV equal to 0.06. The ratio for piles seated in sand till ranged between 0.97 and 1.36 with a mean value of 1.19 and a COV of 0.11.

These results showed that further improvements could be made to better predict the Q_{ult} of H-piles seated in sand till. Adjustments were made to the toe quake until a reasonable average correlation of the ultimate capacities as determined in CAPWAP for H-piles seated in sand till was achieved. A toe quake equal to 6.5 mm yielded a ratio to

CAPWAP of 0.99 with a range of ratio values between 0.79 and 1.18 with a COV equal to 0.12, which is an improvement of 11 percent from the original analyses using the recommended input properties. A toe quake equal to 6.5 mm is considered acceptable according to Rausche et al. (2010) and the recommended values presented in Table 2.3.

Measured transferred energies along with the recommended J_s and quake values (provided in Table 2.1) should be used to assess the Q_{ult} of H-piles seated in silt till and bedrock. For H-piles seated in sand till, the measured transferred energies should also be implemented in the GRLWEAP analysis along with the recommended J_s and quake values, with a slight adjustment; a toe quake of 6.5 mm should be used.

5.2.3.1.3 Comparison of Ultimate Capacities Predicted by the Refined Case Method to the CAPWAP-Calculated Ultimate Capacities

The correlation between the Case method and CAPWAP was improved upon during the refinement analysis. The refined Case method that was used to estimate the Q_{ult} of H-piles was based on a J_c value equal to 0.9. This value of J_c was used because it predicted the most accurate ultimate H-pile capacities 30 percent of the time, which was more frequent than all other J_c values. The ratio of the Q_{ult} as predicted by the refined Case method to the ultimate capacities calculated by CAPWAP was assessed for all H-piles. The ratio ranges between 0.67 and 1.34 with a mean value equal to 1.0 at a COV equal to 0.12. These statistics equate to an improvement in accuracy of 4 percent in comparison to the original predicted values as determined using weighted average values of the J_c factor.

In the case when the most accurate J_c value was selected for analyses, the ratio varies between 0.95 to 1.17 with a mean value of 1.02 and a COV equal to 0.04. These statistics indicate an improvement of 2 percent in the mean value and 0.08 in the COV. This improvement in the accuracy and variability is misleading however. The Case method is only as accurate as the value of J_c selected and monitored during the PDA test. The ability to select the appropriate value requires experience with PDA testing and the Case method. The refined Case method based on a J_c value equal to 0.9 should be used in analysis, unless experience justifies use of another value.

5.3 Chapter 5 Discussion

The overall accuracy of the modified Gates formula in predicting the Q_{ult} of driven steel H-piles in Winnipeg is considered acceptable. Implementing measured transferred energies and empirical factors specific to H-piles seated in till and bedrock leads to promising results. The modified Gates formula may be valuable at the design and production stages in terms of accuracy. However, the variability determined in the correlation study was not pleasing and brings into question its reliability. Caution must be taken when applying this particular dynamic RAM.

The variability in the modified Gates formula correlation is not surprising, considering dynamic analysis has many variables which are absent in this particular dynamic RAM. Predicting the Q_{ult} of H-piles on the basis of empirically formed equations with a mere two variables (*i.e.* transferred energy and blow count) should not be expected to provide consistently accurate results, especially in an area like Winnipeg which is known for its spatial variability. Although the modified Gates formula worked quite well at some sites it performed poorly at other sites. This gives reason to believe that the empirical formula was developed at specific sites with common stratigraphy, pile penetrations, and other components that effect driven H-pile capacity. However, additional PDA testing and CAPWAP analyses should improve the reliability of the modified Gates formula for use in Winnipeg.

The overall accuracy of estimating the Q_{ult} of driven steel H-piles in Winnipeg using the refined GRWLEAP analysis was equally as good as the modified Gates formula. When using toe quakes equal to 2.5 and 6.5 mm for H-piles seated in silt and sand till, respectively, GRLWEAP performed very well. The accuracy observed for H-piles seated in bedrock was equally reputable. The variability of the Q_{ult} as predicted by the refined GRLWEAP analyses for H-piles seated in till was similar to what was observed in the results of the modified Gates formula. The variability in the results for H-piles seated in bedrock was improved with the refined GRLWEAP to a greater extent than the modified Gates formula.

The accuracy and variability of the Q_{ult} predicted by the Case method was excellent when the correct J_c value was selected. The estimated weighted average of the J_c values, however, led to non-conservative and over-predicted results. The difficulty with using the Case method alone to predict the Q_{ult} is selecting which J_c value will provide the most accurate results. This uncertainty of the correct J_c values is reduced with the distribution plot (Figure 5.14) of J_c values. However, further collection of PDA test and CAPWAP data may be required to achieve a more refined sense of the J_c values that best represent Winnipeg stratigraphy, if possible. Research could also be completed to specifically isolate the J_c values that best represent the clay, till, and bedrock in Winnipeg.

Although improvements were made with the refined dynamic RAMs for estimating the Q_{ult} of driven steel H-piles in Winnipeg, the variability in karst bedrock and heterogeneous till worked against achieving an excellent correlation with the ultimate capacities calculated using CAPWAP. A larger sample size would allow for further refinement of the dynamic RAMs.

Chapters 4 and 5 focused specifically on the Q_{ult} of driven steel H-piles in Winnipeg for the purpose of improving the ULS design criteria. Chapter 6 moves the focus towards understanding the performance of driven steel H-piles for the purpose of developing a comprehensive SLS design criteria on the basis of the CAPWAP load-displacement curves.

Table 5.1: The PDA test and CAPWAP analysis data required to construct the bearing graphs.

Site	Test Condition	Hammer Details													
		Model	Type	Fuel Setting / Stroke	Rated Energy	Maximum Transferred Energy	Efficiency	Measured Set	Blow Count	Final Pile Penetration	Pile Size	Total Ultimate Capacity	Ultimate End Bearing Capacity	Ultimate Shaft Capacity	End Bearing Strata
					kN·m	kN·m	%	mm	BPI	m		kN	kN	kN	
1	EOID	I.C.E. I-19	ICH (Diesel)	#2	51.8	27.8	53.7	1.0	25.4	15.8	HP310x110	2853	1900	953	BEDROCK
	EOID	I.C.E. I-19	ICH (Diesel)	#2	51.8	27.8	53.7	1.4	18.1	14.0	HP310x110	2976	2202	774	BEDROCK
	BOR	I.C.E. I-19	ICH (Diesel)	#2	51.8	27.2	52.5	0.6	42.3	12.6	HP310x132	3501	2958	543	BEDROCK
	BOR	I.C.E. I-19	ICH (Diesel)	#2	51.8	25.2	48.7	1.0	25.4	12.4	HP310x132	3300	2593	707	BEDROCK
	BOR	I.C.E. I-19	ICH (Diesel)	#2	51.8	28.4	54.7	1.0	25.4	13.3	HP310x132	3302	2932	370	BEDROCK
	BOR	I.C.E. I-19	ICH (Diesel)	#2	51.8	31.2	60.2	1.4	18.1	13.3	HP310x94	2871	2278	593	BEDROCK
2	EOID	Junttan HHK 5A	ECH (Hydraulic)	0.5	24.5	23.6	96.4	0.6	42.3	14.0	HP200x54	1614	836	778	BEDROCK
	EOID	Junttan HHK 5A	ECH (Hydraulic)	1.1	53.9	45.3	84.1	1.0	25.4	12.3	HP360x132	2862	1663	1198	BEDROCK
	EOID	Junttan HHK 5A	ECH (Hydraulic)	1.1	53.9	50.9	94.4	0.8	31.8	17.8	HP360x132	2302	1567	735	BEDROCK
	EOID	Junttan HHK 5A	ECH (Hydraulic)	1.1	53.9	51.1	94.7	2.0	12.7	14.3	HP360x132	2381	1449	932	BEDROCK
	BOR	Junttan HHK 5A	ECH (Hydraulic)	1.1	53.9	45.5	84.3	0.4	63.5	10.5	HP360x132	4325	2888	1437	BEDROCK
	BOR	Junttan HHK 5A	ECH (Hydraulic)	1.1	53.9	43.1	80.0	0.2	127.0	10.5	HP360x132	4037	3684	353	BEDROCK
3	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.7	34.3	32.3	94.3	1.4	18.1	12.8	HP310x110	2332	1661	671	BEDROCK
4	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.5	24.5	22.4	91.3	1.0	25.4	5.5	HP310x110	2405	2155	250	BEDROCK
	EOID	Junttan HHK 5A	ECH (Hydraulic)	0.5	24.5	21.8	88.8	0.5	50.8	6.9	HP310x110	2543	2254	289	BEDROCK
	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.5	24.5	22.4	91.3	1.0	25.4	5.3	HP310x110	2907	2587	321	BEDROCK
5	EOID	-	ECH (Drop)	2.1	46.7	22.9	49.0	2.3	11.0	10.8	HP310x110	2243	1685	558	BEDROCK
	EOID	-	ECH (Drop)	3.0	66.7	33.2	49.7	1.9	13.4	10.8	HP310x110	3104	1195	1909	BEDROCK
	BOR	-	ECH (Drop)	2.4	53.4	23.9	44.9	2.0	12.7	11.7	HP310x110	2419	1765	653	BEDROCK
	BOR	-	ECH (Drop)	2.4	53.4	24.3	45.6	2.0	12.7	11.4	HP310x110	2414	1332	1082	BEDROCK
	BOR	-	ECH (Drop)	2.4	53.4	28.2	52.9	2.0	12.7	12.5	HP310x110	2101	1360	742	BEDROCK
6	EOID	Delmag D19-32	ICH (Diesel)	#3	47.1	22.9	48.6	1.3	19.5	4.0	HP360x132	2537	2200	337	BEDROCK
	EOID	Delmag D19-32	ICH (Diesel)	#3	47.1	22.3	47.4	1.5	16.9	6.6	HP360x132	2348	1776	573	BEDROCK
	BOR	Delmag D19-32	ICH (Diesel)	#3	47.1	22.9	48.5	2.6	9.8	6.6	HP360x132	1964	1587	377	BEDROCK
	BOR	Delmag D19-32	ICH (Diesel)	#3	47.1	37.9	80.5	1.0	25.4	6.4	HP360x132	3482	3192	290	BEDROCK
	BOR	Delmag D19-32	ICH (Diesel)	#3	47.1	34.0	72.1	2.0	12.7	6.4	HP360x132	3419	3233	186	BEDROCK
7	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	36.1	92.0	1.6	15.9	20.1	HP310x110	2514	1400	1113	SILT TILL
	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	33.6	85.7	2.0	12.7	19.6	HP310x110	2267	1137	1130	SILT TILL
	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	37.3	95.1	2.4	10.6	21.1	HP310x110	2254	1395	859	SILT TILL
	EOID	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	37.3	95.2	6.0	4.2	18.3	HP310x110	1860	1023	837	SILT TILL
	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	32.5	82.8	1.8	14.1	19.7	HP310x110	2217	1476	741	SILT TILL
	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	35.8	91.3	2.0	12.7	18.2	HP310x110	2094	1355	739	SILT TILL
	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	34.7	88.5	2.2	11.5	17.9	HP310x110	2095	1390	704	SILT TILL
	BOR	Junttan HHK 5A	ECH (Hydraulic)	0.8	39.2	34.2	87.3	1.0	25.4	32.2	HP310x110	2384	1835	549	BEDROCK

8	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	28.8	60.3	0.9	29.2	12.2	HP310x110	3420	2570	851	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	26.9	56.3	0.7	38.1	12.4	HP310x110	3350	2503	847	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	29.1	60.9	1.0	25.4	11.5	HP310x110	2803	2135	668	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	30.5	63.8	1.0	25.4	11.2	HP310x110	3232	2480	752	BEDROCK
9	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	25.6	53.5	0.6	42.3	15.4	HP310x125	2464	2039	425	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	28.9	60.4	1.0	25.4	15.0	HP310x125	2342	2005	337	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	23.4	49.0	1.0	25.4	14.3	HP310x125	2520	2025	494	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	21.3	44.6	2.0	12.7	16.1	HP310x125	2410	2055	355	BEDROCK
10	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	24.6	51.4	2.1	11.9	19.0	HP310x132	3001	1901	1100	BEDROCK
11	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	28.1	58.8	1.7	14.9	20.3	HP310x132	2901	2541	360	BEDROCK
	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	26.4	55.2	1.0	25.4	20.3	HP310x132	3201	2801	400	BEDROCK
12	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	24.0	50.2	1.3	19.5	19.9	HP310x125	2130	1586	544	SAND TILL
	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	22.0	46.1	1.0	25.4	20.9	HP310x125	2166	1671	495	SAND TILL
	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	24.1	50.3	1.7	14.9	31.6	HP310x125	2783	2363	419	BEDROCK
	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	23.5	49.1	1.3	19.5	31.5	HP310x125	2525	2184	341	BEDROCK
	EOID	Pileco D19-42	ICH (Diesel)	#4	57.6	28.1	48.9	0.9	28.2	29.8	HP310x125	2724	1840	883	BEDROCK
	EOID	Pileco D19-42	ICH (Diesel)	#4	57.6	30.3	52.7	1.0	25.4	29.5	HP310x125	2668	2145	523	BEDROCK
	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	25.6	53.6	0.8	33.9	22.9	HP310x125	2363	1737	626	SAND TILL
	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	23.9	50.1	1.4	18.1	22.5	HP310x125	2457	1942	515	SAND TILL
	EOID	Pileco D19-42	ICH (Diesel)	#3	47.8	31.6	66.1	2.1	12.2	20.9	HP310x125	3157	2850	307	BEDROCK
	EOID	Delmag D19-32	ICH (Diesel)	#3	47.1	17.9	37.9	1.0	25.4	20.9	HP310x125	2450	2239	211	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	21.1	44.2	1.0	25.4	30.0	HP310x125	2267	1637	629	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	24.5	51.2	1.2	21.2	24.0	HP310x125	1737	1037	700	SAND TILL
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	24.0	50.1	1.4	18.1	24.0	HP310x125	1850	1189	662	SAND TILL
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	22.2	46.3	0.8	33.9	22.5	HP310x125	2171	1631	540	SAND TILL
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	25.4	53.1	0.3	84.7	21.1	HP310x125	3166	2625	541	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	22.4	46.8	0.6	42.3	20.9	HP310x125	3057	2650	407	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	26.2	54.8	0.6	42.3	21.3	HP310x125	2825	2228	597	BEDROCK
	BOR	Pileco D19-42	ICH (Diesel)	#3	47.8	20.9	43.6	0.7	36.3	22.3	HP310x125	2746	2168	578	BEDROCK
	EOID	Delmag D19-32	ICH (Diesel)	#4	57.6	18.7	32.5	0.9	28.2	29.7	HP310x125	2750	2190	560	BEDROCK

Table 5.2: Results of the dynamic RAM correlation study.

Site	Observed Set	Blow Count	Pile Size	CAPWAP			Case Method						GRLWEAP				Modified Gates Formula				End Bearing Strata
				Ultimate Capacity	Ultimate End Bearing Capacity	Ultimate Shaft Capacity	Predicted			Refined			Predicted		Refined		Predicted		Refined		
	mm	BPI		kN	kN	kN	Jc	kN	Correlation	RMX	kN	Correlation	kN	Correlation	kN	Correlation	kN	Correlation	kN	Correlation	
1	1.00	25.4	HP310x110	2853	1900	953	0.80	2800	0.98	RX8	2800	0.98	2300	0.81	2535	0.89	1945	0.68	2745	0.96	BEDROCK
	1.40	18.1	HP310x110	2976	2202	774	0.80	2927	0.98	RX7	3007	1.01	2150	0.72	2380	0.80	1800	0.60	2552	0.86	BEDROCK
	0.60	42.3	HP310x132	3501	2958	543	0.80	3799	1.08	RA2	3637	1.04	3040	0.87	3390	0.97	2229	0.64	3040	0.87	BEDROCK
	1.00	25.4	HP310x132	3300	2593	707	0.80	3647	1.11	RX9	3559	1.08	2760	0.84	3030	0.92	2004	0.61	2745	0.83	BEDROCK
	1.00	25.4	HP310x132	3302	2932	370	0.80	3838	1.16	RA2	3656	1.11	2760	0.84	3030	0.92	2004	0.61	2745	0.83	BEDROCK
	1.40	18.1	HP310x94	2871	2278	593	0.80	3259	1.14	RA2	3051	1.06	2155	0.75	2300	0.80	1904	0.66	2552	0.89	BEDROCK
2	0.60	42.3	HP200x54	1614	836	778	0.70	1584	0.98	RX4	1615	1.00	1240	0.77	1390	0.86	1876	1.16	2649	1.64	BEDROCK
	1.00	25.4	HP360x132	2862	1663	1198	0.70	2885	1.01	RX8	2866	1.00	3030	1.06	3250	1.14	2733	0.95	3761	1.31	BEDROCK
	0.80	31.8	HP360x132	2302	1567	735	0.70	2338	1.02	RX8	2303	1.00	3145	1.37	3335	1.45	2861	1.24	3931	1.71	BEDROCK
	2.00	12.7	HP360x132	2381	1449	932	0.70	2472	1.04	RX9	2412	1.01	2700	1.13	2935	1.23	2335	0.98	3235	1.36	BEDROCK
	0.40	63.5	HP360x132	4325	2888	1437	0.70	4571	1.06	RX9	4318	1.00	3400	0.79	3530	0.82	3259	0.75	4457	1.03	BEDROCK
	0.20	127.0	HP360x132	4037	3684	353	0.70	4220	1.05	RX8	4087	1.01	3600	0.89	3690	0.91	3657	0.91	4984	1.23	BEDROCK
3	1.40	18.1	HP310x110	2332	1661	671	0.70	2978	1.28	RA2	2364	1.01	2260	0.97	2600	1.11	1865	0.80	2703	1.16	BEDROCK
4	1.00	25.4	HP310x110	2405	2155	250	0.80	2789	1.16	RA2	2283	0.95	2180	0.91	2250	0.94	1589	0.66	2388	0.99	BEDROCK
	0.50	50.8	HP310x110	2543	2254	289	0.80	2994	1.18	RA2	2812	1.11	2430	0.96	2535	1.00	1843	0.72	2743	1.08	BEDROCK
	1.00	25.4	HP310x110	2907	2587	321	0.80	3369	1.16	RA2	3177	1.09	2180	0.75	2250	0.77	1589	0.55	2388	0.82	BEDROCK
6	1.30	19.5	HP360x132	2537	2200	337	0.70	2965	1.17	RAU	2611	1.03	2975	1.17	2710	1.07	1893	0.75	2147	0.85	BEDROCK
	1.50	16.9	HP360x132	2348	1776	573	0.70	2579	1.10	RX9	2495	1.06	2810	1.20	2575	1.10	1829	0.78	2077	0.88	BEDROCK
	2.60	9.8	HP360x132	1964	1587	377	0.70	1999	1.02	RX8	1967	1.00	2190	1.11	2000	1.02	1586	0.81	1807	0.92	BEDROCK
	1.00	25.4	HP360x132	3482	3192	290	0.70	3812	1.09	RAU	3464	0.99	3330	0.96	3270	0.94	2203	0.63	2629	0.75	BEDROCK
	2.00	12.7	HP360x132	3233	3047	186	0.70	3505	1.08	RAU	3325	1.03	2650	0.82	2710	0.84	1871	0.58	2244	0.69	BEDROCK
7	1.60	15.9	HP310x110	2514	1400	1113	0.70	2458	0.98	RX6	2534	1.01	2135	0.85	2270	0.90	2007	0.80	2321	0.92	SILT TILL
	2.00	12.7	HP310x110	2267	1137	1130	0.70	2370	1.05	RX9	2282	1.01	2030	0.90	2200	0.97	1899	0.84	2200	0.97	SILT TILL
	2.40	10.6	HP310x110	2254	1395	859	0.70	2368	1.05	RX9	2277	1.01	2460	1.09	2125	0.94	1811	0.80	2100	0.93	SILT TILL
	6.00	4.2	HP310x110	1860	1023	837	0.70	1929	1.04	RX9	1889	1.02	1510	0.81	1650	0.89	1367	0.74	1600	0.86	SILT TILL
	1.80	14.1	HP310x110	2217	1476	741	0.70	2183	0.98	RA2	2219	1.00	2065	0.93	2230	1.01	1950	0.88	2257	1.02	SILT TILL
	2.00	12.7	HP310x110	2094	1355	739	0.70	2112	1.01	RX8	2077	0.99	2030	0.97	2200	1.05	1899	0.91	2200	1.05	SILT TILL
	2.20	11.5	HP310x110	2095	1390	704	0.70	2183	1.04	RX9	2139	1.02	1990	0.95	2160	1.03	1853	0.88	2148	1.03	SILT TILL
	1.00	25.4	HP310x110	2384	1835	549	0.70	2758	1.16	RX9	2540	1.07	2300	0.96	2555	1.07	2234	0.94	3139	1.32	BEDROCK
8	0.87	29.2	HP310x110	3421	2570	851	0.80	3543	1.04	RX9	3483	1.02	2770	0.81	2750	0.80	2234	0.65	2706	0.79	BEDROCK
	0.67	38.1	HP310x110	3350	2503	847	0.80	3532	1.05	RA2	3338	1.00	2885	0.86	2860	0.85	2357	0.70	2854	0.85	BEDROCK
	1.00	25.4	HP310x110	2803	2135	668	0.80	3868	1.38	RA2	3119	1.11	2695	0.96	2680	0.96	2168	0.77	2629	0.94	BEDROCK
	1.00	25.4	HP310x110	3232	2480	752	0.80	3450	1.07	RX9	3354	1.04	2695	0.83	2680	0.83	2168	0.67	2629	0.81	BEDROCK
9	0.60	42.3	HP310x125	2464	2039	425	0.80	2684	1.09	RA2	2487	1.01	3220	1.31	3210	1.30	2275	0.92	2912	1.18	BEDROCK
	1.00	25.4	HP310x125	2342	2005	337	0.80	2432	1.04	RX9	2353	1.00	2915	1.24	3040	1.30	2046	0.87	2629	1.12	BEDROCK
	1.00	25.4	HP310x125	2520	2025	494	0.80	3013	1.20	RA2	2624	1.04	2915	1.16	3040	1.21	2046	0.81	2629	1.04	BEDROCK
	2.00	12.7	HP310x125	2410	2055	355	0.80	2489	1.03	RX9	2396	0.99	2390	0.99	2500	1.04	1734	0.72	2244	0.93	BEDROCK

10	2.14	11.9	HP310x132	3001	1901	1100	0.80	2858	0.95	RX3	2998	1.00	2080	0.69	2020	0.67	1722	0.57	2207	0.74	BEDROCK
11	1.70	14.9	HP310x132	2901	2541	360	0.80	3404	1.17	RA2	2916	1.01	2460	0.85	2500	0.86	1863	0.64	2334	0.80	BEDROCK
	1.00	25.4	HP310x132	3201	2801	400	0.80	3478	1.09	RX9	3370	1.05	2850	0.89	2900	0.91	2108	0.66	2629	0.82	BEDROCK
12	1.30	19.5	HP310x125	2130	1586	544	0.60	2168	1.02	RA2	2139	1.00	2610	1.23	2085	0.98	1898	0.89	2025	0.95	SAND TILL
	1.00	25.4	HP310x125	2166	1671	495	0.60	2666	1.23	RX9	2275	1.05	2775	1.28	2185	1.01	2014	0.93	2148	0.99	SAND TILL
	1.70	14.9	HP310x125	2783	2363	419	0.70	2279	0.82	RX3	2790	1.00	2430	0.87	2525	0.91	1821	0.65	2334	0.84	BEDROCK
	1.30	19.5	HP310x125	2525	2184	341	0.70	1996	0.79	RX3	2570	1.02	2630	1.04	2730	1.08	1943	0.77	2483	0.98	BEDROCK
	0.90	28.2	HP310x125	2724	1840	883	0.80	2197	0.81	RX5	2647	0.97	2690	0.99	2900	1.06	2240	0.82	2993	1.10	BEDROCK
	1.00	25.4	HP310x125	2668	2145	523	0.80	1781	0.67	RX4	2754	1.03	2640	0.99	2850	1.07	2190	0.82	2929	1.10	BEDROCK
	0.75	33.9	HP310x125	2363	1737	626	0.70	2440	1.03	RX8	2291	0.97	2700	1.14	2175	0.92	2142	0.91	2283	0.97	SAND TILL
	1.40	18.1	HP310x125	2457	1942	515	0.70	2247	0.91	RX5	2448	1.00	2380	0.97	1930	0.79	1865	0.76	1990	0.81	SAND TILL
	2.08	12.2	HP310x125	3157	2850	307	0.70	3429	1.09	RX9	3220	1.02	2235	0.71	2300	0.73	1734	0.55	2191	0.69	BEDROCK
	1.00	25.4	HP310x125	2450	2239	211	0.70	3087	1.26	RX9	2874	1.17	2660	1.09	2520	1.03	1961	0.80	2276	0.93	BEDROCK
	1.00	25.4	HP310x125	2267	1637	629	0.80	1781	0.79	RX5	2198	0.97	2445	1.08	2670	1.18	1945	0.86	2629	1.16	BEDROCK
	1.20	21.2	HP310x125	1737	1037	700	0.80	1704	0.98	RX8	1704	0.98	2360	1.36	2050	1.18	1866	1.07	2063	1.19	SAND TILL
	1.40	18.1	HP310x125	1850	1189	662	0.80	1828	0.99	RX8	1828	0.99	2375	1.28	1985	1.07	1800	0.97	1990	1.08	SAND TILL
	0.75	33.9	HP310x125	2171	1631	540	0.70	2063	0.95	RX6	2170	1.00	2700	1.24	2175	1.00	2142	0.99	2283	1.05	SAND TILL
	0.30	84.7	HP310x125	3166	2625	541	0.70	3289	1.04	RX8	3180	1.00	3300	1.04	3425	1.08	2613	0.83	3254	1.03	BEDROCK
	0.60	42.3	HP310x125	3057	2650	407	0.70	3115	1.02	RX8	3007	0.98	3040	0.99	3140	1.03	2298	0.75	2873	0.94	BEDROCK
	0.60	42.3	HP310x125	2825	2228	597	0.70	2882	1.02	RX7	2882	1.02	3040	1.08	3140	1.11	2298	0.81	2873	1.02	BEDROCK
	0.70	36.3	HP310x125	2746	2168	578	0.70	2364	0.86	RX5	2676	0.97	2940	1.07	3050	1.11	2229	0.81	2789	1.02	BEDROCK
	0.90	28.2	HP310x125	2750	2190	560	0.70	2629	0.96	RX5	2780	1.01	3000	1.09	2890	1.05	2212	0.80	2620	0.95	BEDROCK

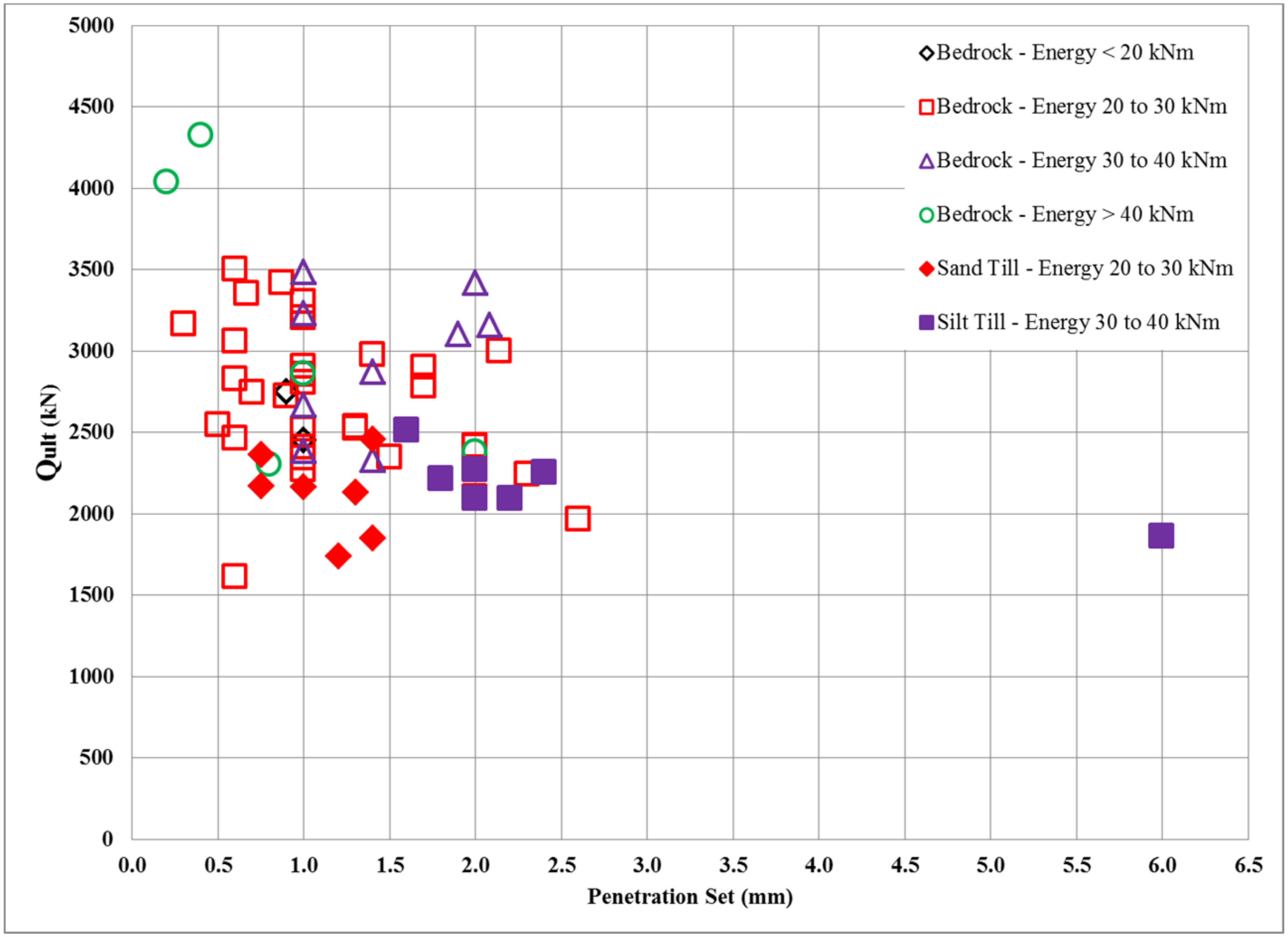


Figure 5.1: General bearing graph of Q_{ult} versus penetration set. Categorized by transferred energy and end bearing stratum.

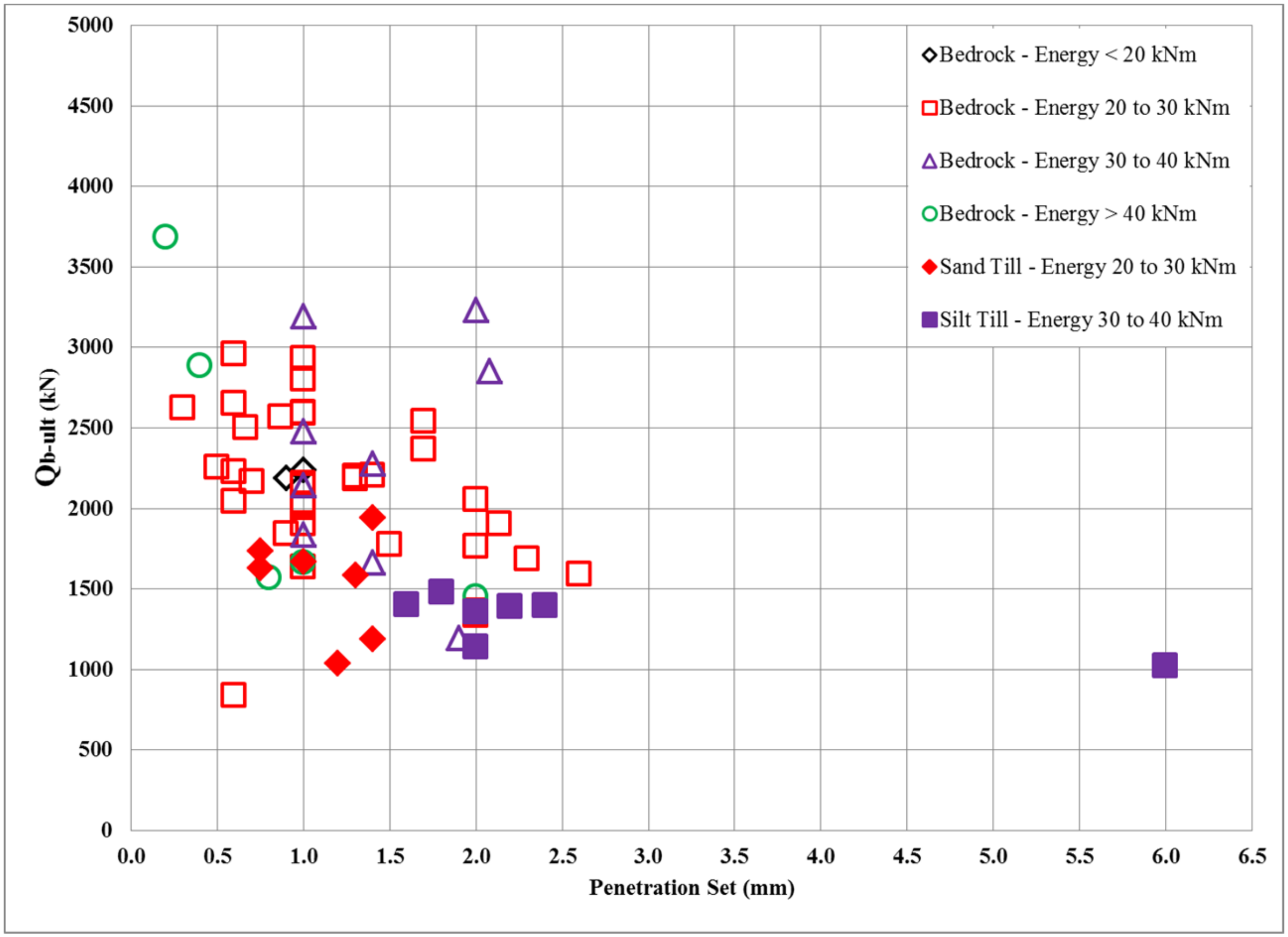


Figure 5.2: General bearing graph of $Q_{b\text{-}ult}$ versus penetration set. Categorized by transferred energy and end bearing stratum.

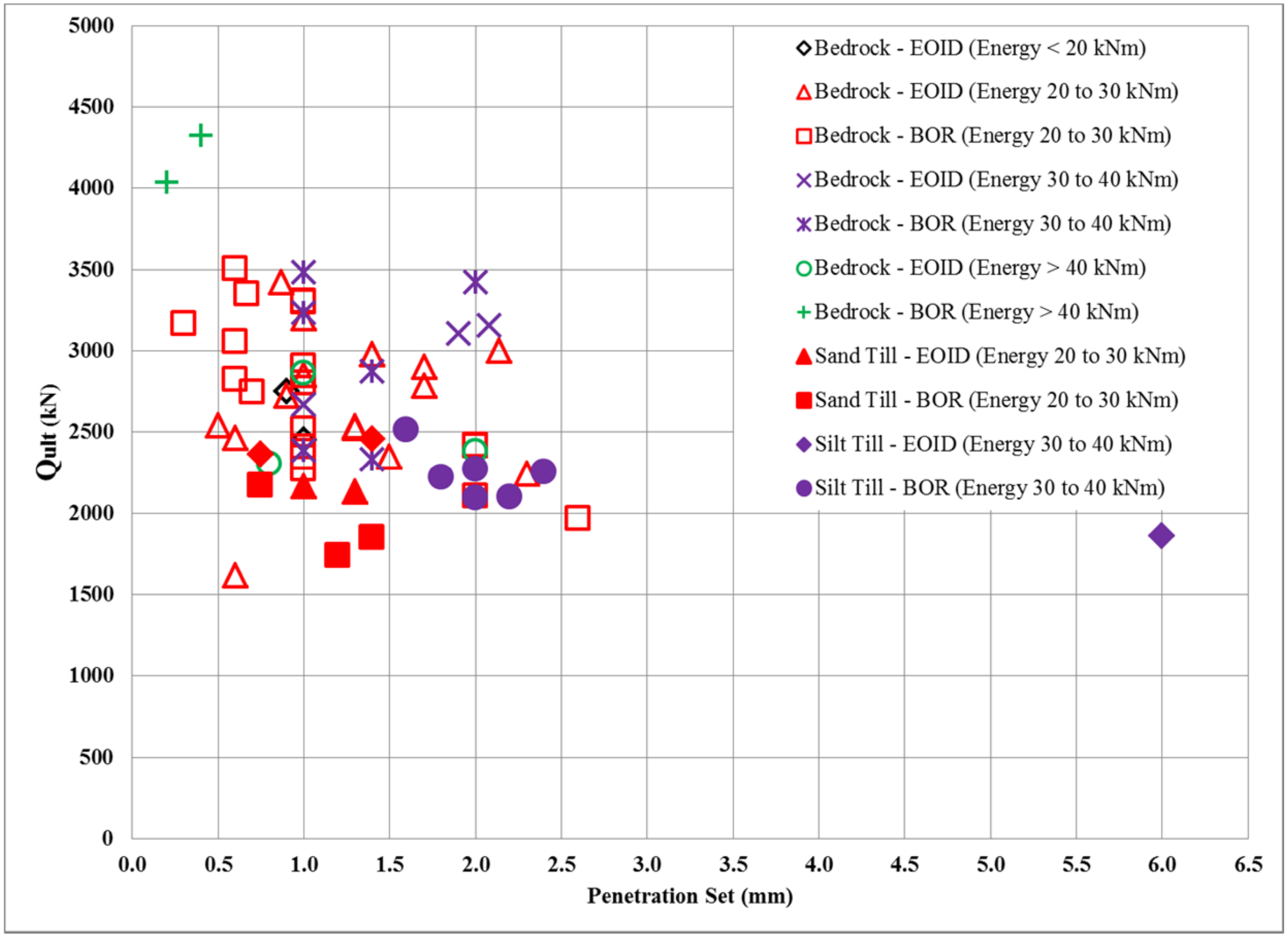


Figure 5.3: General bearing graph of Q_{ult} versus penetration set. Categorized by transferred energy, end bearing stratum, and test condition.

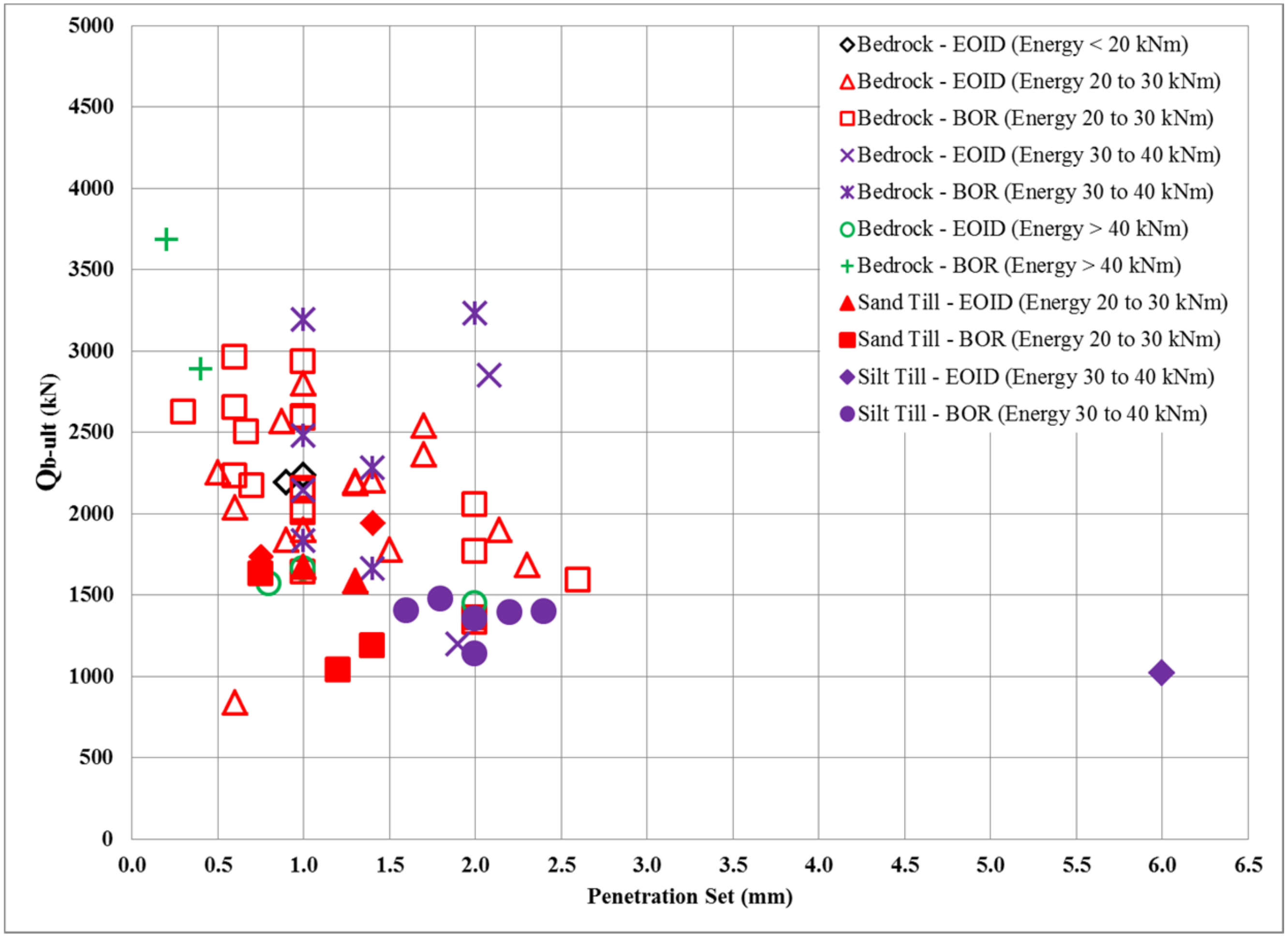


Figure 5.4: General bearing graph of $Q_{b\text{-}ult}$ versus penetration set. Categorized by transferred energy, end bearing stratum, and test condition.

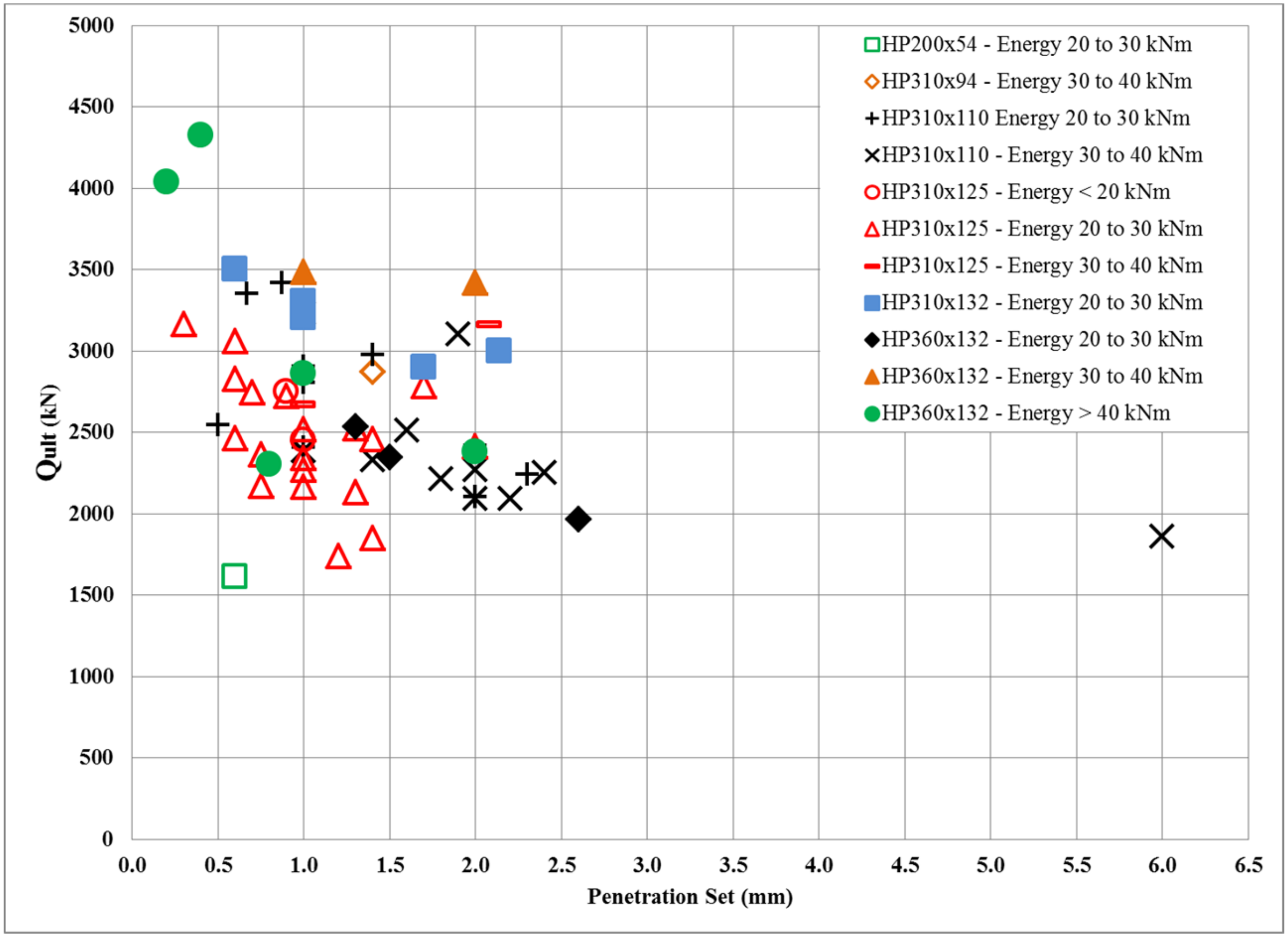


Figure 5.5: General bearing graph of Q_{ult} versus penetration set. Categorized by transferred energy and H-pile size.

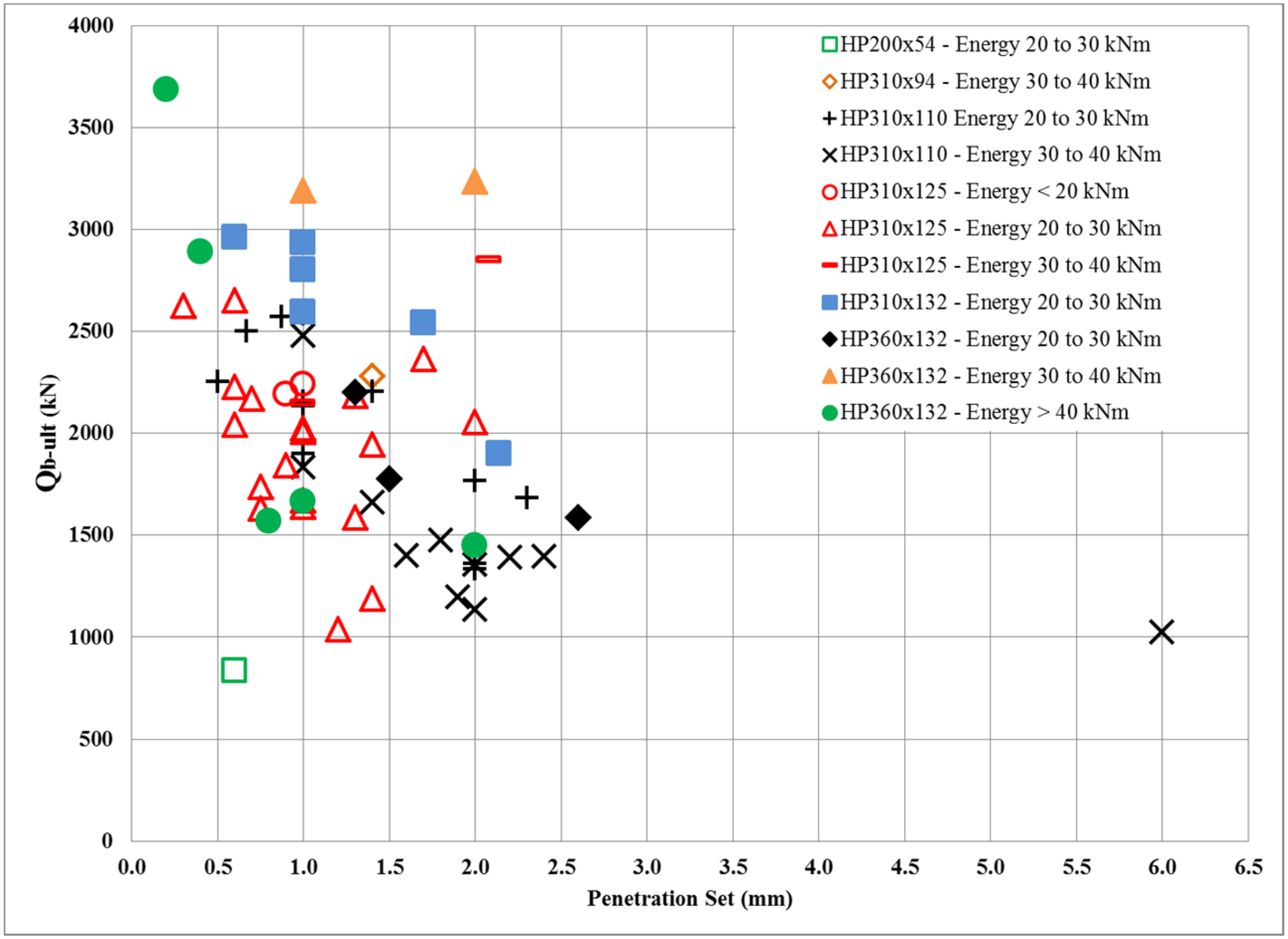


Figure 5.6: General bearing graph of $Q_{b\text{-}ult}$ versus penetration set. Categorized by transferred energy and H-pile size.

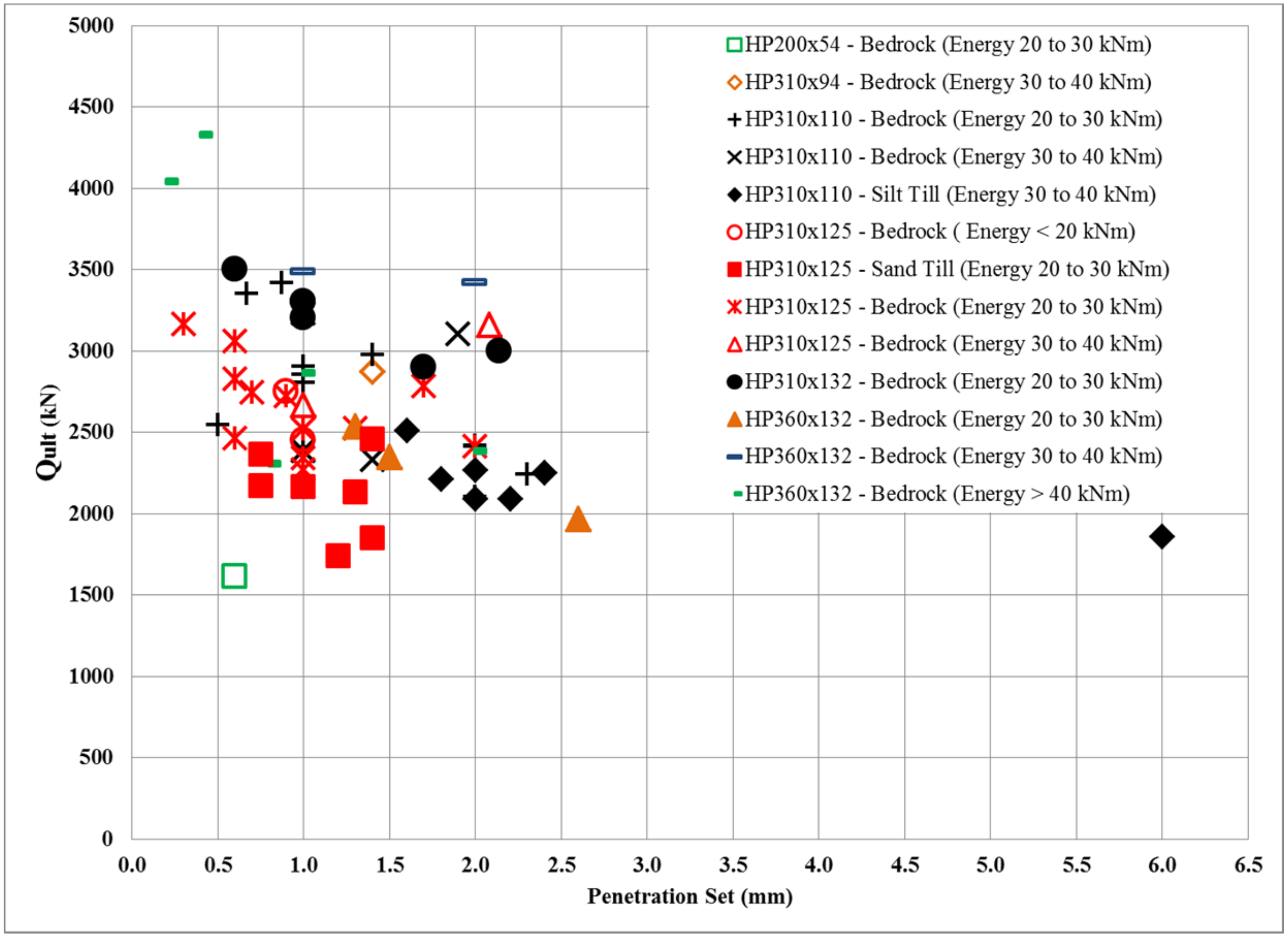


Figure 5.7: General bearing graph of Q_{ult} versus penetration set. Categorized by transferred energy, end bearing stratum, and H-pile size.

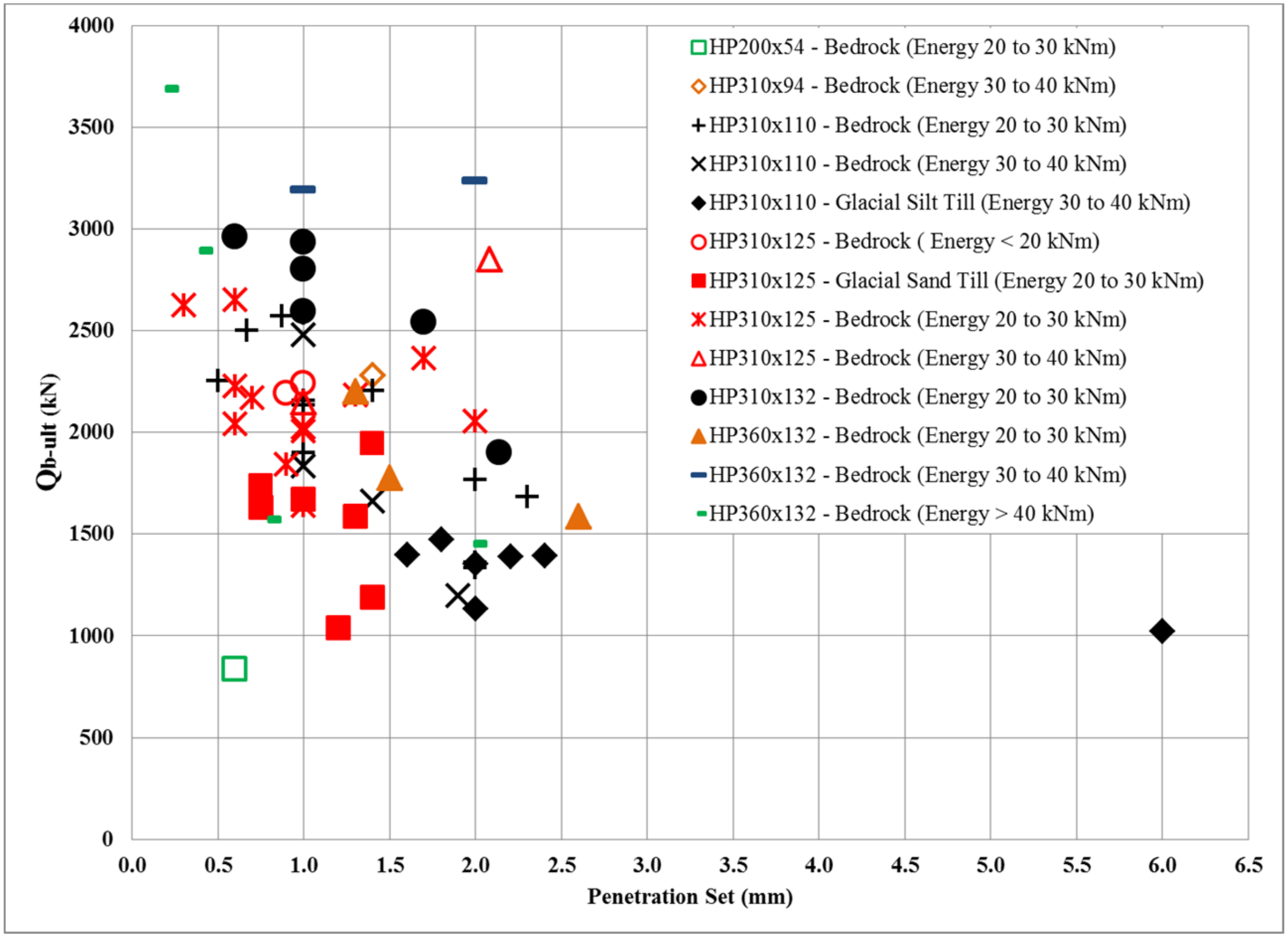


Figure 5.8: General bearing graph of $Q_{b\text{-}ult}$ versus penetration. Categorized by transferred energy, end bearing stratum, and H-pile size.

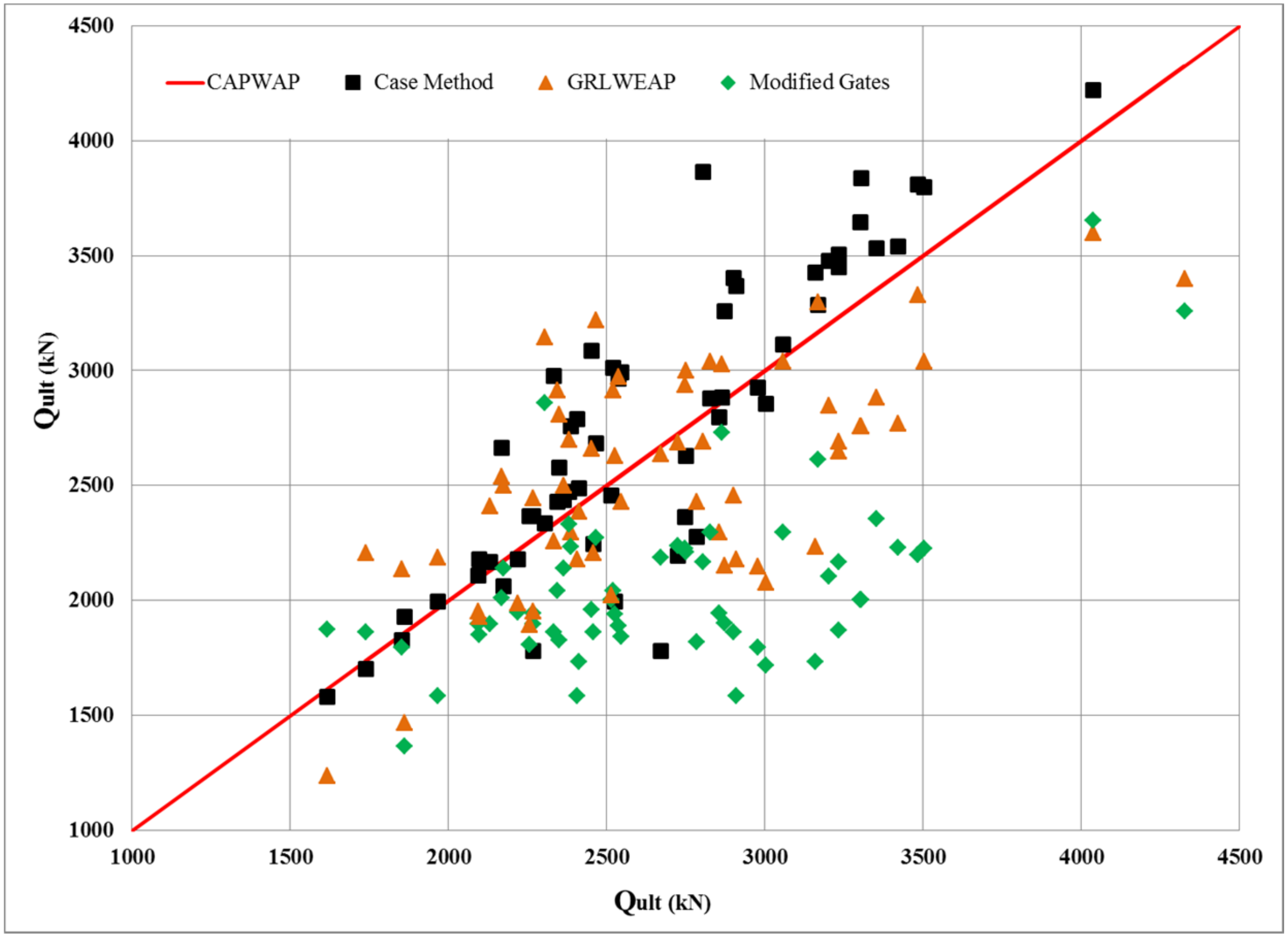


Figure 5.9: Correlation of the estimated Q_{ult} as determined by dynamic RAMs on the basis of recommended input properties, to the Q_{ult} calculated by CAPWAP.

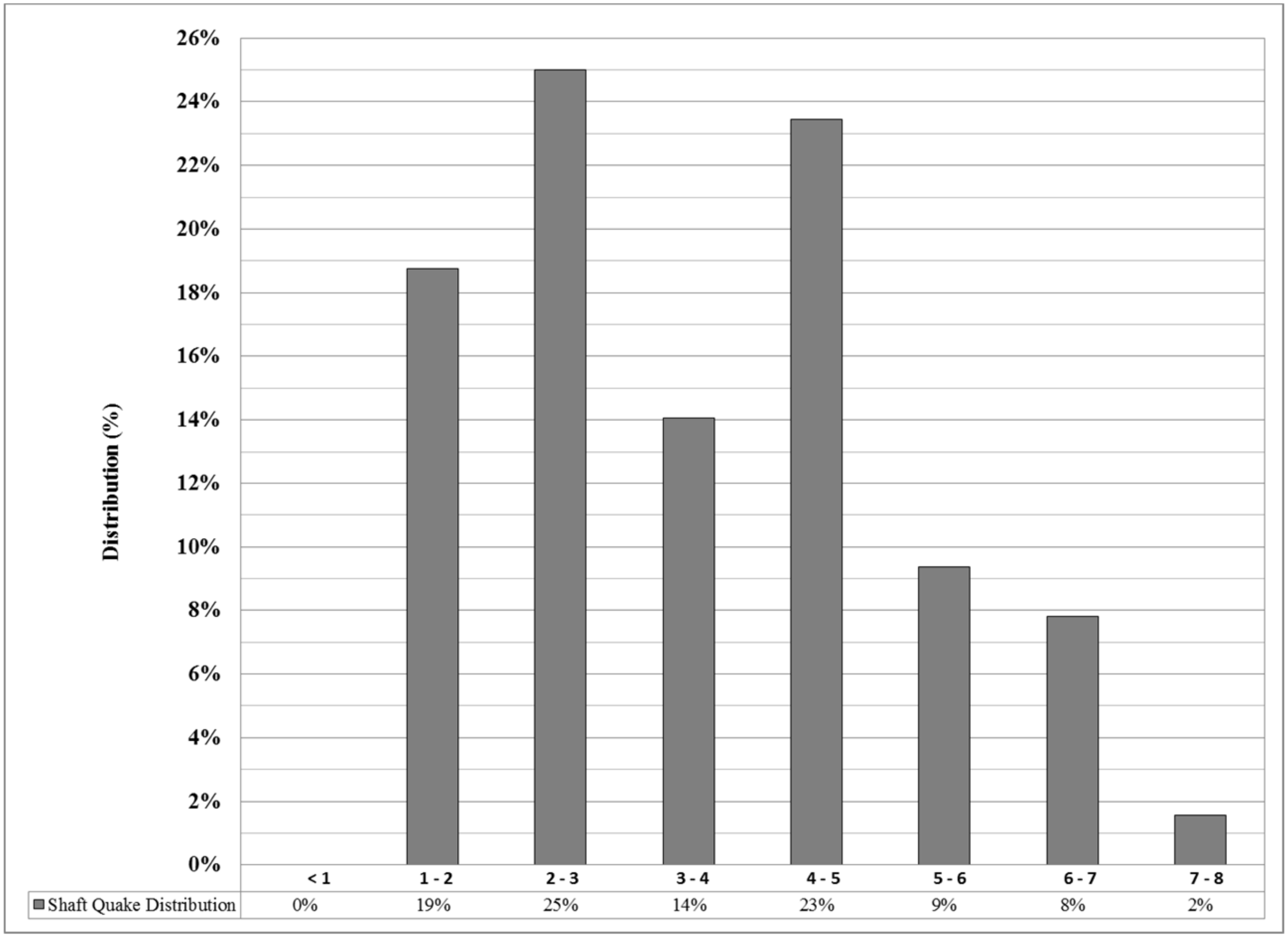


Figure 5.10: Distribution of CAPWAP-based quake values along the pile shaft.

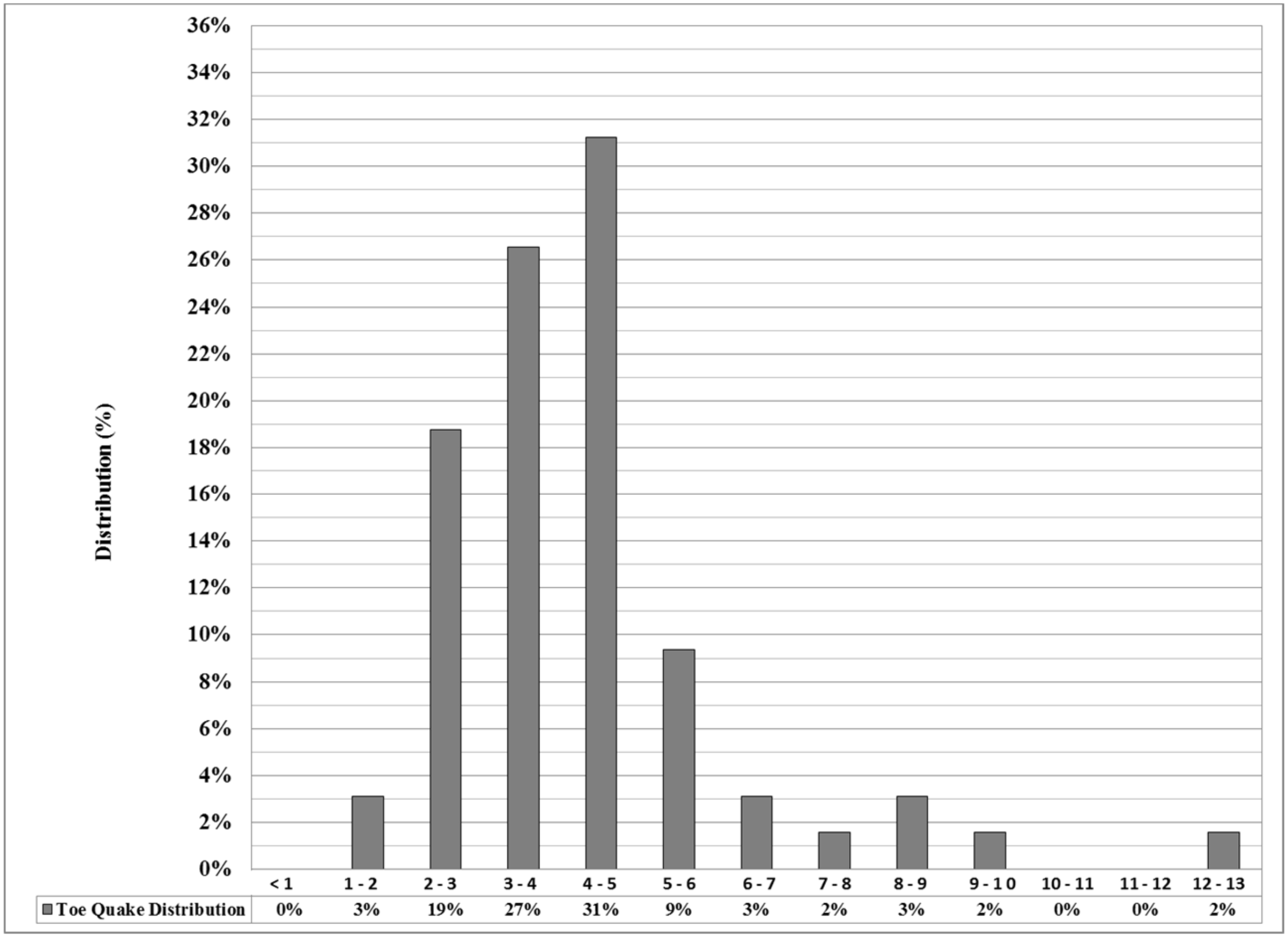
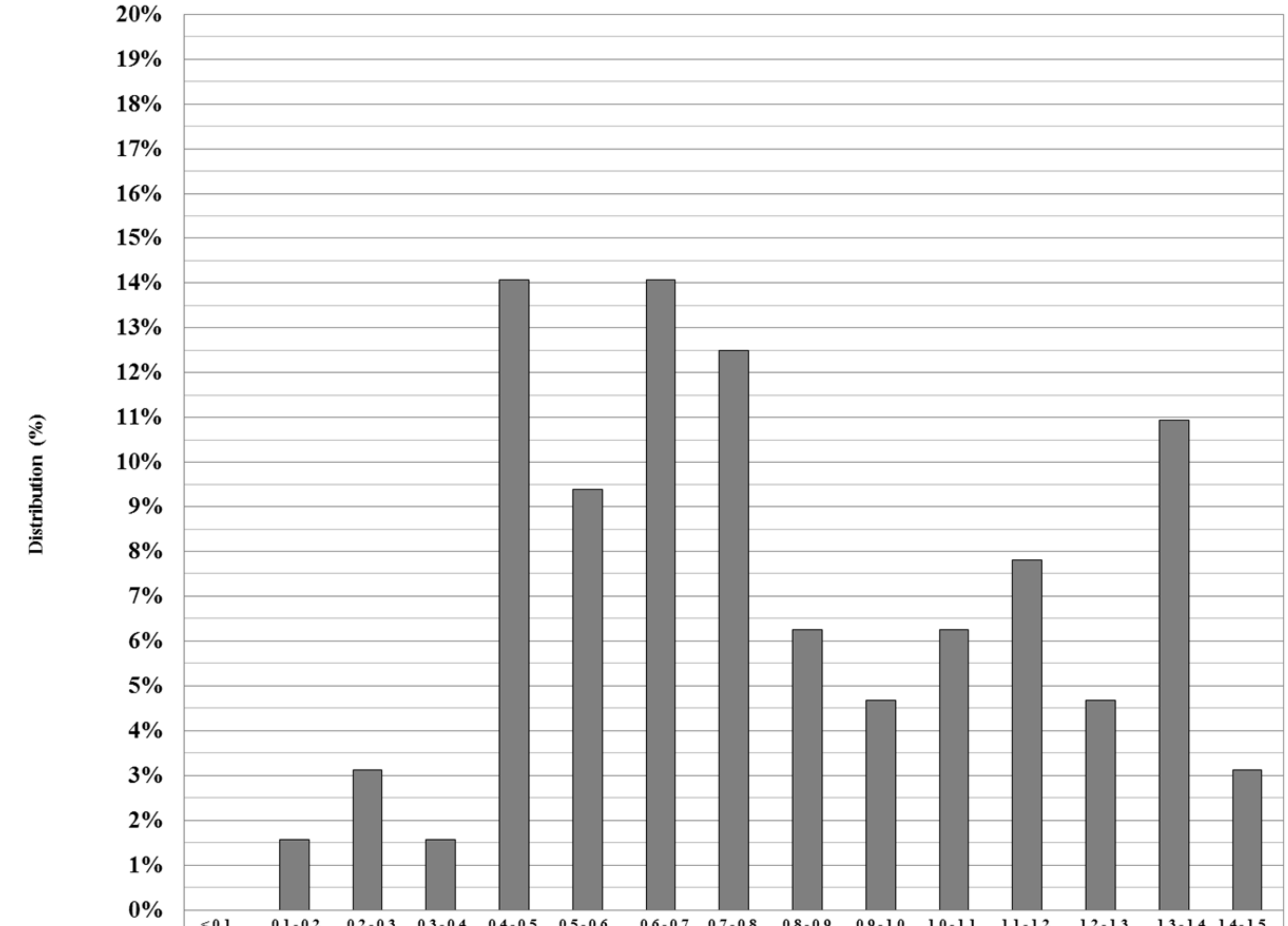


Figure 5.11: Distribution of CAPWAP-based quake values at the pile toe.



	< 0.1	0.1 - 0.2	0.2 - 0.3	0.3 - 0.4	0.4 - 0.5	0.5 - 0.6	0.6 - 0.7	0.7 - 0.8	0.8 - 0.9	0.9 - 1.0	1.0 - 1.1	1.1 - 1.2	1.2 - 1.3	1.3 - 1.4	1.4 - 1.5
Js Shaft Distribution	0%	2%	3%	2%	14%	9%	14%	13%	6%	5%	6%	8%	5%	11%	3%

Figure 5.12: Distribution of CAPWAP-based J_s values along the pile shaft.

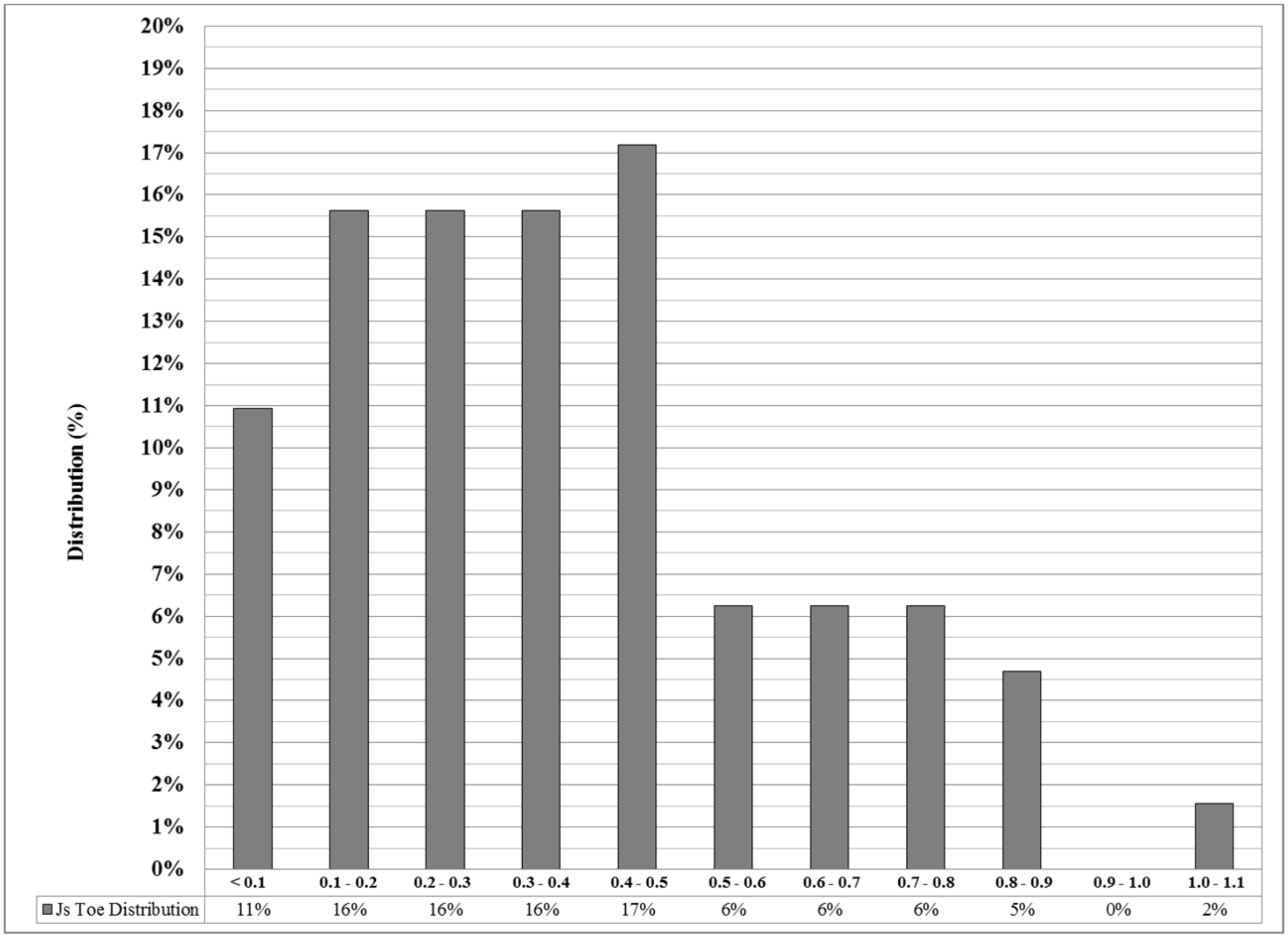


Figure 5.13: Distribution of CAPWAP-based J_s values at the pile toe.

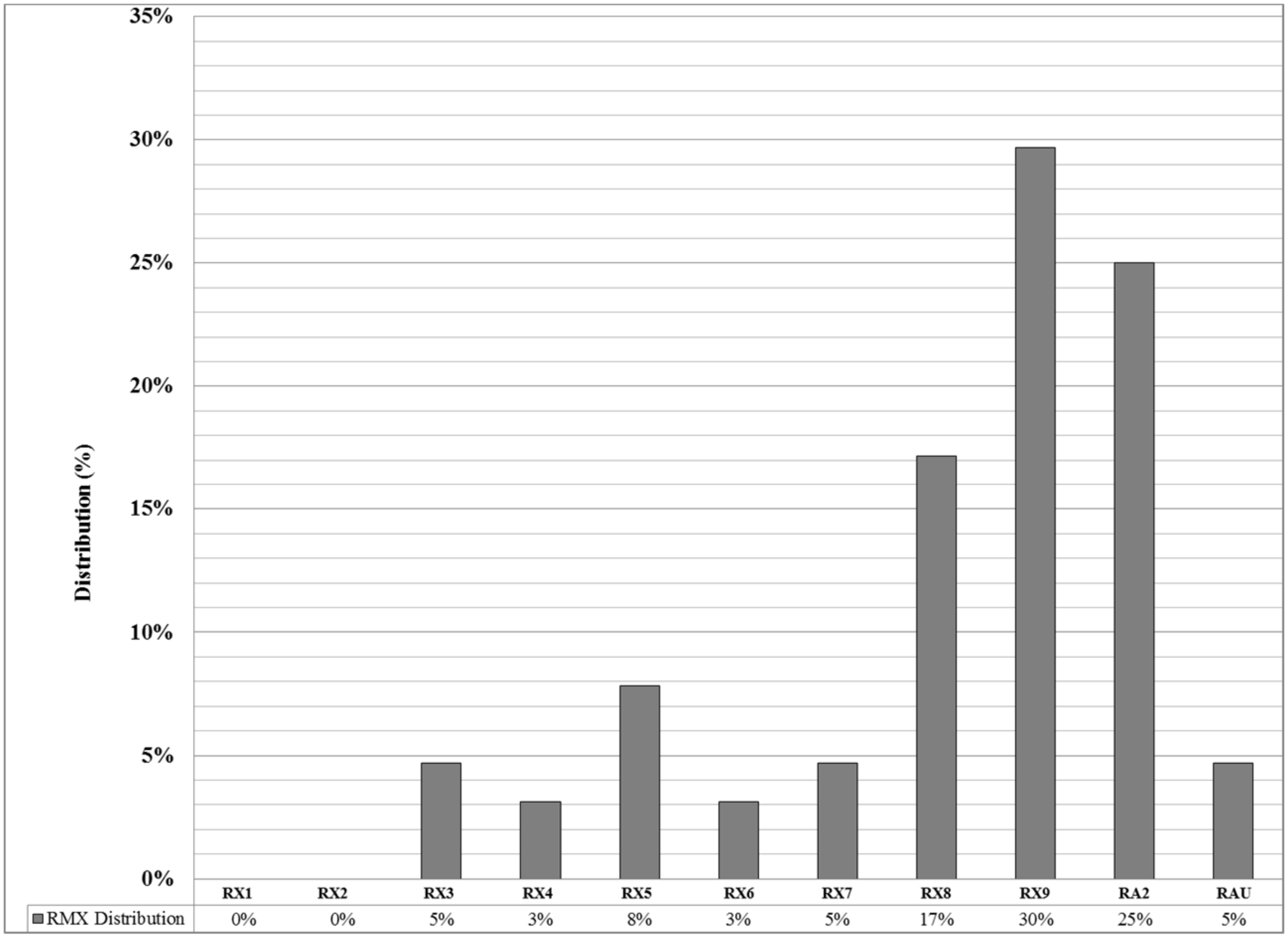


Figure 5.14: Distribution of the J_c values that produce the Q_{ult} that best correlates to the CAPWAP-calculated Q_{ult}.

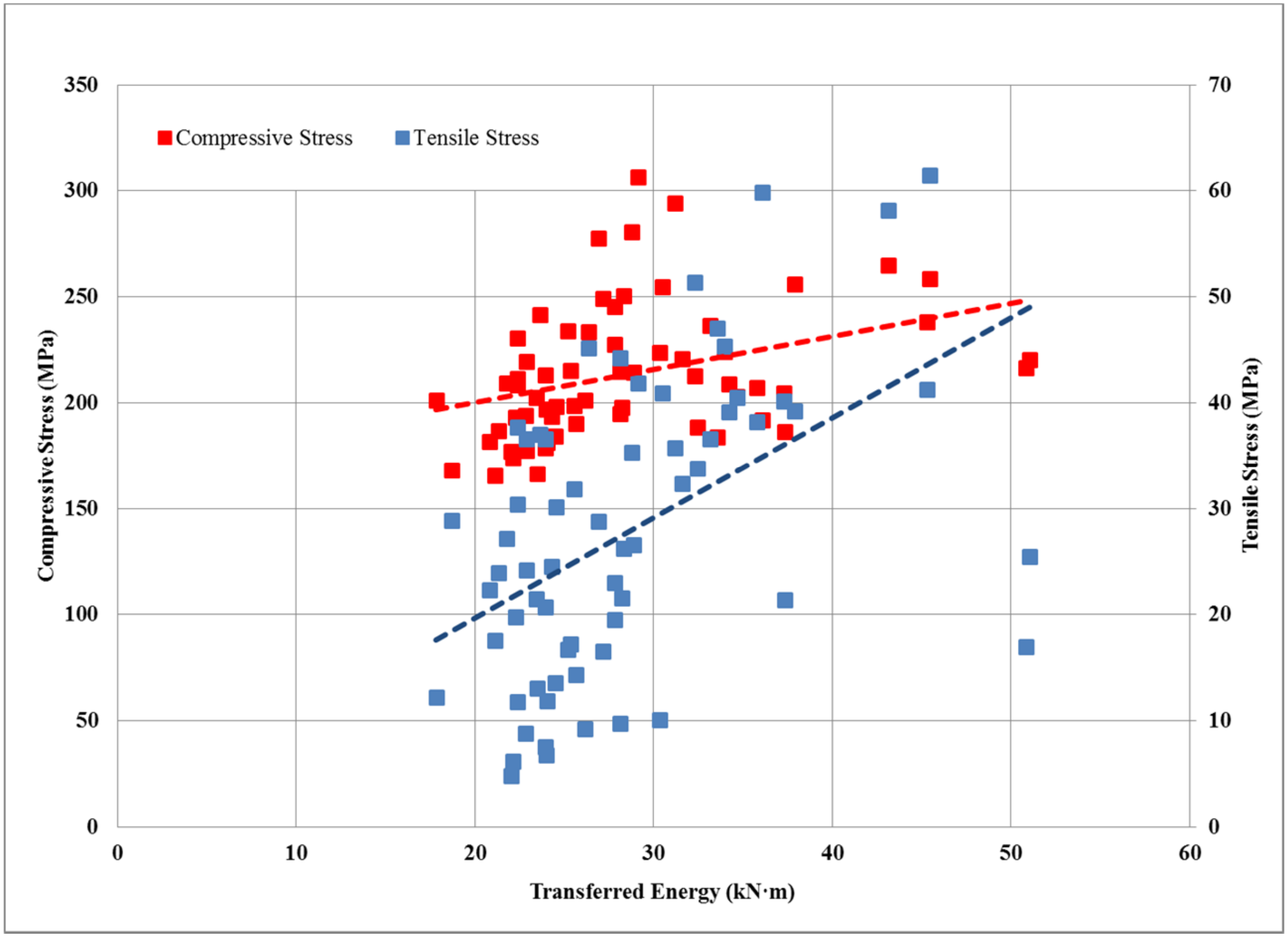


Figure 5.15: Transferred energy versus compressive and tensile driving stresses.

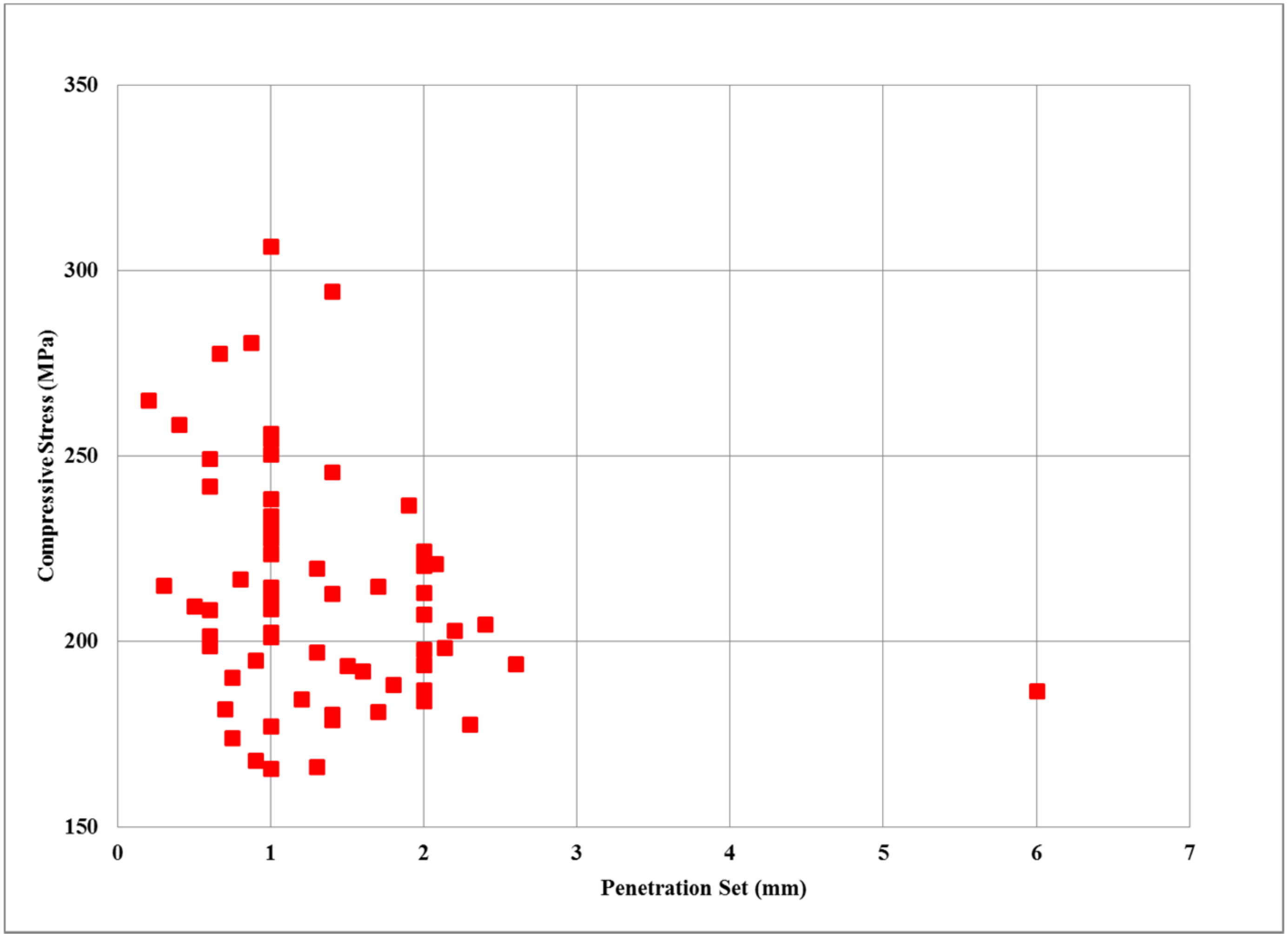


Figure 5.16: Penetration set versus compressive driving stress.

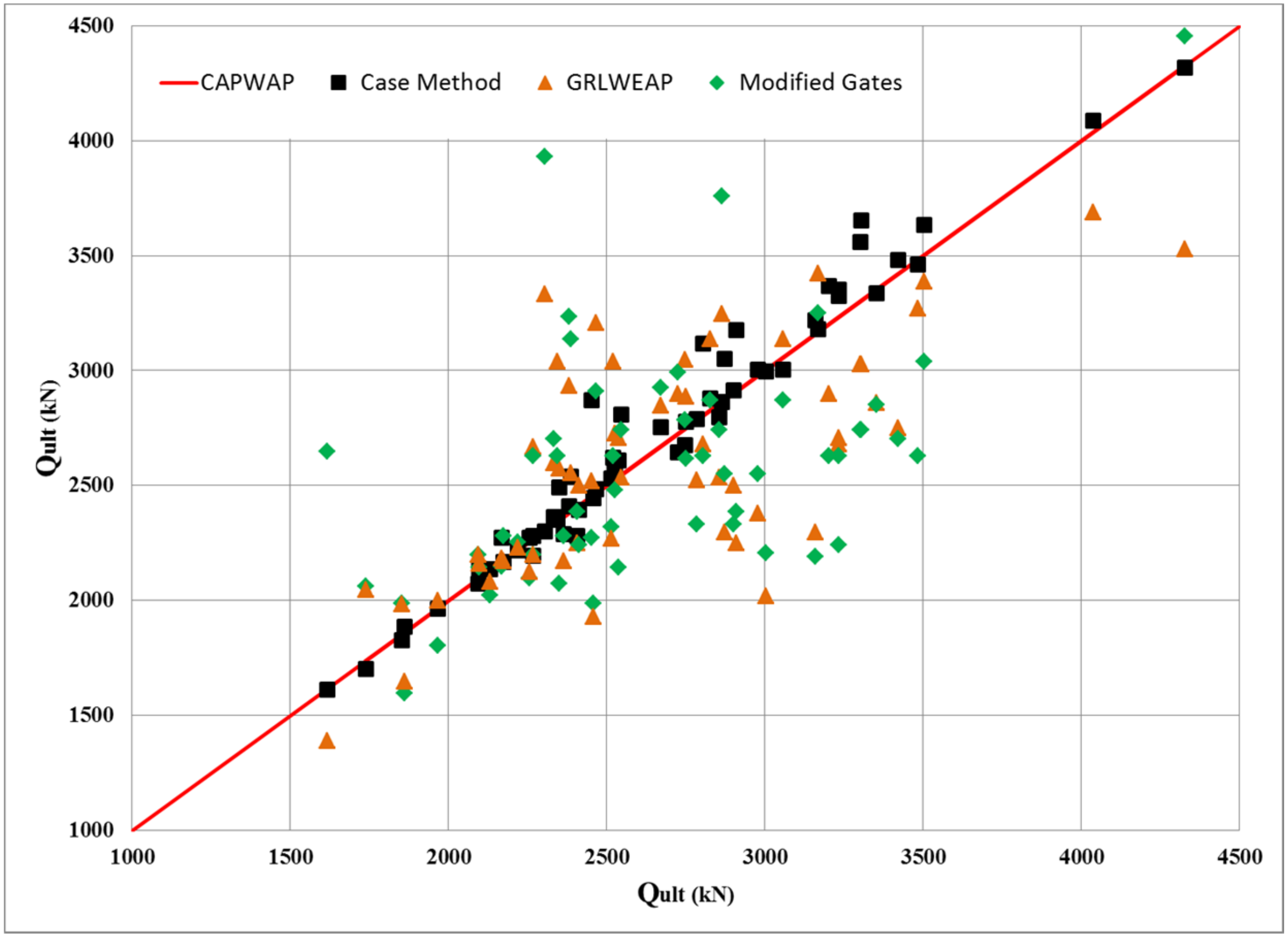


Figure 5.17: Correlation of the estimated Q_{ult} as determined by dynamic RAMs on the basis of measured input properties to the Q_{ult} calculated by CAPWAP.

Chapter 6: Serviceability of Driven Steel H-piles in Winnipeg

6.0 Introduction

The serviceability of driven steel H-piles in Winnipeg is covered in this chapter. Serviceability limit state capacity typically governs design and is based on the performance of the pile under applied service loads. The working stress design method was used, until recently, for design recommendations which included providing a single global formula (Equation 1.4). With the emergence of LSD, both SLS and ULS components of H-pile capacity must now be provided in geotechnical design recommendations. Chapter 1 discussed the conventional LSD criteria used in Winnipeg for driven steel H-piles and their shortcomings. Chapters 4 and 5 covered the ultimate capacities (*i.e.* Q_{ult}, $Q_{b\text{-}ult}$, and $Q_{s\text{-}ult}$) of driven steel H-piles under axial compressive loading. This chapter will discuss H-pile performance for the purpose of developing a SLS design criterion.

In this chapter, an investigation of the load-displacement behavior of driven steel H-piles in Winnipeg is undertaken. The investigation was completed by evaluating the ultimate capacities calculated using CAPWAP and the available load-displacement curves. The displacement and corresponding magnitudes of axial load at the top and bottom of the H-piles was reviewed. The displacements were assessed to determine the degree of loading that can be mobilized without jeopardizing the performance of the H-piles and, consequently, the structures they support.

6.1 Load-Displacement Behavior

Displacement of a pile under an applied load is generally referred to as settlement; which may be misleading when pile serviceability is the topic of discussion. Clarification of the term settlement in the context of this thesis must be differentiated from consolidation settlement. The author suggests that compressive settlement is a more fitting manner in which to describe pile settlement. Compressive settlement consists of two parts: elastic compression of the pile plus settlements associated with mobilization of base resistance. Compressive settlement must occur to mobilize pile capacity. These immediate mobilizing settlements are considered elastic and recoverable under service loads. Consolidation settlement can be considered as long-term, elastic-plastic, and non-recoverable movements that occur as a result of volume strains and shear strains in the soil or rock that surround the pile. In the context of this thesis, settlement will refer to compressive settlement, as described above. It should be noted that a majority of pile movements occur upon initial loading during the construction phase.

The criterion for serviceability assessment is based on the amount of settlement that is considered to be tolerable under service loads. The load-displacement curves are a useful tool for assessing the serviceability of driven steel H-piles under various loads. Analysis of the load-displacement curves consists of separating the top and bottom curves to understand where and how much settlement is occurring.

6.1.1 Pile Toe Settlement

Understanding the load-displacement behavior at the pile toe is important since pile parameters, soil stratigraphy, and load distribution vary from site to site. The settlements that occur at the pile toe are resistance mobilizing movements of the underlying soil. The degree of settlement at the pile toe depends on the strength of the soil at the toe and the load transferred to the toe. Toe settlement is independent of elastic compression and the amount of load shed along the pile shaft. The load-displacement behavior at the pile toe, as simulated in CAPWAP, is based on the toe quake and the $Q_{b\text{-}ult}$ of the pile.

Table 6.1, provides all the information used in the settlement analyses at the pile toe. The analysis consisted of assessing the amount of settlement at the pile toe at failure and the amount of end bearing capacity mobilized at what was considered by the author to be tolerable settlement. The amount of end bearing capacity mobilized at 1 and 2 mm of toe settlement for different H-pile sections were assessed and the results are discussed below. These values of toe settlement (*i.e.* 1 and 2 mm) were chosen for assessment because they did not induce failure in most (96.9 percent) of the H-piles. The data were not categorized in terms of the end bearing stratum since quake values did not correlate well to the different strata.

6.1.1.1 Pile Toe Settlements at Failure

It is important to note that there was no discernible difference between bedrock and sand till in terms of quake. The toe quake values, previously discussed in Chapter 5, determine the settlement at the H-pile toe. To summarize the results provided in Chapter

5, the pile toe settlements at failure range between 1.01 and 12.98 mm with an average value of 4.36 mm and a COV equal to 0.45. Failure did not occur at toe settlements of 1 mm or less, but were mobilized at, or less than, 2, 3, 4, 5, and 10 mm for 3.1, 21.8, 48.4, 75, and 98.4 percent of the H-piles, respectively. One H-pile had a toe settlement of 12.98 mm at failure. These statistics indicate that failure of most (*i.e.* 75 percent) of the H-piles occurred at more than 1 mm and at, or less than, 5 mm of settlement at the toe.

6.1.1.2 End Bearing Capacity Mobilized at Tolerable Settlements at the Pile Toe

Table 6.1 summarizes the results of the percentage of the $Q_{b\text{-}ult}$ that is mobilized at 1, 2, 3, 4, 5, and 10 mm of settlement at the pile toe for each analysed pile. Figures 6.1 to 6.5 illustrate the end bearing capacity mobilized at 1 and 2 mm of pile toe settlements for each H-pile size. Assessment of 1 and 2 mm of pile toe settlement is conservative but practical considering an increasing amount of piles reach failure at toe settlements greater than 2 mm. Considering all pile sizes, an average of 27.3 percent of the $Q_{b\text{-}ult}$ is mobilized at 1 mm of settlement at the H-pile toe. The overall average fraction of the $Q_{b\text{-}ult}$ that develops at the pile toe at 2 mm of toe settlement is 51 percent.

This concludes the discussion of the load-displacement behavior at the pile toe. The remainder of this chapter investigates the top-of-pile settlement.

6.1.2 Top-of-Pile Settlement

The top-of-pile load-displacement curve shows the total settlements that will occur in a driven steel H-pile in under service loading. The settlements that occur at the pile head are based on the Q_{ult} (failure), load distribution, elastic compression, and settlement at the pile toe. The load applied to the top of steel H-pile results in elastic compression which in turn mobilizes the shaft friction capacity. Load that is transferred to the pile toe results in settlement at the toe and mobilization of the end bearing capacity. The sum of the settlement at the toe and elastic compression of the shaft equals the top-of-pile settlement.

Elastic compression is essential for mobilization of soil resistance and, subsequently, development of pile capacity. It constitutes a majority of the overall top-of-pile settlement. Theoretically, it is based on the applied load, load distribution, Poisson's ratio of the soil and pile length, area, and elastic modulus (Vesic 1977). In CAPWAP, it is calculated using the pile properties, quake value, and static capacity at each element in the pile shaft. Since elastic compression is well documented in other literature, such as the CFEM (2006), an elaborate investigation of its parameters is not undertaken in this thesis. However, a brief and elementary summary of elastic compression measurements is provided to present its significance.

This part of the chapter analyzes the components of the top-of-pile settlement. Table 6.2, presents all the information used in the top-of-pile settlement analyses. The first analysis examines the influence that Q_{ult}, pile size, and pile length have on the amount of top-of-pile settlement. The ratio of amount of elastic compression to the top-of-pile settlement is investigated in the second analysis. The portion of the Q_{ult} mobilized at 5 and 10 mm settlement at the top-of-pile is then assessed. Fourth, the Q_{ult}, the capacity mobilized at tolerable settlements, and the conventional Q_{SLS} of the varying H-pile sizes are studied.

The top-of-pile settlements induced at the conventional Q_{SLS} are then determined and assessed. The analysis of the top-of-pile settlement is based on the load-displacement curves of twenty-nine CAPWAP tests; all the H-piles are seated in bedrock. The remaining CAPWAP analyses did not provide a load-displacement curve output and therefore could not be assessed.

6.1.2.1 Top-of-Pile Settlements at Failure

Ultimate capacity was reached at top-of-pile settlements ranging between 14.6 and 33.7 mm with an average of 21.4 mm and a COV equal to 0.21. These statistics do not signify much without analysing the influence that pile size and length have on the Q_{ult}.

Figure 6.6 shows that the cross-sectional area of the H-piles did not have an obvious influence on the amount of top-of-pile settlement that occurred under failure loads. This may be a result of the minor differences in the cross-sectional area between the H-pile sizes. Figure 6.7 shows that the Q_{ult} and H-pile length had a noticeable impact on the amount of top-of-pile settlement. According to Vesic (1977), the applied load and the pile length directly affect the amount of compressive settlement of a pile. Piles that are subjected to higher load or are longer in length will exhibit more compressive settlement than more lightly loaded piles and piles of shorter lengths, respectively. This occurs because elastic strain is directly related to length; therefore, the longer the pile the greater the strain and the settlement. Figure 6.7 illustrates that longer the piles have larger top-of-pile settlements.

Review of Figure 6.7 shows that, in general, higher Q_{ult} results in greater amounts of top-of-pile settlement. This behavior occurs because the top-of-pile settlement is directly related to the applied load (Vesic 1977). However, many H-piles exhibiting small Q_{ult} experienced large amounts of top-of-pile settlement and many H-piles exhibiting large Q_{ult} had relatively small amounts of top-of-pile settlement. Additionally, the results of Q_{ult} and top-of-pile settlement were not perfectly clear and some scatter is observed in the plotted data. The author believes that this is due to the influence of the heterogeneity of the soil and rock strength and the corresponding settlements that are required to mobilize the Q_{ult}.

The correlation between H-pile length and top of pile settlement is more distinct and direct with a closer-fitting trend present in plotted data. H-piles that had lengths less than 21.5 m had top-of-pile settlements less than 25 mm. H-piles that had top-of-pile settlements greater than 25 mm had lengths greater than 21.5 m. To further investigate the influence of length of the top-of-pile settlement, pile length intervals of less than 15 m, 15 to 20 m, and greater than 20 m were reviewed. H-piles with lengths less than 15 m have top-of-pile settlements ranging between 14.6 and 22.5 m with a mean value equal to 18.4 mm. H-piles with lengths between 15 and 20 m have top-of-pile settlements ranging between 17.9 and 24.6 mm with an average of 20.2 mm. Finally, H-piles with lengths greater than 20 m had top-of-pile settlements varying from 20.4 to 33.7 mm with a mean value of 27 mm.

6.1.2.1.1 Influence of Elastic Compression on Top-of-Pile Settlement

The amount of elastic compression that is simulated in the CAPWAP-based load-displacement curves was measured directly and found to be the main element of top-of-pile settlement as shown in Figure 6.8. The contribution of elastic compression to the overall amount of top-of-pile settlement ranges from 8.1 to 32.7 mm (50.4 to 97 percent) with a mean value of 16.7 mm (77.4 percent). The overall amount of elastic compression during mobilization of Q_{ult} constitutes between 0.07 and 0.13 percent of the total H-pile length with an average value of 0.1 percent and a COV of 0.16. There was no discernible difference in the ratio of elastic compression to pile length in terms of Q_{ult}, pile size, or pile length.

Not unlike the top-of-pile settlement, the amount of elastic compression that occurs under load is influenced by the magnitude of the Q_{ult} and the H-pile length. Figure 6.9 shows the relationship between Q_{ult}, H-pile length, and elastic compression. The same trends found, and discussed above, with top-of-pile settlement can be seen in this figure in regard to Q_{ult} and H-pile length. H-pile size did not appear to have a discernible influence in the amount of elastic compression.

6.1.2.2 Capacity Mobilized at Tolerable Top-of-Pile Settlements

The percentage of the Q_{ult} mobilized at top-of-pile settlements of 5 and 10 mm were investigated. These values were investigated because they were assumed, by the author, to be acceptable amounts of settlement under service loading. Figures 6.10 to 6.13 illustrate the relationship between Q_{ult}, the mobilized capacity at 5 and 10 mm of top-of-pile settlement, and the Q_{SLS} determined by the conventional design criterion for each H-pile size. At 5 mm of top-of-pile settlement, a range of 15.8 to 43.3 percent of the Q_{ult} is mobilized with an average of 29.3 percent. At 10 mm of top-of-pile settlement, a range of 31.6 to 80.9 percent of the Q_{ult} is mobilized with a mean value of 55.4 percent.

6.2 Assessment of the Conventional SLS Design Criterion

The conventional design criterion for the Q_{SLS} was compared to the Q_{ult}. Considering all sixty-four driven steel H-piles and ignoring the end bearing strata, the ratio of Q_{ult} to Q_{SLS} was measured between 1 and 2.5 with an average of 1.6 and a COV equal to 0.18. For H-piles seated in bedrock the same ratio ranges from 1.1 to 2.5 with a mean value equal to 1.7 and a COV of 0.17. H-piles seated in till exhibited a lower ratio of Q_{ult} to Q_{SLS}. H-piles in sand till had a ratio calculated between 1 and 1.5 with an average value equal to 1.3 and a COV equal to 0.12. H-piles seated in silt till revealed a ratio of Q_{ult} to Q_{SLS} ranging from 1.3 to 1.7 with a mean value of 1.5 and a COV equal to 0.09.

The twenty-nine CAPWAP-calculated load-displacement curves for H-piles driven to bedrock were evaluated to compare the Q_{SLS} at tolerable settlements to the Q_{ult}. The conventional Q_{SLS} values did not exceed the Q_{ult} values in any case. The conventional Q_{SLS} values, in the previous section, match up fairly well to the capacities mobilized at 10 mm of top-of-pile settlement as determined by the CAPWAP load-displacement curves. The top-of-pile settlement at which the conventional Q_{SLS} is mobilized ranges between 7.2 and 17.5 mm with a mean value equal to 10.6 mm and a COV equal to 0.21. A comparison between the absolute difference in conventional Q_{SLS} and CAPWAP-based

Q_{SLS} mobilized at 10 mm top-of-pile settlement yielded a COV of 0.92, indicating high variability in the conventional Q_{SLS}. Review of the Q_{SLS}, as dictated by 10 mm of top-of-pile settlement, showed results similar to the Q_{SLS} dictated by the conventional design criterion. The ratio of Q_{ult} to Q_{SLS} in this case is equal to 1.9. The ratio between the Q_{ult} and the conventionally determined Q_{SLS} as determined from the twenty-nine load-displacement curves ranged between 1.1 and 2.5 with a mean value equal to 1.8 and a COV of 0.19.

6.3 Development of the LSD Recommendations

The analysis results of top-of-pile settlement were used, in part, to form the thesis recommendations (presented in Chapter 7) for the SLS and ULS design cases. The SLS recommendations are based on the percentage of the Q_{ult} mobilized at tolerable settlements. The ULS recommendations include new recommended RFs and refined RAMs for H-pile design against failure. The processes undertaken to develop the recommended LSD criteria are explained here.

6.3.1 Development of the Proposed SLS Design Recommendations

The author knows of no reported incidences of driven steel H-piles failing to perform under service conditions. Therefore, there is no reason to believe that the conventional design criteria are liberal. For the piles that were studied, the conventional Q_{SLS} was less than the measured Q_{ult} for 100 percent of the piles. However, significant variability can be seen when comparing these capacities. Understanding the relationships between the ultimate capacities (*i.e.* Q_{ult}, Q_{s-ult}, and Q_{b-ult}), the load distribution, the mobilized capacity, and corresponding settlement, the elastic compression, and the toe quake reduces the uncertainty of the Q_{SLS} of driven steel H-piles in Winnipeg.

The serviceability analysis performed in the thesis is more representative of H-pile behavior under service loads than the conventional design criteria. Estimating the performance of an H-pile under service loads using the results discussed in Chapter 6, allows for fine-tuning of top-of-pile settlements and corresponding capacities to specifically suit different project constraints. For example if 10 mm of top-of-pile settlement is too much, an engineer can assess the amount of capacity that can be mobilized at a lesser amount of top-of-pile settlement. The options to increase or reduce the number of piles to satisfy project constraints are easily performed if the magnitudes of mobilized capacities and the corresponding top-of-pile settlements are understood.

A majority of the top-of-pile settlement that will occur in an H-pile under service loading is a result of the elastic compression of the pile which mobilizes the shaft friction resistance and eventually transfers load to the pile toe. At working stresses, relatively little load is transferred to the pile toe. Therefore, a very small amount of the total settlement, if any, occurs as a result of soil compression at the pile toe.

6.3.2 Development of the Proposed ULS Design Recommendations

The performance of H-piles under the service loads plays a major role in determining appropriate RFs for ULS design. Theoretically, since no piles exhibited Q_{ult} less than the Q_{SLS} at a top-of-pile settlement of 10 mm, the probability of this occurring is essentially zero. Therefore, it can be established that the Q_{ULS} is greater than the Q_{SLS}. To assess the RF that should be used for each dynamic RAM, the Q_{SLS} was first assessed in terms of the worst-case percentage of CAPWAP-calculated capacities mobilized at 10 mm of top-of-pile settlement. Based on this result, the highest value of Q_{ult} predicted by each RAM is iteratively matched to the minimum Q_{SLS} by adjusting the RF.

This approach is logical and conservative for three reasons: 1) the SLS condition of 10 mm of top pile settlement is 4.6 mm less than the lowest top-of-pile settlement measured at failure (Q_{ult}) out of all of the sixty-four tested piles, 2) the highest over-predicted Q_{ult} value (compared to CAPWAP) of each RAM is used to represent the Q_{ULS}, and 3) the Q_{ULS} is matched to the Q_{SLS}, not a higher capacity, which could be justified considering the first reason.

Table 6.3 presents recommended RFs for the RAMs refined in this thesis, for the purpose of estimating the Q_{ult} and Q_{ULS} of driven steel H-piles in Winnipeg. It is an improvement to the NBCC (2010) because it is based on actual measurements of Q_{ult}. The recommendations for RFs are less conservative than the NBCC (2010), which would result in more cost-effective foundation systems for large projects. The ULS recommendations are also more complete than the NBCC (2010) because they include three additional RAMs and discern between CAPWAP and high-strain dynamic testing (PDA testing). It must be understood; that these ULS recommendations are entirely based on the author's opinion and should not be taken as a replacement to the current NBCC (2010).

6.4 Chapter 6 Discussion

Settlements of structures under service loads are often misunderstood. A common impression is that a structure, especially a large structure, should not move under service loads. However, it must be understood that compressive settlement must occur for the pile to develop resistance.

The simulated CAPWAP load-displacement curves provide the opportunity to assess the settlement of driven steel H-piles in Winnipeg under axial compressive load. The load transferred to the pile toe dictates the amount of settlement that occurs at the pile toe. The top-of-pile settlement is based on the total applied load which includes the load that is shed along the pile shaft and the load transferred to the pile toe. Top-of-pile settlement includes elastic compression of the pile.

Understanding the components of load-displacement behavior will allow engineers to reach a more informed conclusion as to how H-piles will perform during the initial loading stage. Understanding how soil settlements are mobilized at the H-pile toe, the amount of resistance that is shed through the pile shaft, and the amount of elastic compression that can be expected will produce more dependable estimates of performance, as opposed to applying a global factor of safety to the Q_{ult}. Furthermore, increased assurance, or piece of mind, in the amount of settlement that will occur under service loads will aid engineers in selecting appropriate values of Q_{SLS} to use in LSD.

Based on the findings of the load-displacement behavior at the pile toe, it is determined that the average amount of settlement exhibited by the soil at $Q_{b\text{-}ult}$ is in the order of 4 to 5 mm. About 25 and 50 percent of the $Q_{b\text{-}ult}$, on average, is mobilized at 1 and 2 mm (or less) of toe settlement, respectively.

A majority of the observed top-of-pile settlements are a result of elastic compression. Elastic compression constitutes between about 75 and 80 percent of the top-of-pile settlement of an H-pile, which is in the order of 0.1 percent of the total pile length. As should be expected and described by Vesic (1977), longer H-piles result in greater amounts of elastic compression.

The statistics presented in this chapter are valuable when considering the performance of driven steel H-piles in Winnipeg. The conventional Q_{SLS} is on average about 59 percent of the Q_{ult} which correlates to about 10 mm of top-of-pile settlement. However, the Q_{SLS} and the Q_{ULS} can be determined to satisfy the requirements of LSD first by identifying the Q_{ult} and $Q_{b\text{-}ult}$, using the RAMs presented in Chapters 4 and 5. The Q_{ult} and $Q_{b\text{-}ult}$ must be determined to calculate the Q_{SLS} on the basis of the material covered in this thesis. The author concludes that Q_{SLS} may not govern design, depending on the RF applied to the RAMs in ULS design case.

Table 6.1: Summary of the CAPWAP-calculated toe settlements of the sixty-four driven steel H-piles.

Pile Size	CAPWAP Capacity			Load Displacement at Pile Toe													End Bearing Strata
	Total Ultimate Capacity	Ultimate End Bearing Capacity	Ultimate Shaft Capacity	Ultimate End Bearing Capacity Mobilized at 1 mm of Toe Displacement		Ultimate End Bearing Capacity Mobilized at 2 mm of Toe Displacement		Ultimate End Bearing Capacity Mobilized at 3 mm of Toe Displacement		Ultimate End Bearing Capacity Mobilized at 4 mm of Toe Displacement		Ultimate End Bearing Capacity Mobilized at 5 mm of Toe Displacement		Ultimate End Bearing Capacity Mobilized at 10 mm of Toe Displacement		Toe Quake	
	kN	kN	kN	kN	%	kN	%	kN	%	kN	%	kN	%	kN	%	mm	
HP200x54	1614	836	778	559	67%											1.5	BEDROCK
HP310x110	2853	1900	953	582	31%	1164	61%	1746	92%							3.3	BEDROCK
HP310x110	2976	2202	774	608	28%	1215	55%	1823	83%							3.6	BEDROCK
HP310x110	2332	1661	671	420	25%	841	51%	1261	76%							4.0	BEDROCK
HP310x110	2405	2155	250	659	31%	1318	61%	1977	92%							3.3	BEDROCK
HP310x110	2543	2254	289	674	30%	1349	60%	2023	90%							3.3	BEDROCK
HP310x110	2907	2587	321	1054	41%	2108	81%									2.5	BEDROCK
HP310x110	2243	1685	558	329	20%	658	39%	987	59%	1315	78%	1644	98%			5.1	BEDROCK
HP310x110	3104	1195	1909	308	26%	616	52%	924	77%							3.9	BEDROCK
HP310x110	2419	1765	653	472	27%	944	53%	1415	80%							3.7	BEDROCK
HP310x110	2414	1332	1082	277	21%	553	42%	830	62%	1106	83%					4.8	BEDROCK
HP310x110	2101	1360	742	244	18%	489	36%	733	54%	977	72%	1221	90%			5.6	BEDROCK
HP310x110	2514	1400	1113	439	31%	877	63%	1316	94%							3.2	SILT TILL
HP310x110	2267	1137	1130	337	30%	674	59%	1011	89%							3.4	SILT TILL
HP310x110	2254	1395	859	330	24%	661	47%	991	71%	1322	95%					4.2	SILT TILL
HP310x110	1860	1023	837	298	29%	597	58%	895	88%							3.4	SILT TILL
HP310x110	2217	1476	741	306	21%	613	42%	919	62%	1225	83%					4.8	SILT TILL
HP310x110	2094	1355	739	198	15%	396	29%	594	44%	792	58%	990	73%			6.8	SILT TILL
HP310x110	2095	1390	704	557	40%	1115	80%									2.5	SILT TILL
HP310x110	2384	1835	549	717	39%	1433	78%									2.6	BEDROCK
HP310x110	3421	2570	851	489	19%	979	38%	1468	57%	1958	76%	2447	95%			5.3	BEDROCK
HP310x110	3350	2503	847	544	22%	1088	43%	1632	65%	2176	87%					4.6	BEDROCK
HP310x110	2803	2135	668	443	21%	887	42%	1330	62%	1774	83%					4.8	BEDROCK
HP310x110	3232	2480	752	556	22%	1112	45%	1669	67%	2225	90%					4.5	BEDROCK
HP310x125	2464	2039	425	448	22%	895	44%	1343	66%	1791	88%					4.6	BEDROCK
HP310x125	2342	2005	337	402	20%	804	40%	1205	60%	1607	80%	2009	100%			5.0	BEDROCK
HP310x125	2520	2025	494	529	26%	1059	52%	1588	78%							3.8	BEDROCK
HP310x125	2410	2055	355	702	34%	1404	68%									2.9	BEDROCK
HP310x125	2130	1586	544	328	21%	656	41%	984	62%	1312	83%					4.8	SAND TILL
HP310x125	2166	1671	495	610	37%	1220	73%									2.7	SAND TILL
HP310x125	2783	2363	419	524	22%	1048	44%	1572	67%	2096	89%					4.5	BEDROCK
HP310x125	2525	2184	341	740	34%	1479	68%									3.0	BEDROCK
HP310x125	2724	1840	883	328	18%	655	36%	983	53%	1311	71%	1639	89%			5.6	BEDROCK
HP310x125	2668	2145	523	453	21%	907	42%	1360	63%	1813	85%					4.7	BEDROCK
HP310x125	2363	1737	626	363	21%	726	42%	1089	63%	1452	84%					4.8	SAND TILL
HP310x125	2457	1942	515	395	20%	790	41%	1184	61%	1579	81%					4.9	SAND TILL
HP310x125	3157	2850	307	562	20%	1123	39%	1685	59%	2247	79%	2808	99%			5.1	BEDROCK
HP310x125	2450	2239	211	665	30%	1329	59%	1994	89%							3.4	BEDROCK
HP310x125	2267	1637	629	474	29%	948	58%	1423	87%							3.5	BEDROCK
HP310x125	1737	1037	700	239	23%	478	46%	718	69%	957	92%					4.3	SAND TILL
HP310x125	1850	1189	662	350	29%	701	59%	1051	88%							3.4	SAND TILL
HP310x125	2171	1631	540	372	23%	744	46%	1117	68%	1489	91%					4.4	SAND TILL
HP310x125	3166	2625	541	738	28%	1476	56%	2215	84%							3.6	BEDROCK
HP310x125	3057	2650	407	1151	43%	2302	87%									2.3	BEDROCK
HP310x125	2825	2228	597	744	33%	1488	67%									3.0	BEDROCK
HP310x125	2746	2168	578	496	23%	992	46%	1488	69%	1984	92%					4.4	BEDROCK
HP310x125	2750	2190	560	554	25%	1109	51%	1663	76%							4.0	BEDROCK

HP310x132	3501	2958	543	1148	39%	2295	78%									2.6	BEDROCK
HP310x132	3300	2593	707	801	31%	1602	62%	2404	93%							3.2	BEDROCK
HP310x132	3302	2932	370	719	25%	1438	49%	2157	74%	2876	98%					4.1	BEDROCK
HP310x132	3001	1901	1100	380	20%	760	40%	1140	60%	1520	80%	1901	100%			5.0	BEDROCK
HP310x132	2901	2541	360	508	20%	1016	40%	1524	60%	2033	80%	2541	100%			5.0	BEDROCK
HP310x132	3201	2801	400	1120	40%	2241	80%									2.5	BEDROCK
HP310x94	2871	2278	593	534	23%	1068	47%	1601	70%	2135	94%					4.3	BEDROCK
HP360x132	2862	1663	1198	611	37%	1222	73%									2.7	BEDROCK
HP360x132	2302	1567	735	158	10%	315	20%	473	30%	630	40%	788	50%			9.9	BEDROCK
HP360x132	2381	1449	932	112	8%	223	15%	335	23%	446	31%	558	39%	1116	77%	13.0	BEDROCK
HP360x132	4325	2888	1437	1371	47%	2743	95%									2.1	BEDROCK
HP360x132	4037	3684	353	3636	99%											1.0	BEDROCK
HP360x132	2537	2200	337	361	16%	721	33%	1082	49%	1442	66%	1803	82%			6.1	BEDROCK
HP360x132	2348	1776	573	298	17%	597	34%	895	50%	1194	67%	1492	84%			6.0	BEDROCK
HP360x132	1964	1587	377	200	13%	400	25%	600	38%	800	50%	1000	63%			7.9	BEDROCK
HP360x132	3482	3192	290	391	12%	783	25%	1174	37%	1566	49%	1957	61%			8.2	BEDROCK
HP360x132	3233	3047	186	373	12%	746	24%	1119	37%	1492	49%	1865	61%			8.2	BEDROCK

Table 6.2: Summary of the CAPWAP-calculated top-of-pile settlements of twenty-nine driven steel H-piles.

Pile Size	CAPWAP Capacity			Conventional Design Capacity		Load Displacement at Top of Pile									End Bearing Strata
								Elastic Shortening			5 mm Top of Pile Displacement		10 mm Top of Pile Displacement		
	Total Ultimate Capacity	Ultimate End Bearing Capacity	Ultimate Shaft Capacity	Total Ultimate Capacity	Fraction of Total Ultimate Capacity	Pile Length	Displacement at Total Ultimate Capacity	CAPWAP	Fraction of Pile Length	Fraction of Top of Pile Displacement	Total Capacity Mobilized	Fraction of Total Ultimate Capacity	Total Capacity Mobilized	Fraction of Total Ultimate Capacity	
	kN	kN	kN	kN	%	m	mm	mm	%	%	kN	%	kN	%	
HP200x54	1614	836	778	718	44%	21.0	24.9	23.4	0.11%	94.0%	365	23%	710	44%	BEDROCK
HP310x94	2871	2278	593	1250	44%	15.0	22.5	18.2	0.12%	81.0%	760	26%	1399	49%	BEDROCK
HP310x110	2101	1360	742	1481	70%	17.7	19.3	13.7	0.08%	71.2%	659	31%	1260	60%	BEDROCK
HP310x110	2243	1685	558	1481	66%	17.0	19.4	14.3	0.08%	73.6%	684	30%	1348	60%	BEDROCK
HP310x110	2332	1661	671	1481	63%	16.8	17.9	13.9	0.08%	77.9%	794	34%	1508	65%	BEDROCK
HP310x110	2405	2155	250	1481	62%	17.4	18.4	15.1		82.2%	701	29%	1402	58%	BEDROCK
HP310x110	2414	1332	1082	1481	61%	17.0	19.9	15.1	0.09%	75.8%	698	29%	1357	56%	BEDROCK
HP310x110	2419	1765	653	1481	61%	17.7	19.1	15.4	0.09%	80.4%	775	32%	1376	57%	BEDROCK
HP310x110	2543	2254	289	1481	58%	17.7	19.5	16.2	0.09%	82.9%	730	29%	1398	55%	BEDROCK
HP310x110	2853	1900	953	1481	52%	17.8	20.4	17.1	0.10%	84.0%	578	20%	1581	55%	BEDROCK
HP310x110	2907	2587	321	1481	51%	17.6	20.5	18.0	0.10%	88.0%	790	27%	1490	51%	BEDROCK
HP310x110	2976	2202	774	1481	50%	17.8	22.3	18.7	0.10%	83.7%	791	27%	1490	50%	BEDROCK
HP310x110	3104	1195	1909	1481	48%	17.0	24.6	20.7	0.12%	84.2%	760	24%	1465	47%	BEDROCK
HP310x132	2901	2541	360	1754	60%	20.2	21.9	16.9	0.08%	77.2%	867	30%	1500	52%	BEDROCK
HP310x132	3001	1901	1100	1754	58%	19.0	20.6	15.6	0.08%	75.7%	905	30%	1658	55%	BEDROCK
HP310x132	3201	2801	400	1754	55%	20.2	20.4	17.9	0.09%	87.7%	656	21%	1695	53%	BEDROCK
HP310x132	3300	2593	707	1754	53%	13.8	16.6	13.4	0.10%	80.5%	1171	35%	2200	67%	BEDROCK
HP310x132	3302	2932	370	1754	53%	15.3	19.6	15.5	0.10%	79.2%	993	30%	1809	55%	BEDROCK
HP310x132	3501	2958	543	1754	50%	14.0	16.8	14.2	0.10%	84.7%	1257	36%	2270	65%	BEDROCK
HP360x132	1964	1587	377	1764	90%	10.5	16.0	8.1	0.08%	50.4%	851	43%	1500	76%	BEDROCK
HP360x132	2302	1567	735	1764	77%	22.7	26.6	16.7	0.07%	62.6%	836	36%	1242	54%	BEDROCK
HP360x132	2348	1776	573	1764	75%	10.5	14.6	8.6	0.08%	59.2%	1014	43%	1900	81%	BEDROCK
HP360x132	2381	1449	932	1764	74%	21.5	30.7	17.7	0.08%	57.7%	836	35%	1267	53%	BEDROCK
HP360x132	2537	2200	337	1764	70%	17.4	20.6	14.5	0.08%	70.4%	730	29%	1399	55%	BEDROCK
HP360x132	2862	1663	1198	1764	62%	19.8	20.2	17.5	0.09%	86.5%	851	30%	1642	57%	BEDROCK
HP360x132	3233	3047	186	1764	55%	11.0	20.8	12.6	0.11%	60.7%	922	29%	1774	55%	BEDROCK
HP360x132	3482	3192	290	1764	51%	11.0	21.5	13.3	0.12%	62.1%	958	28%	1880	54%	BEDROCK
HP360x132	4037	3684	353	1764	44%	25.6	33.7	32.7	0.13%	97.0%	730	18%	1368	34%	BEDROCK
HP360x132	4325	2888	1437	1764	41%	25.6	31.1	29.0	0.11%	93.2%	684	16%	1368	32%	BEDROCK

Table 6.3: Recommended resistance factors from the NBCC (2010) and Belbas (2013) for driven steel H-piles in Winnipeg.

Resistance Analysis Method	**Φ (NBCC 2010)**	**Resistance Analysis Method**	**Φ (Belbas 2013)**
Dynamic Load Testing	0.50	CAPWAP	0.75
Case Method	-	Case Method	0.55
GRLWEAP	-	GRLWEAP	0.50
Modified Gates Formula	-	Modified Gates	0.40
In-situ Testing and Lab Data	0.40	*In-situ* and Lab Testing Data	0.40

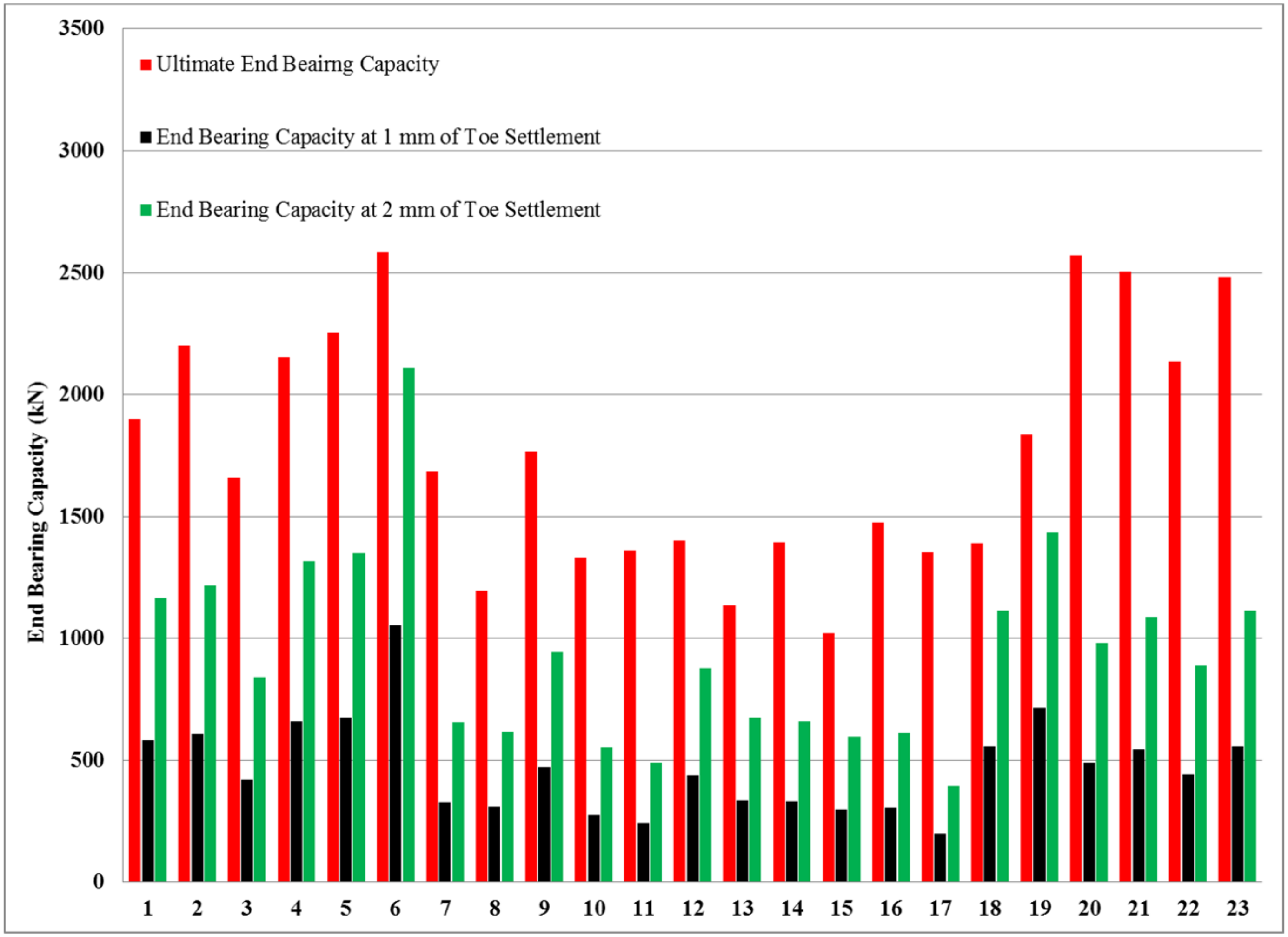


Figure 6.1: The $Q_{b\text{-}ult}$ and mobilized end bearing capacities at 1 and 2 mm of toe settlement of HP310x110 sections.

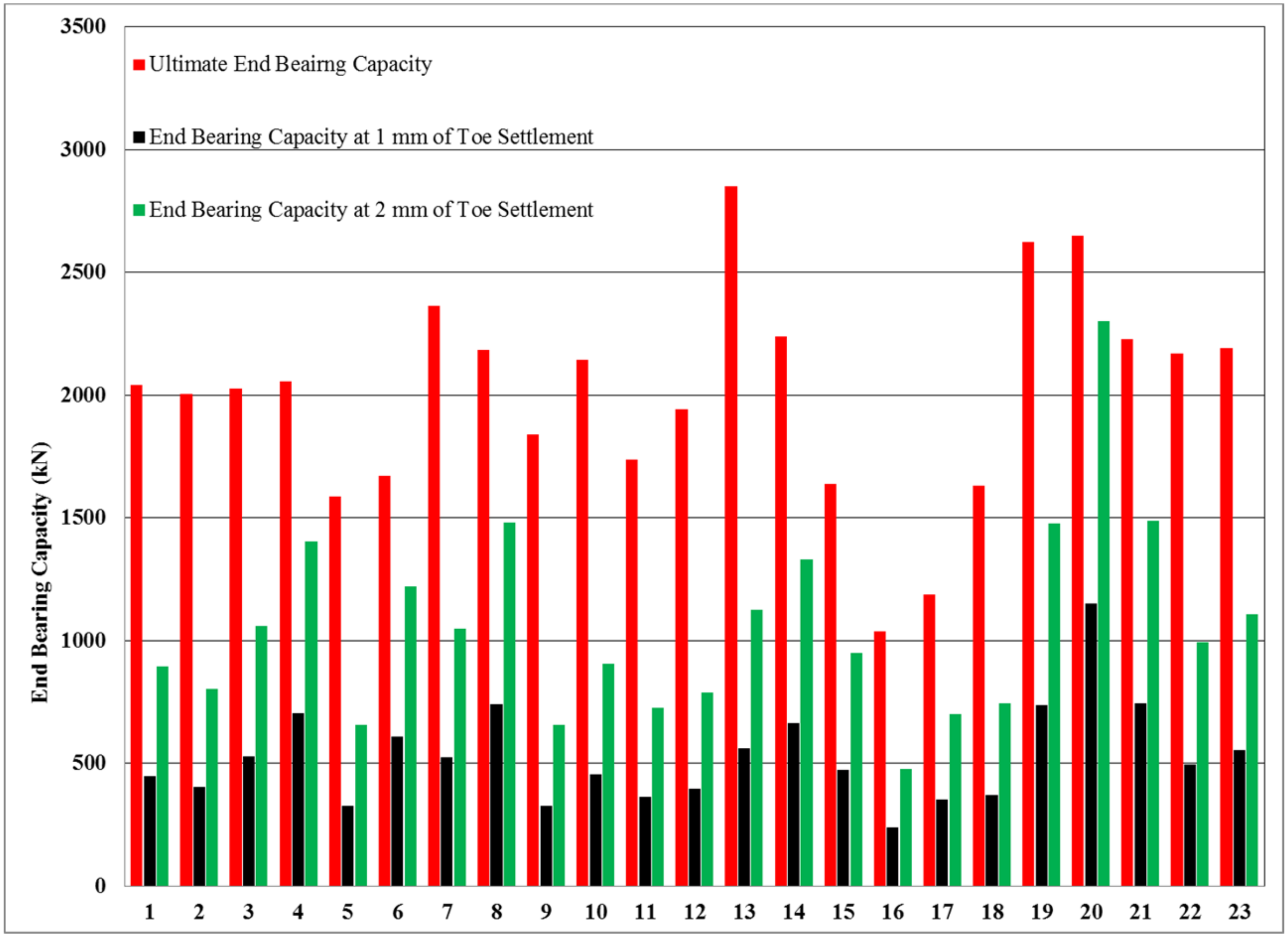


Figure 6.2: The $Q_{b\text{-}ult}$ and mobilized end bearing capacities at 1 and 2 mm of toe settlement for HP310x125 H-pile sections.

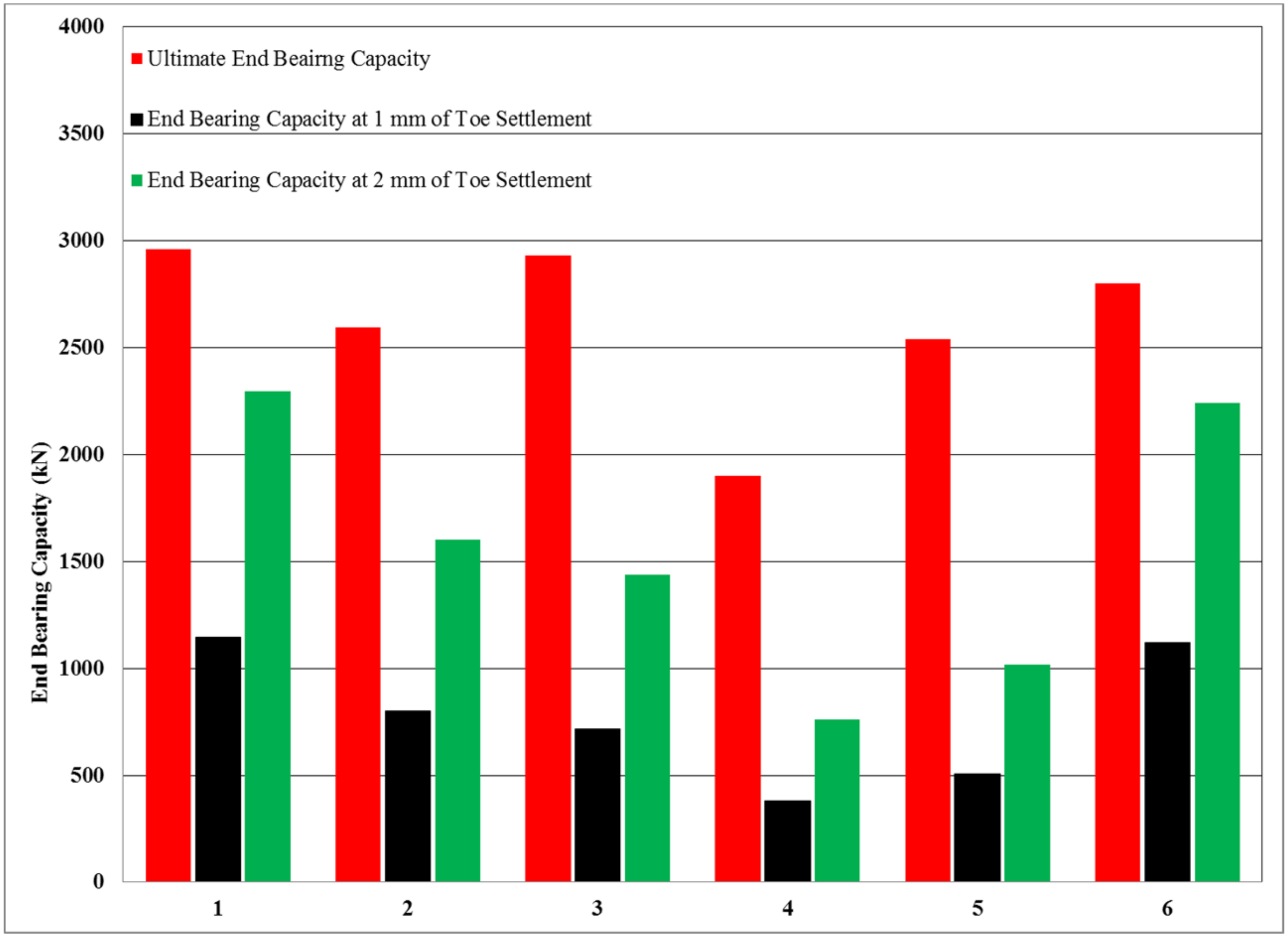


Figure 6.3: The $Q_{b\text{-}ult}$ and mobilized end bearing capacities at 1 and 2 mm of toe settlement for HP310x132 sections.

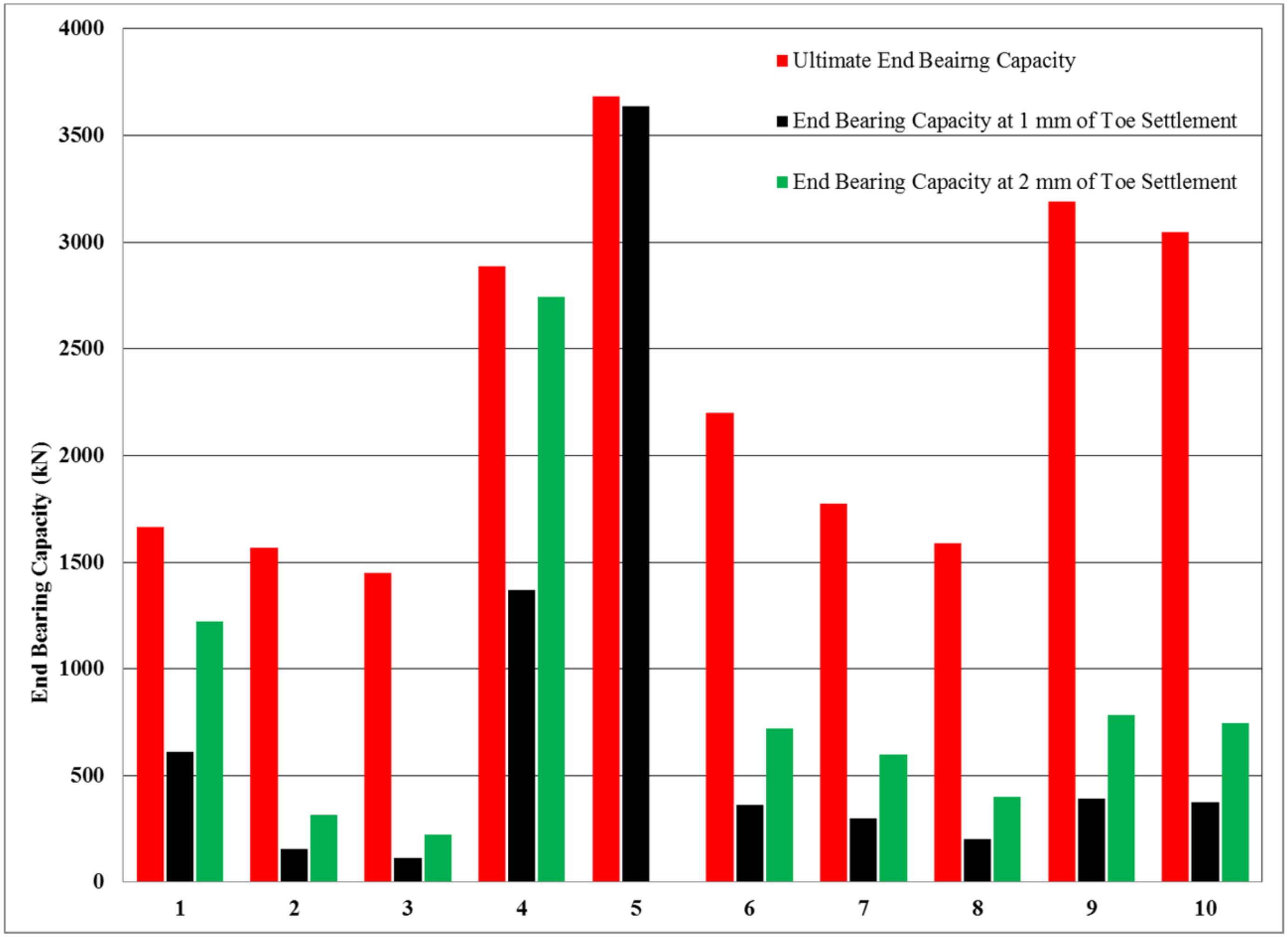


Figure 6.4: The $Q_{b\text{-}ult}$ and mobilized end bearing capacities at 1 and 2 mm of toe settlement for HP360x132 sections.

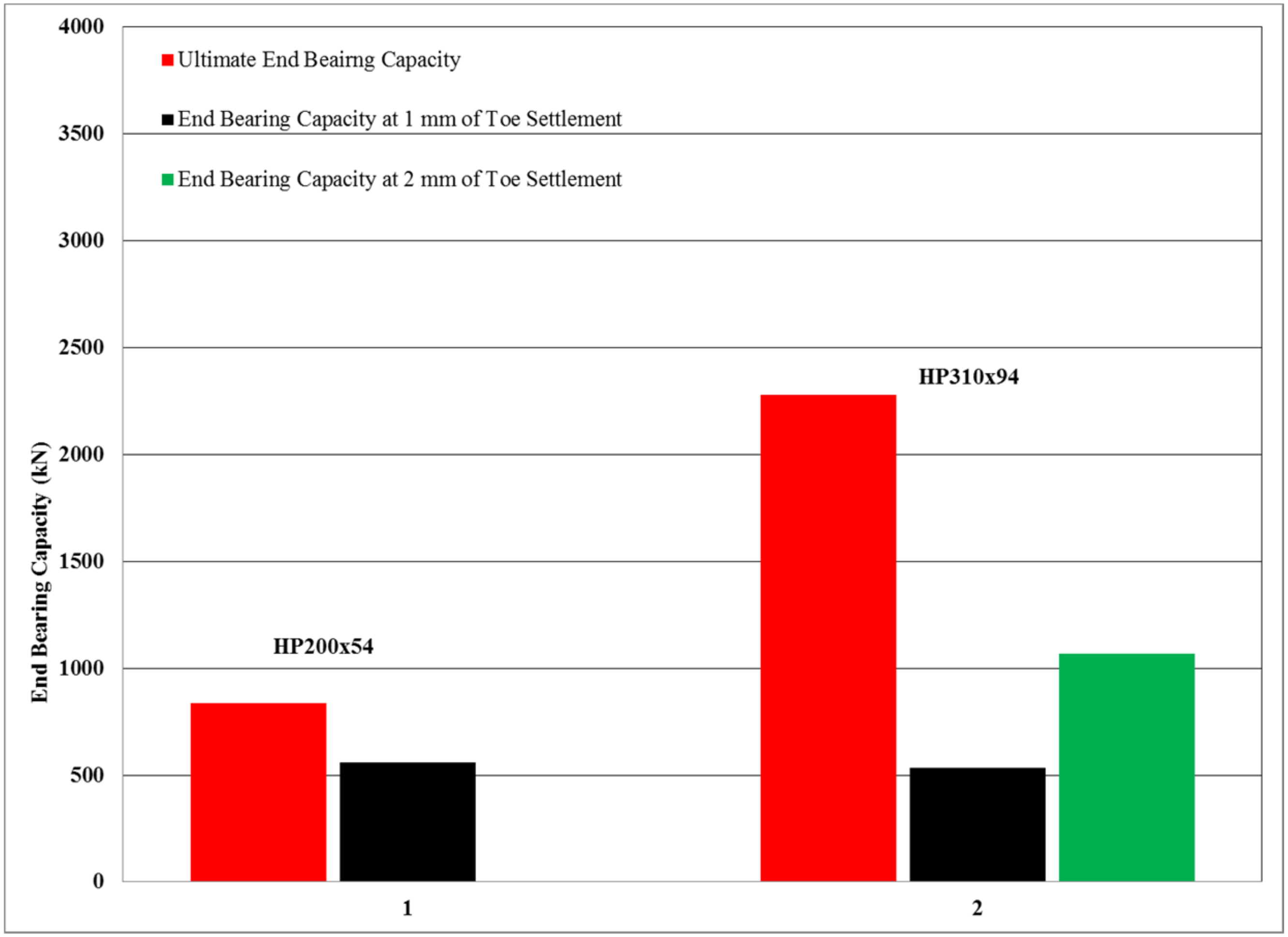


Figure 6.5: The $Q_{b\text{-}ult}$ and mobilized end bearing capacities at 1 and 2 mm of toe settlement for HP200x54 and HP310x94 sections.

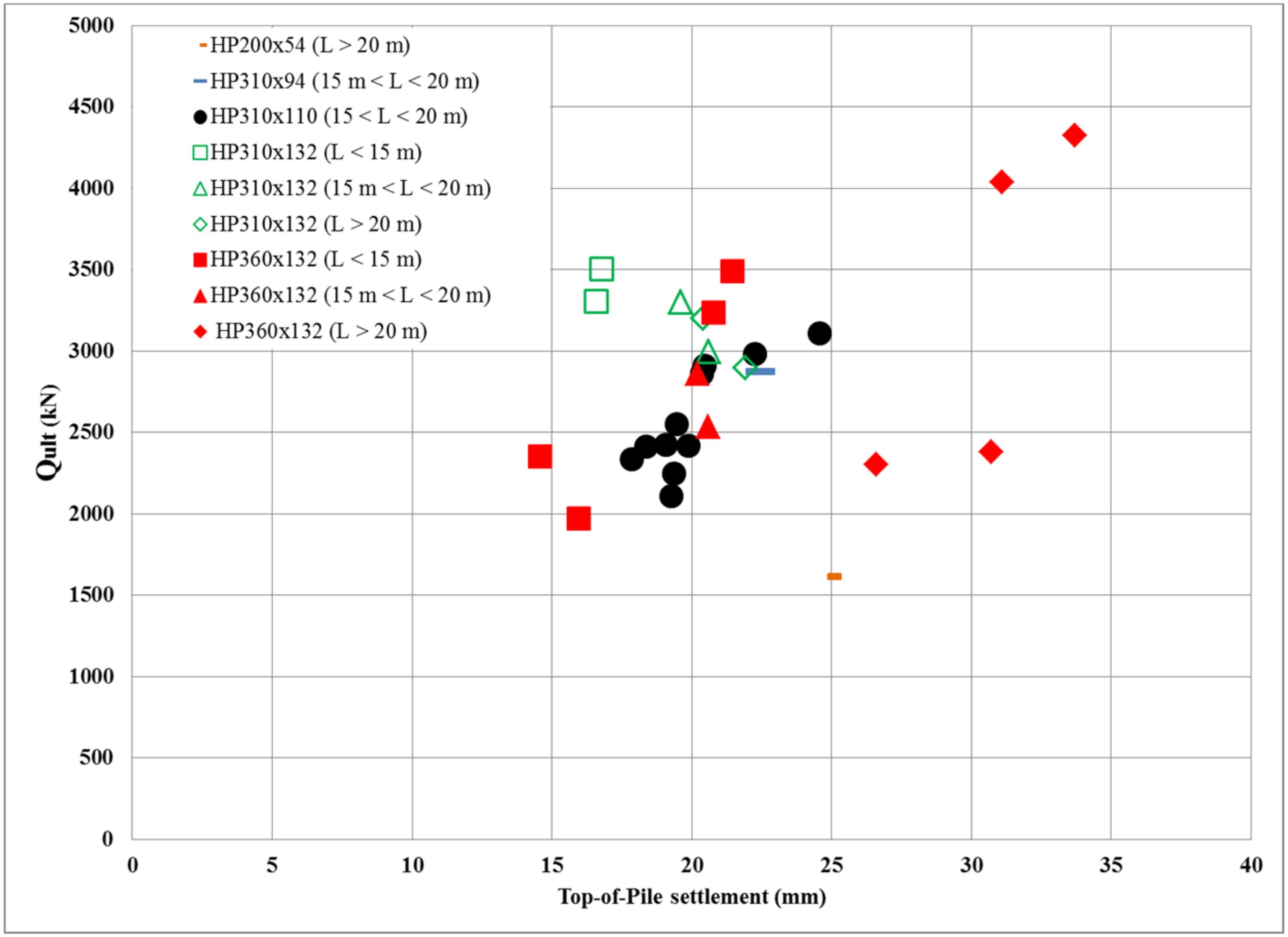


Figure 6.6: Relationship between Q_{ult}, pile size, pile length, and top-of-pile settlement.

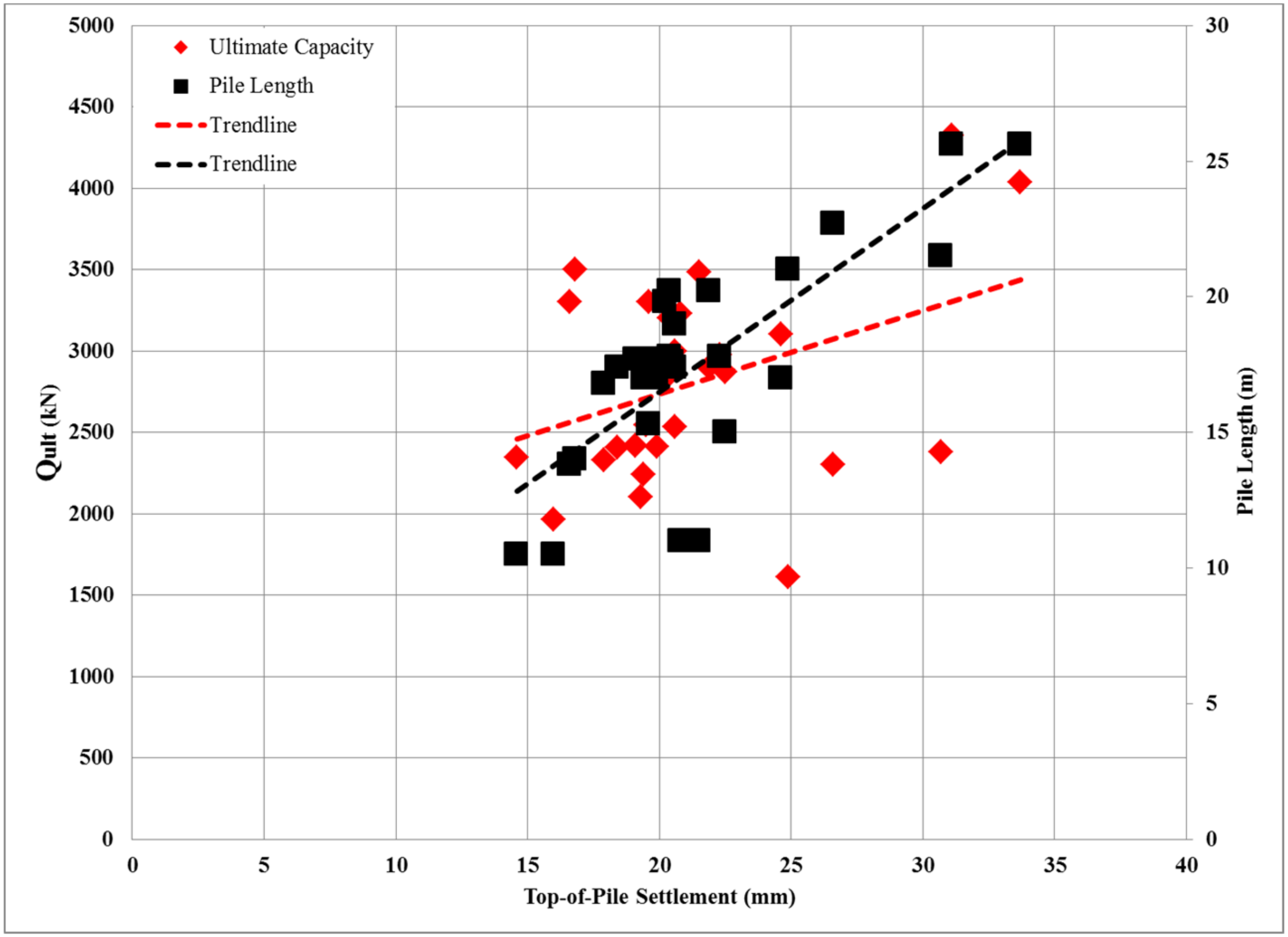


Figure 6.7: Relationship between Q_{ult}, pile length, and top-of-pile settlement at failure.

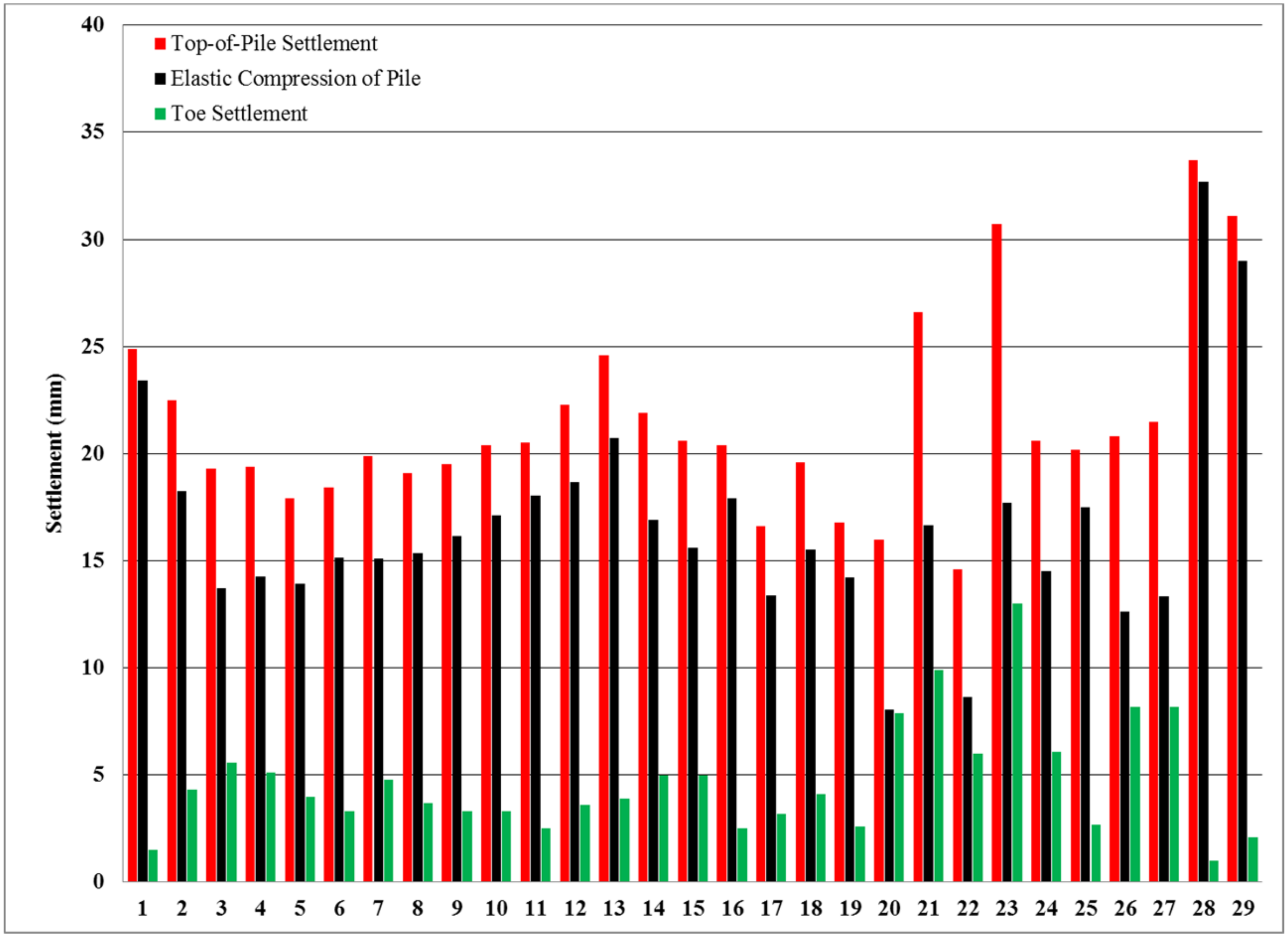


Figure 6.8: Top-of-pile settlement, elastic compression of pile, and toe settlement at failure.

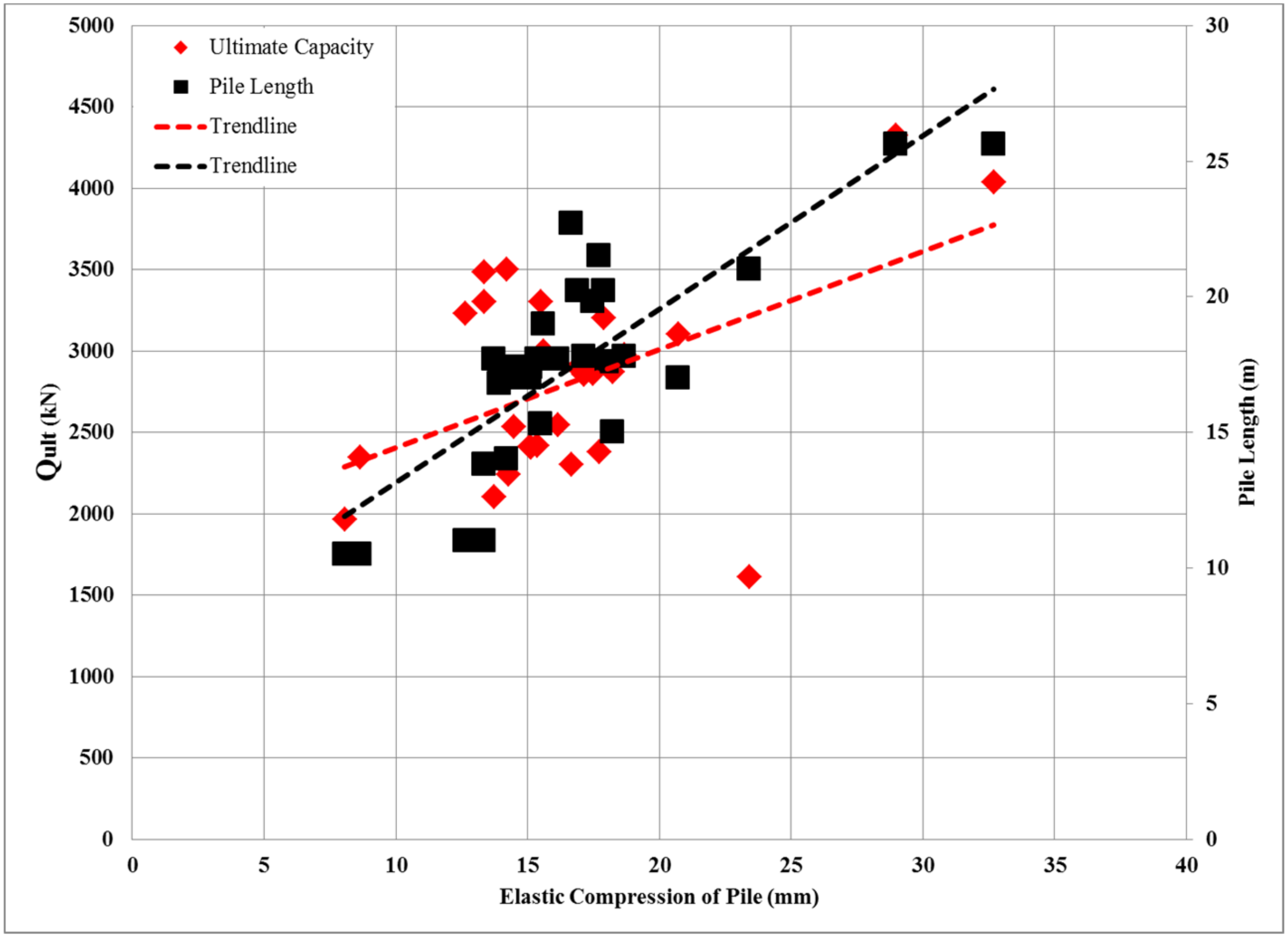


Figure 6.9: Relationship between Q_{ult}, pile length, and elastic compression of the pile at failure.

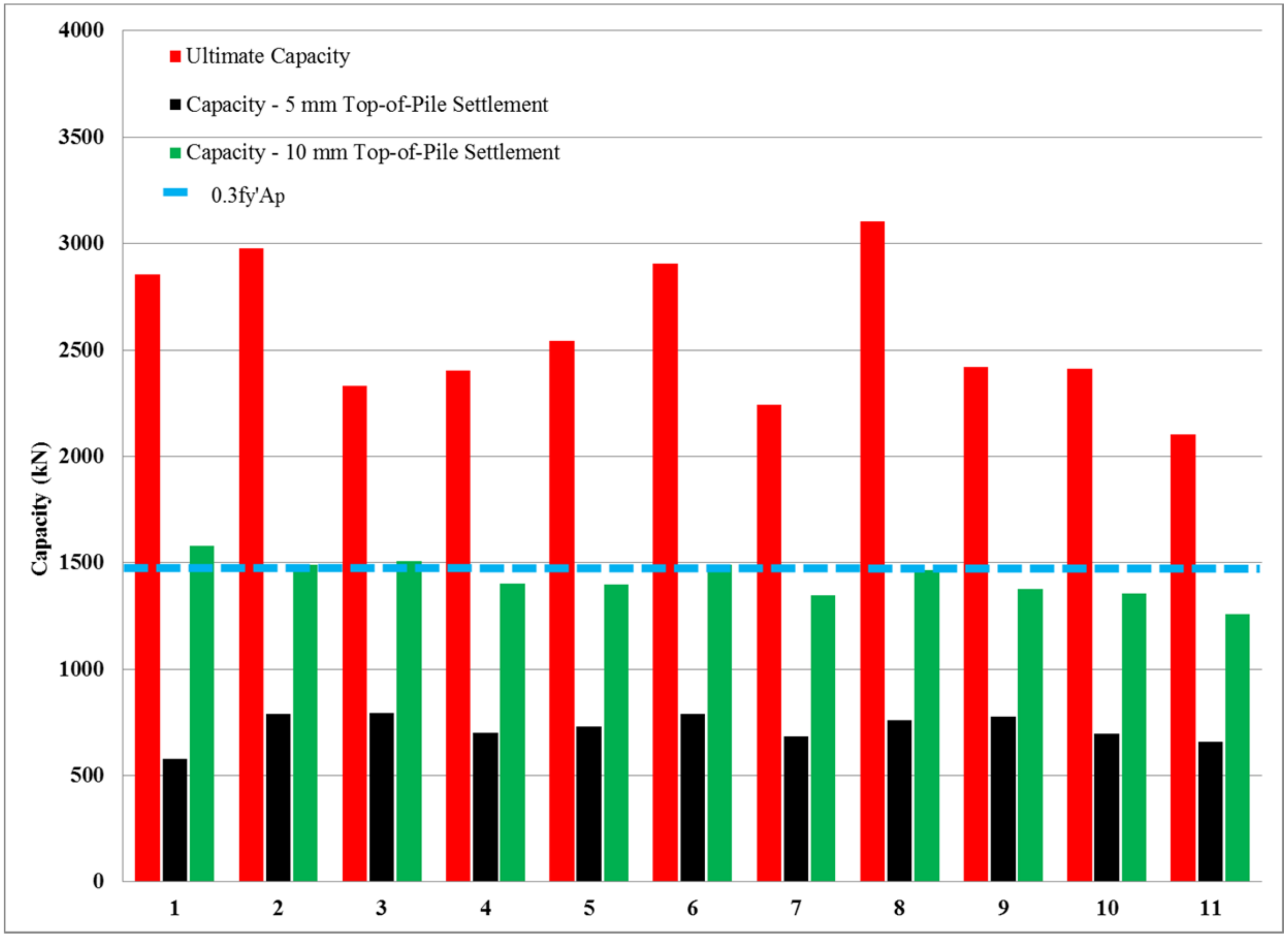


Figure 6.10: The Q_{ult} and mobilized capacities at 5 and 10 mm of top-of-pile settlement compared to the conventional SLS design criterion for HP310x110 sections.

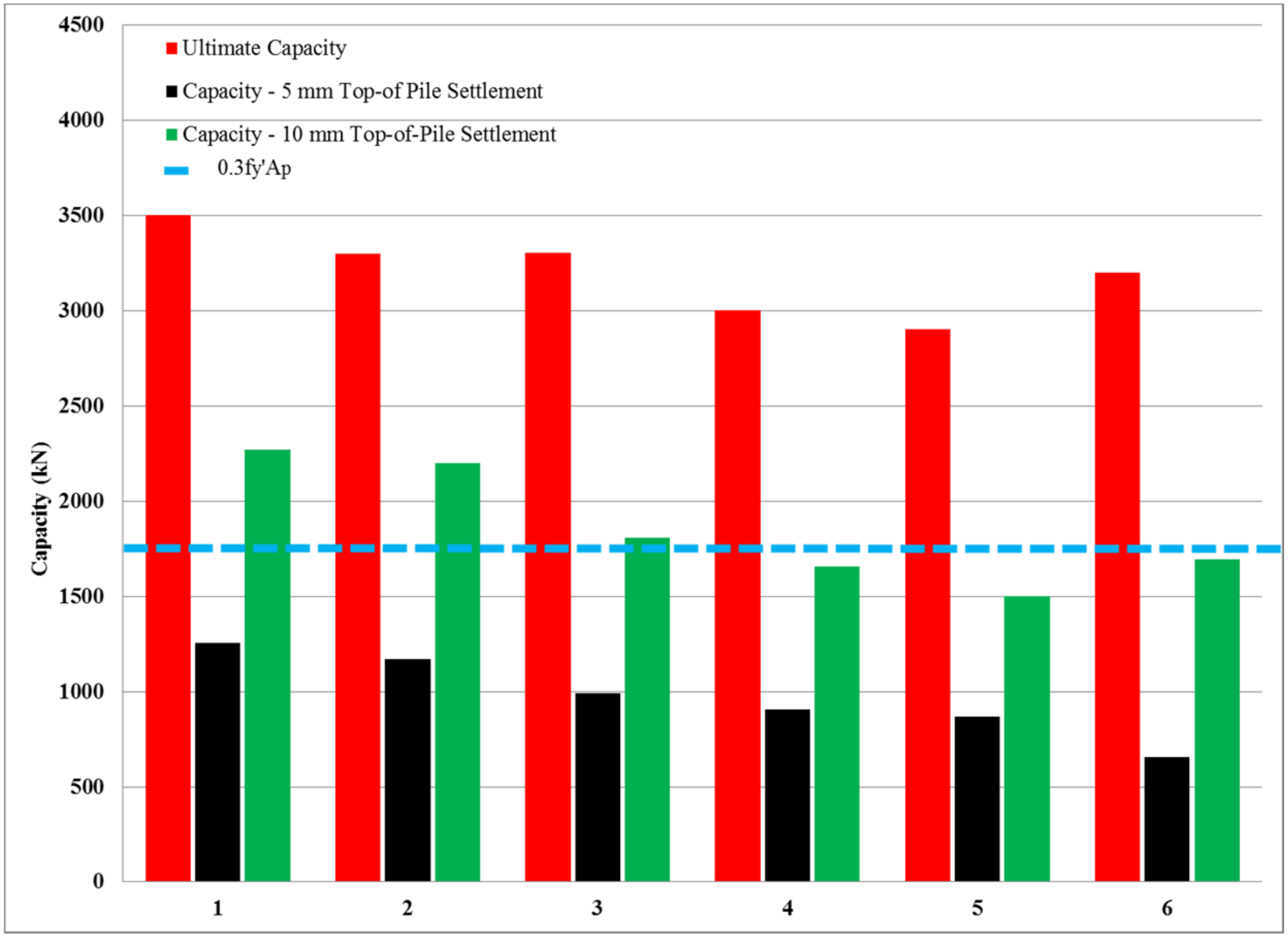


Figure 6.11: The Q_{ult} and mobilized capacities at 5 and 10 mm of top-of-pile settlement compared to the conventional SLS design criterion for HP310x132 sections.

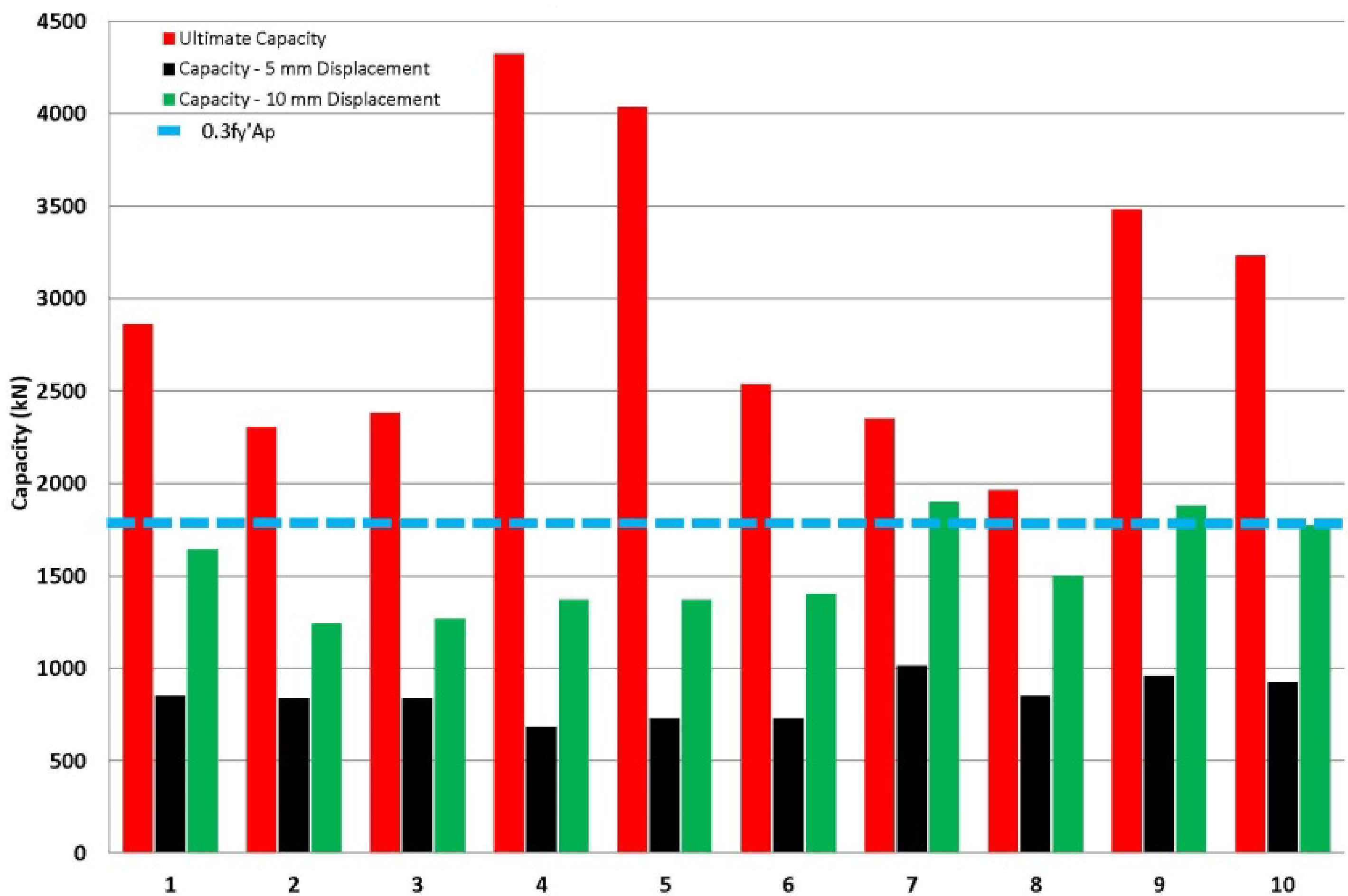


Figure 6.12: The Q_{ult} and mobilized capacities at 5 and 10 mm of top-of-pile settlement compared to the conventional SLS design criterion for HP360x132 sections.

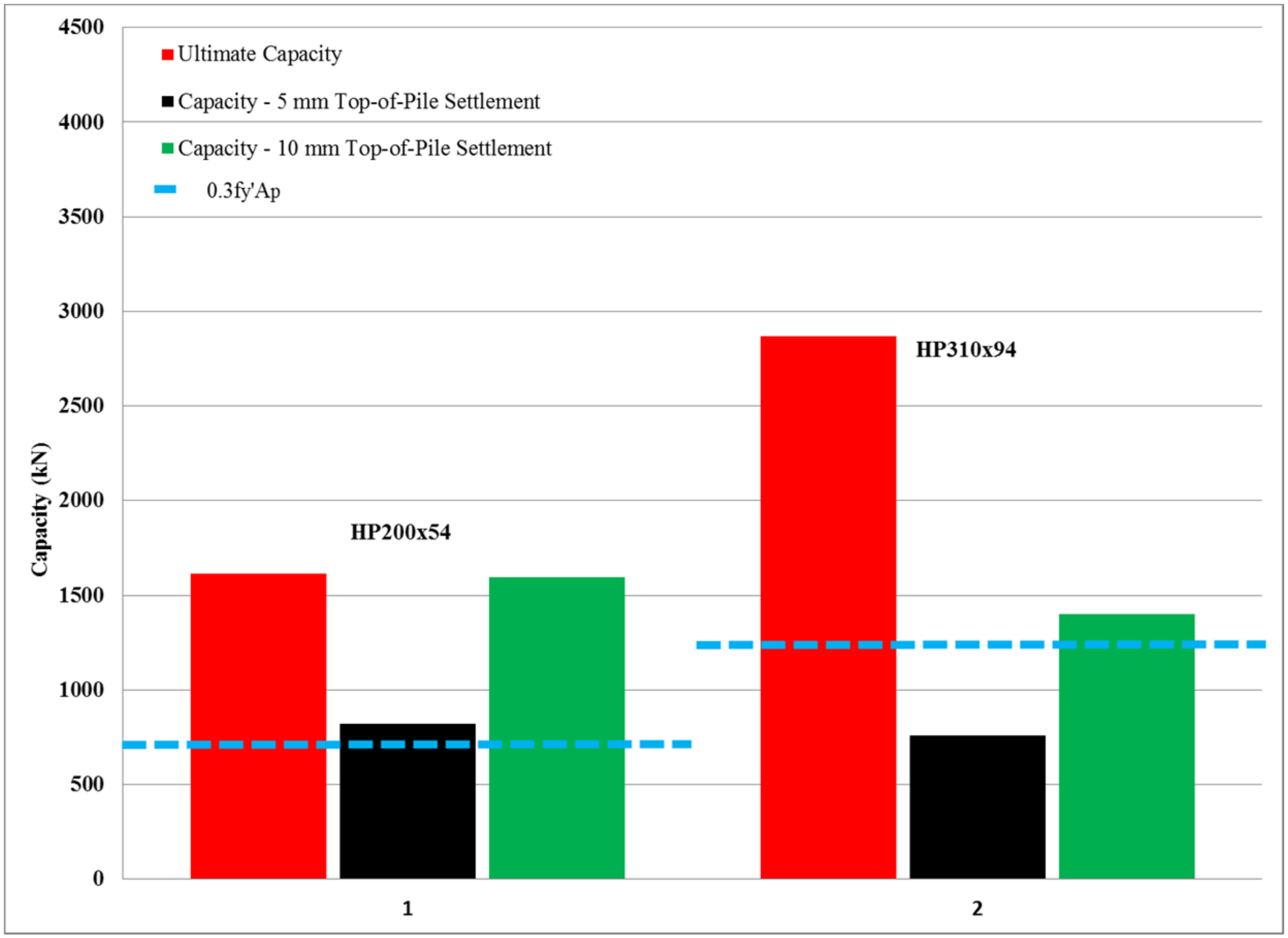


Figure 6.13: The Q_{ult} and mobilized capacities at 5 and 10 mm of top-of-pile settlement compared to the conventional SLS design criterion for HP200x54 and HP310x94 sections.

Chapter 7: Thesis Summary, Conclusions, and Recommendations

7.0 Introduction

Chapter 7 provides a summary of the analyses performed, conclusions made from the results of the analyses, and recommendations for LSD criteria of driven steel H-piles in Winnipeg. The main objective of this thesis was to develop comprehensive LSD criteria to be used specifically for the design of driven steel H-piles in Winnipeg.

The thesis analyses prove that the conventional design criteria used in Winnipeg (presented in Chapter 1) are fundamentally incorrect and incomplete; the Q_{ult} of driven steel H-piles should not be based only on the structural strength of steel, but should include elements of the strength of the bedrock and overburden soil, and the energy delivered by the pile driving system. The Q_{SLS} should be based on the load-displacement behavior of the H-pile and the corresponding amount of settlement that is determined to be tolerable for the supported structure. Understanding the elements that dictate the Q_{ult} and load-displacement behavior of driven steel H-piles is paramount for establishing effective LSD criteria and determining more accurate and reliable predictions of Q_{ult} and levels of serviceability of driven steel H-piles in Winnipeg.

7.1 Summary and Conclusions

This chapter summarizes the main elements established in the static, dynamic, and serviceability analyses. These main elements are important for estimating the Q_{ult}, driving criteria, and settlements of H-piles for the purpose of ULS and SLS design. A considerable amount of information is provided in Chapters 4, 5, and 6, but only a brief summary of the most important work will be provided in this chapter.

7.1.1 Chapter 4 – Static Analyses

The following describes the summary of the work performed in Chapter 4 and the conclusions made from the results of the work.

7.1.1.1 Summary

The measured values of Q_{ult} indicate the inaccuracy and variability of the conventional design criteria. This led to refinement of common static RAMs to predict the Q_{ult} of driven steel H-piles specifically in Winnipeg soils and bedrock. The CAPWAP-measured shaft and end bearing resistances of the H-piles were correlated to soil and rock strength properties that are easily and commonly attained during geotechnical investigations. These correlations were used to refine the static RAMs.

The static RAMs refined on the basis of measured values of the input parameters include the α-method, SPT-method and moisture content method for determining the $f_{s\text{-}ult}$ of driven steel H-piles in lacustrine clay and till, respectively, and the SPT-method and the q_u-method for calculating the $q_{b\text{-}ult}$ of H-piles in till and bedrock, respectively. These static RAMs are not new geotechnical analysis methods; but they are practical as predictive applications to estimate the Q_{ult} of driven steel H-piles in Winnipeg.

7.1.1.2 Conclusions

The following conclusions were made on the basis of the static analyses:

- The variability in the measured Q_{ult} values indicates that the structural strength of steel cannot be the governing factor in the capacity of driven steel H-piles in Winnipeg. Strength properties of steel are much more consistent than the amount of variability observed in the measured results. H-piles should not be designed on the basis of a single global formula meant to encompass the Q_{ult} of all sizes and lengths in varying bearing stratum and conditions.
- The conventional design ULS criterion, which is based on the structural strength of steel, over-predicts the Q_{ult} of driven steel H-piles in Winnipeg. The Q_{ULS} may be much closer than expected to the Q_{SLS}.
- The refinement of the static RAMs used to estimate the $f_{s\text{-}ult}$ of H-piles was successful and resulted in more reliable RAMs. These RAMs should be used in conjunction with the dynamic RAMs (presented in Chapter 5) to estimate the $Q_{s\text{-}ult}$ of driven steel H-piles in Winnipeg. The refinement of the RAMs used to estimate the $q_{b\text{-}ult}$ of H-piles was inconclusive due to the limited soil and rock data available at the pile toe. Existing static RAMs available in common foundation sources (*e.g.* CFEM 2006) should be used to predict the $Q_{b\text{-}ult}$ of driven steel H-piles in Winnipeg, unless more data can be collected to refine the RAMs to a more reliable degree.
- Driven steel H-piles are partial friction piles. The amount of $Q_{s\text{-}ult}$ that an H-pile will develop is based on its size, length, and the strength of the surrounding soil.

7.1.2 Chapter 5 – Dynamic Analyses

The following describes the summary of the work performed in Chapter 5 and the conclusions made from the results of the work.

7.1.2.1 Summary

The original purpose of the dynamic analyses was to provide a method for determining a practical refusal criterion to compliment the Q_{ult} as determined by the static RAMs that were refined in Chapter 4. However, it was realized that dynamic RAMs serve as additional and perhaps more reliable predictor methods for estimating the Q_{ult}. Therefore, an extensive dynamic analysis was performed. The results were valuable. The first part of the dynamic analyses performed in Chapter 5 outlines the relatively unfamiliar dynamic variables involved in dynamic analysis of driven piles. The dynamic variables as they pertain to driven steel H-piles are described by way of general bearing graphs. The bearing graphs indicate the interaction of the distinct dynamic variables on H-pile driving behavior. The important variables include, pile capacity (Q_{ult} and $Q_{b\text{-}ult}$), penetration set, transferred energy, and pile size and length. These bearing graphs can serve as stand-alone dynamic RAMs and are useful for refining the refusal criterion for a calculated Q_{ult} (or $Q_{b\text{-}ult}$) or determining a Q_{ult} (or $Q_{b\text{-}ult}$) on the basis of a penetration set.

The second part of the dynamic analyses involved refining common dynamic RAMs to predict the Q_{ult} of driven steel H-piles specifically in Winnipeg soils and bedrock. The modified Gates formula, GRLWEAP, and the Case method were assessed using recommended input variables and refined using measured transferred energies. A rigorous and lengthy correlation study of the two cases (recommended versus refined analyses) of the dynamic RAMs to the CAPWAP-calculated capacities was completed to assess possible improvements in the accuracy and variability. The overall accuracy and variability in calculating the Q_{ult} using the modified Gates formula, GRLWEAP, and the Case method were improved.

7.1.2.2 Conclusions

The following conclusions were made on the basis of the dynamic analyses:

- The general bearing graphs can be used to help estimate the refusal criterion and to the select pile size and the driving system for driven steel H-piles in Winnipeg.
- The refined dynamic RAMs provide good predictor tools to estimate the Q_{ult} of driven steel H-piles in Winnipeg provided the recommended input variables and measured transferred energies are used. The dynamic RAMs are meant to be used in conjunction with the static analyses presented in Chapter 4.
- Resistance factors can be assigned to each refined dynamic RAM based their reliability for the purpose of ULS design.

7.1.3 Chapter 6 – Serviceability Analysis

The following describes the summary of the work performed in Chapter 6 and the conclusions made from the results of the work.

7.1.3.1 Summary

Since it was determined from the static and dynamic analyses that the Q_{ult} should not be based on the structural strength of steel, it is safe to assume that the Q_{SLS} should not be either. An attempt to develop a more reliable SLS criterion was therefore made on the basis of the measured load-displacement behavior of the H-piles. Mobilizing capacities and the corresponding settlements at the pile toe and pile head were assessed. It was determined that the capacities mobilized at reasonable top-of-pile settlements were significantly lower than the Q_{ult}. The capacities mobilized at reasonable settlements were in the order of those determined by the conventional SLS criterion, but with less variability, because they were based on measured values. Based on the findings of the serviceability analyses, the SLS criterion proposed by the author is more reliable than the conventional SLS criterion.

7.1.3.2 Conclusions

The following conclusions were made on the basis of the serviceability analyses:

- Serviceability limit states capacity should not be determined on the basis of a single global formula (Equation 1.4) that is meant to encompass the serviceability of all H-pile sizes and lengths in all soil and rock conditions.
- The serviceability of driven steel H-piles can be assessed on the basis of settlement. The top-of-pile settlement of an H-pile is based on the applied load, load distribution, mobilizing soil settlements at the pile toe, and elastic compression in the pile shaft.

7.2 Thesis Benefits to the Engineering Field

The work performed in this thesis is a benefit to the academic and practicing geotechnical engineering community in Winnipeg. It provides engineers with an improved understanding of the capacity and load-displacement behavior of driven steel H-piles. This increased knowledge has improved the conventional LSD criteria to be more complete and representative of the actual soils and rock underlying Winnipeg. This thesis also provides a base for further research on the capacity and load-displacement behavior of driven steel H-piles. Considering the increasingly frequent use of this pile type in Winnipeg, additional research would be helpful.

Predictor tools (the refined static RAMs) for estimating the Q_{ult} were developed on the basis of the static analyses performed in Chapter 4. Understanding the Q_{ult} of H-piles will help engineers to make more reliable foundation recommendations in terms of satisfying the ULS condition. The measured Q_{ult} of H-piles determined in the static analyses also serve as target values for the dynamic RAMs, if dynamic RAMs are preferred.

The dynamic analyses were completed to present important dynamic variables, as they are related to pile driving, and their influence on the Q_{ult} and $Q_{b\text{-}ult}$ of driven steel H-piles. The refined dynamic RAMs presented in Chapter 5 provide engineers with the ability to estimate the Q_{ult} of driven steel H-piles with increased confidence, which will also lead to more reliable designs in terms of satisfying the ULS condition. Additionally, understanding the dynamic behavior of driven steel H-piles will result in fewer surprises and better solutions to problems that occur during construction.

The main benefit of assessing the settlement of driven steel H-piles under service loads in Chapter 6 is to understand mobilized capacity and the corresponding settlements for the purpose of SLS design. Appreciating the amount of H-pile settlement that occurs under a service load in relation to the amount of capacity mobilized will provide a sense of assurance in SLS design. Recognizing the difference between mobilizing settlements from long-term settlements, such as creep, may prevent panic when an initial top-of-pile survey is completed and compared to an as-built survey or a survey completed many years later.

7.3 LSD Recommendations

This thesis was meant to critique and improve the existing conventional LSD design criteria (summarized in Chapter 1). The intent was to provide the geotechnical tools (the RAMs presented in Chapters 4 and 5 and the load-displacement behavior described in Chapter 6) to aid engineers in predicting the Q_{ult} and settlement of driven steel H-piles in Winnipeg for the purpose of developing more reliable LSD criteria. The author was originally concerned that the thesis analyses would discredit the conventional design criteria in a manner that would result in a requirement of more piles to satisfy the LSD conditions; or the conventional design criteria would discredit the thesis work. The author is mindful to the fact that the conventional design method works and has for many years; however, he is unconvinced of the principal theory behind its success. Founding the design capacities on the structural strength of steel rather than on the weaker component (soil and rock) does not seem dependable or safe.

The thesis work revealed a large amount of variability in the measured ultimate capacities (*i.e.* Q_{ult}, $Q_{b\text{-}ult}$, and $Q_{s\text{-}ult}$), raising the author's concerns of reliability of the conventional design criteria, which uses a global equation (Equation 1.4) to estimate Q_{SLS} and Q_{ULS}. The SLS and ULS assessment and analyses completed in this thesis addresses these concerns and presents more representative tools for predicting the performance of driven steel H-piles in Winnipeg under service and failure loads.

The work performed in this thesis showed positive results. The RFs suggested in the 2010 version of the NBCC are conservative for determining the Q_{ULS} of driven steel H-piles in Winnipeg, and the description of the RAMs is incomplete. The work shows that higher RFs for dynamic monitoring, and separate RFs for the modified Gates formula, GRLWEAP, and the Case method are justified for driven steel H-piles in Winnipeg. Additionally, clarification of and distinction between the Case method and CAPWAP is required due to significant differences in accuracy and reliability between the two RAMs. The number of H-piles tested on a project should be conducted to cover all site conditions, especially large projects where spatial variability may be significant. To justify using a RF of 0.75, AASHTO (2010) recommends that the driving criteria be established by dynamic testing conducted on 100% of production piles. However, the number piles tested should be based on the size of the project and risk associated with that project. If spatial variability can be mapped with confidence throughout the site, the number of piles tested can be significantly decreased. Arguments can be made that uniform stratigraphy and strengths of the strata do not require more than 1 test. However, variability is inherent in karst bedrock and there is no way to guarantee uniformity at a site. A specified dynamic testing program should be established by the structural and geotechnical engineers and should allow for additional piles to be tested when site anomalies are encountered.

The conventional SLS criterion is not only fundamentally incorrect but it is also incomplete, because it does not consider settlement. The Q_{SLS} should be determined on the basis of the amount of settlement that can be tolerated for a structure. The work shows that the Q_{SLS} can be linked to settlement on the basis of the estimated Q_{ult}. For this reason, a more accurate estimation of Q_{ult} will lead to a better approximation of pile settlement.

The following recommendations for LSD criteria of driven steel H-piles in Winnipeg are based on the work performed in this thesis and the opinion of the author, and are as follows:

- The $Q_{b\text{-}ult}$ should be estimated using existing static RAMs that are based on a compressive failure mode (e.g. Prakoso and Kulhawy 2002). The statistical results, summarized in Table 4.3, should be used to assist in determining the $Q_{b\text{-}ult}$.
- The $Q_{s\text{-}ult}$ should be estimated using the refined static RAMs developed in Chapter 4.
- The Q_{ult} should be estimated using the static analysis criteria (described in the first two bullets) or the refined dynamic RAMs presented in Chapter 5.

- The RFs and the corresponding RAMs, presented in Table 6.3, can be used to determine the Q_{ULS}.
- The refusal criterion can be roughly estimated using the bearing graphs presented in Chapter 5, but should be confirmed by a GRLWEAP analysis.
- The Q_{SLS} should be based on the capacity mobilized at tolerable top-of-pile settlements. The calculated Q_{ult} can be used as benchmark for determining the mobilized capacity at tolerable top-of-pile settlements. Consideration of elastic compression of the pile shaft and mobilizing settlements at the pile toe should be made to confirm the top-of-pile settlements under applied service loads.
- A minimum of 5 percent of the H-piles for a building structure, encompassing all subsurface conditions, should be PDA tested with CAPWAP analysis. For bridges, a minimum of one PDA test and CAPWAP analysis should be conducted on an H-pile at each pier and abutment and per subsurface condition.

7.3.1 PDA Testing and CAPWAP Analysis

The PDA test and CAPWAP analysis should become the standard design RAM for large structures that are to be founded on driven steel H-piles; rather than used as a mere tool to confirm Q_{ult}. This would reduce the risk of over (or under) designing the foundation system at the design stage and result in more cost-effective and efficient designs. However, it can be difficult to convince clients to spend extra money at the pre-design and design stages in a budget conscientious world.

7.3.1.1 Testing Condition

The potential for pile set-up and relaxation should be considered when developing a PDA test and CAPWAP analysis schedule. End of initial drive and BOR tests should be performed. Comparing the results of the EOID and BOR testing conditions will determine if a re-driving program is required. Time between EOID and BOR tests should be at least 6 days to allow for set-up or relaxation to occur (Likins and Rausche, 2004). The best way to represent long term Q_{ult} is to perform re-drives after a significant amount of time has passed after the EOID; the longer the wait, the better the results will represent long term Q_{ult}.

7.4 Future Research

The work performed in this thesis will improve the understanding of the capacity and load-displacement behavior of driven steel H-piles in Winnipeg strata; however, improvements can be made. The sample number of PDA tested and CAPWAP analysed H-piles is good but needs to be increased to improve accuracy of the proposed static and dynamic RAMs and decrease variability in the results. This will lead to refinement of the recommended RFs. A reliability based design of RFs can conducted if a substantial amount of data is available.

Assessment of the performance of H-piles under service load should be further refined. This may be done by means of SLTs. Although extremely expensive, relative to PDA tests and CAPWAP analyses, the SLT will provide the ability to take more accurate measurements of settlement versus applied load. A SLT will also determine the load-distribution shape along the pile shaft and how the resistance is mobilized under service conditions. Static load tests will also provide the opportunity to discern and refine damping and quake values of Winnipeg strata for further improvement of WEA.

7.5 Final Thesis Discussion

This chapter provides a brief summary of the key misconceptions and omissions of the conventional design criteria shown for driven steel H-piles in Winnipeg. The findings in this thesis substantiate the hypothesis presented in Chapter 1, such that:

- The Q_{ult} of driven steel H-piles in Winnipeg is governed by the strength and condition of the soil and rock in which the H-piles are embedded, and the pile driving system used to install them; not the structural strength of steel.
- The RAMs can be refined to more accurately predict the Q_{ult} of driven steel H-piles in lacustrine clay, till and karst bedrock. Higher RFs can be used to determine Q_{ULS} based on the refined RAMs.
- General bearing graphs can be constructed and used as a stand-alone RAM to predict the Q_{ult} of driven steel H-piles in lacustrine clay, till and karst bedrock. The corresponding refusal criterion can be roughly estimated by the bearing graphs.
- The CAPWAP-based load-displacement curves can be used to estimate pile settlement and develop reasonable SLS criteria for calculation of Q_{SLS} for driven steel H-piles.

By and large, the objectives in this thesis were to improve the conventional LSD criteria for driven steel H-piles in Winnipeg. Rigid ULS design criteria is essential and the SLS design criterion needed to be revamped and improved to account for known settlement of H-piles. The author is confident that the work performed in this thesis has achieved these objectives.

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Appendices

Appendix A:

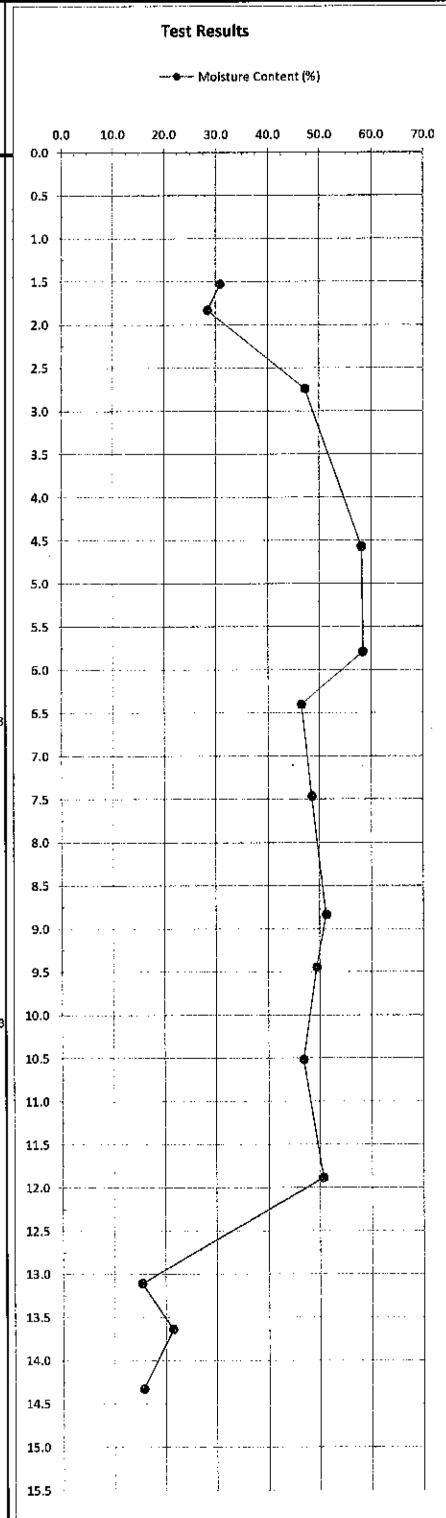
Test Hole Logs

Site 1

[redacted]		PROJECT #	TEST HOLE NO.
		113339	1

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	South end of Site	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	April 23, 2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	20

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
0.00	100.00		0 - 2.7 m CLAY (Fill) - silty - trace sand, trace gravel - trace wood and asphalt pieces - grey to black (variable) - moist, high plasticity - variable consistency (firm to stiff)		
0.50	99.50				
1.00	99.00				
1.50	98.50				
2.00	98.00				
2.50	97.50				
3.00	97.00		2.7 - 13.1 m CLAY - silty - grey, mottled brown below 4.6 m - stiff, moist - high plasticity - trace silt inclusions (< 6 mm dia.)		
3.50	96.50				
4.00	96.00				
4.50	95.50				
5.00	95.00				
5.50	94.50				
6.00	94.00				Qu = 35 kPa TV = 57 kPa PP = 78 kPa γbulk = 16.7 kN/m^3
6.50	93.50				
7.00	93.00				
7.50	92.50				
8.00	92.00				
8.50	91.50				
9.00	91.00		- grey below 8.8 m		
9.50	90.50				Qu = 36 kPa TV = 53 kPa PP = 58 kPa γbulk = 17.0 kN/m^3
10.00	90.00				
10.50	89.50				
11.00	89.00				
11.50	88.50				
12.00	88.00				
12.50	87.50				
13.00	87.00		13.1 - 14.3 m SAND - tan - loose, wet		
13.50	86.50			8	
14.00	86.00				
14.50	85.50		14.3 - 16.0 m SILT (Till) - trace sand, trace gravel - grey, loose, moist - 300 mm diameter boulder at 14.5 m		
15.00	85.00				
15.50	84.50		- very dense, dry below 15.9 m	>50	spoon bouncing on cobble / boulder



[redacted]	PROJECT # 113339	TEST HOLE NO. 1

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	South end of Site	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	April 23, 2012
METHOD:	Acker Soil Sentry - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	20

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
16.00	84.00		16.0 to 20.0 m LIMESTONE BEDROCK		
16.50	67.50		off white to light brown in color		
			massive, vertical fractures in upper 1.5 m		
17.00	67.00		horizontal fractures generally tight		
			evidence of water flow in some fractures		
17.50	66.50		25 mm thick red clay seam at 17.3 m		
			15 mm thick red clay seam at 18.7 m		
18.00	66.00				
			16.0 to 17.0 m) L recovered 95%		
18.50	65.50		RQD 65%		
			17.0 to 18.5 m) L recovered 100%		
19.00	65.00		RQD 80%		
			18.5 to 20.0 m) L recovered 98%		
19.50	64.50		RQD 90%		
20.00	64.00		20.0 m END OF TEST HOLE AT 20.0 m IN LIMESTONE BEDROCK		

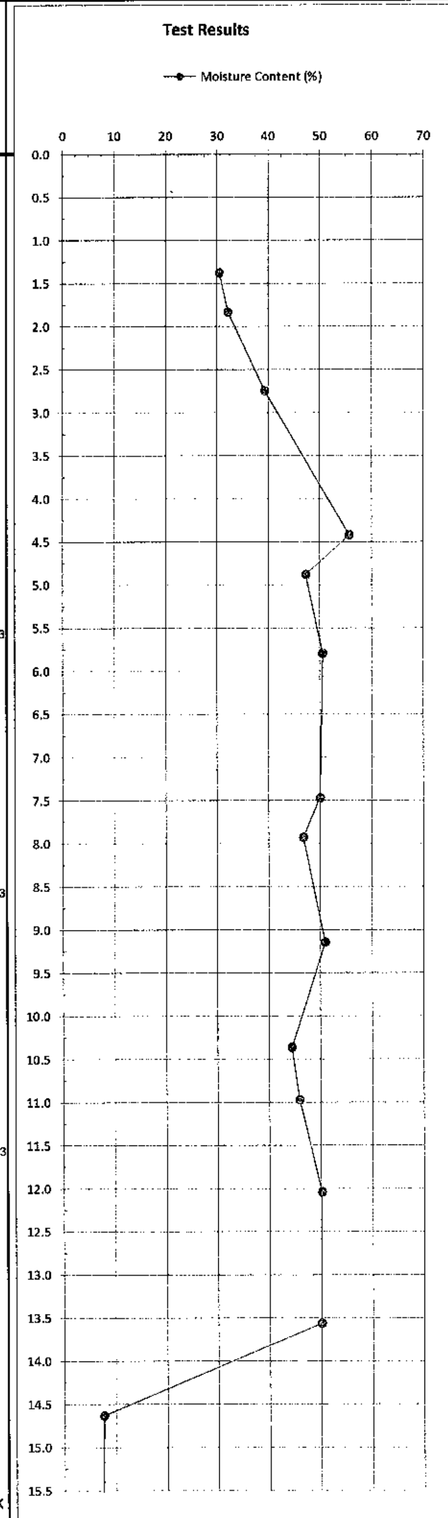
NOTES:

1. Seepage and sloughing from sand layer.
   Water level at 8.8 m below grade 10 minutes after encountering sand
2. Auger refusal at 15.5 m, switched to HQ coring.
3. Test hole backfilled with cement/bentonite grout and capped with bentonite chips

[redacted]	PROJECT # 113339	TEST HOLE NO. 2

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	Northeast Corner of Site	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	April 23, 2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	21.6

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
0.00	100.00		0 - 3.05 m CLAY (Fill) - silty - trace sand, trace gravel		
0.50	99.50		- trace wood chips, topsoil - dark grey to black (variable)		
1.00	99.00		- moist, high plasticity - variable consistency (firm to stiff)		
1.50	98.50				
2.00	98.00				
2.50	97.50				
3.00	97.00		3.05 - 14.33 m CLAY - silty - grey		
3.50	96.50		- stiff, moist - high plasticity		
4.00	96.00		- trace silt inclusions (< 6 mm dia.)		
4.50	95.50				Qu = 56 kPa
5.00	95.00				TV = 75 kPa PP = 106 kPa
5.50	94.50				γbulk = 17.0 kN/m^3
6.00	94.00				
6.50	93.50				
7.00	93.00				
7.50	92.50				Qu = 50 kPa
8.00	92.00				TV = 56 kPa PP = 65 kPa
8.50	91.50				γbulk = 17.3 kN/m^3
9.00	91.00				
9.50	90.50				
10.00	90.00				
10.50	89.50				Qu = 43 kPa
11.00	89.00				TV = 50 kPa PP = 53 kPa
11.50	88.50				γbulk = 16.5 kN/m^3
12.00	88.00				
12.50	87.50				
13.00	87.00				
13.50	86.50				
14.00	86.00				
14.50	85.50		14.33-15.75 SILT (Till) - trace sand, trace gravel (<6 mm)		
15.00	85.00		- grey to light brown in color - moist, loose becoming very dense at 15.0 m		
15.50	84.50			>50	spoon bouncing on rock



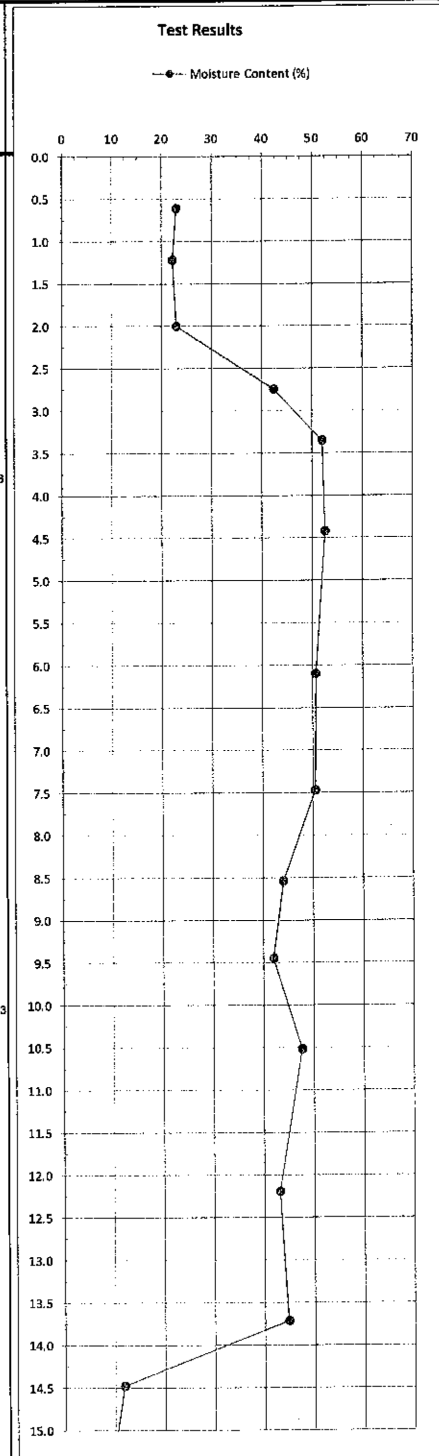
[redacted]		PROJECT # 113339	TEST HOLE NO. 2
PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	Northeast Corner of Site	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	April 23, 2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	21.6

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
16.00	84.00		15.75 - 21.6 m LIMESTONE BEDROCK		
16.50	67.50		off white to light brown in color		
17.00	67.00		massive, vertical fractures in upper 1 m horizontal fractures generally tight		
17.50	66.50		evidence of water flow in some fractures 15 mm thick clay seams at 16.9 m, 18.9 m, 19.4 m		
18.00	66.00		15 to 25 mm thick clay seams at 21 m, 21.3 m		
18.50	65.50		15.75 to 17.1 m) L recovered 95% RQD (15.75 to 16.5 m) 31%		
19.00	65.00		RQD (16.5 to 17.1 m) 95% 17.1 to 18.6 m) L recovered 100%		
19.50	64.50		RQD 83% 18.6 to 20.1 m) L recovered 95%		
20.00	64.00		RQD 93% 20.1 to 21.6 m) L recovered 95%		
20.50	63.50		RQD 91%		
21.00	63.00				
21.50	62.50		END OF TEST HOLE AT 21.6 m IN LIMESTONE BEDROCK		

NOTES:
1. Seepage observed at 13.6 m (possible sand layer)
   Water level 13.7m below grade
2. Auger refusal at 15.9 m, switched to HQ coring.
3. Test hole backfilled with cement/bentonite grout and capped with bentonite chips

[redacted]	PROJECT #	TEST HOLE NO. 3
PROJECT: [redacted]	LOGGED BY: [redacted]	
LOCATION: South East Corner of Parking Lot	REVIEWED BY: [redacted]	
CONTRACTOR: Paddock Drilling Ltd.	DRILL DATE: May 7,2012	
METHOD: Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m): 26.2	

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION		SPT (N) (blows / 300 mm)	COMMENTS
0.00	100.00		0 - 0.04 m	ASPHALT (40 mm thick)		
			0.04 - 1.5 m	CLAY (Fill) - silty		
0.50	99.50			- some sand, some gravel - black		
1.00	99.00			- stiff, dry to moist, intermediate plasticity		
1.50	98.50		1.5 - 2.3 m	SILT - light grey, loose, moist		
2.00	98.00					
			2.3 - 15.1 m	CLAY - silty		
2.50	97.50			- grey/brown mottled - stiff, moist, high plasticity		
3.00	97.00					Qu = 37 kPa TV = 54 kPa PP = 60 kPa γbulk = 17.0 kN/m³
3.50	96.50					
4.00	96.00					
4.50	95.50					
5.00	95.00					
5.50	94.50					
6.00	94.00			- trace silt inclusions (< 12 mm dia.) below 6.1 m		
6.50	93.50					
7.00	93.00					
7.50	92.50					
8.00	92.00					
8.50	91.50			- grey below 8.7 m		
9.00	91.00					
9.50	90.50					Qu = 48 kPa TV = 57 kPa PP = 52 kPa γbulk = 17.0 kN/m³
10.00	90.00					
10.50	89.50					
11.00	89.00					
11.50	88.50					
12.00	88.00					
12.50	87.50					
13.00	87.00					
13.50	86.50					
14.00	86.00					
14.50	85.50					
15.00	85.00					



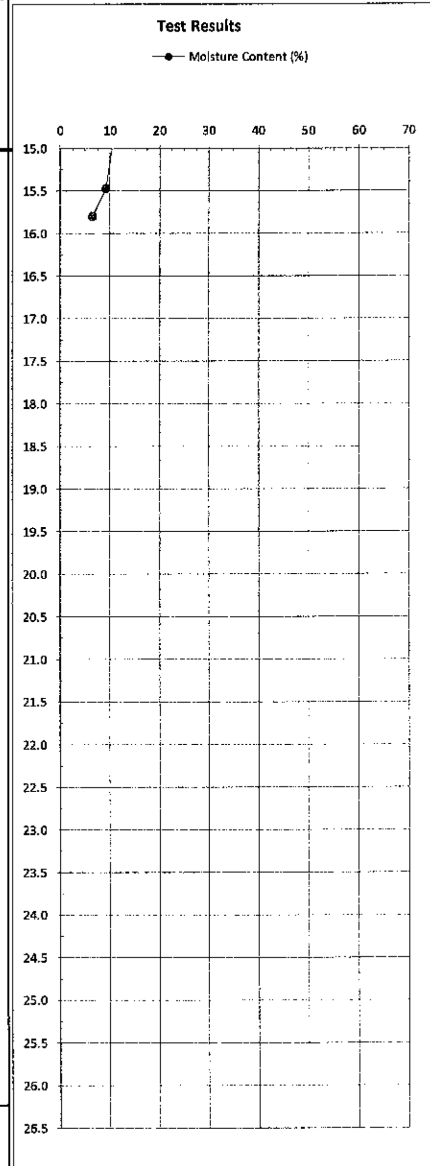
[redacted]	PROJECT # 113339	TEST HOLE NO. 3

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	South East Corner of Parking Lot	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	May 7,2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	26.2

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
15.00	85.00		15.1 - 16.0 m SILT (Till) - trace sand, trace gravel		
15.50	69.50		- light grey, very dense, dry	86	blows for 175 mm
16.00	69.00		16 m LIMESTONE BEDROCK		
16.50	68.50		bedrock is whitish / light brown color medium strong to strong		
17.00	68.00		close to moderately close discontinuity spacing gapped to open apertures		
17.50	67.50		evidence of water flow, seepage filling varies: silt, clay or sand		
18.00	67.00		16.0 to 17.1 m) L recovered 97%		
18.50	66.50		RQD (16.0 to 17.1 m) 40% 17.1 to 18.6 m) L recovered 87%		
19.00	66.00		RQD 38% silt filling present		
19.50	65.50		18.6 to 20.1 m) L recovered 40% RQD 17%		
20.00	65.00		grey silt and clay filling evidence 20.1 to 20.8 m) L recovered 80%		
20.50	64.50		RQD 32% fine sand from 20.1 to 20.5 m		
21.00	64.00		20.8 to 21.6 m) L recovered 60% RQD 32%		
21.50	63.50		75 mm olive grey clay just above 21.6 m 21.6 to 23.2 m) L recovered 60%		
22.00	63.00		RQD 43% grey & yellowish brown filling from 21.6 to 21.8 m		
22.50	62.50		23.2 to 24.7 m) L recovered 90% RQD 77%		
23.00	62.00		100 mm yellowish brown silt filling at 23.4 m		
23.50	61.50				
24.00	61.00				
24.50	60.50		24.7 to 26.2 m)		
25.00	60.00		No limestone rock recovery light brown fine sand over silt / clay		
25.50	59.50				
26.00	59.00		26.2 m END OF TEST HOLE AT 26.2 m IN SAND		
26.50	58.50				

NOTES: 1. Auger refusal at 16.1 m, switch to HQ coring.

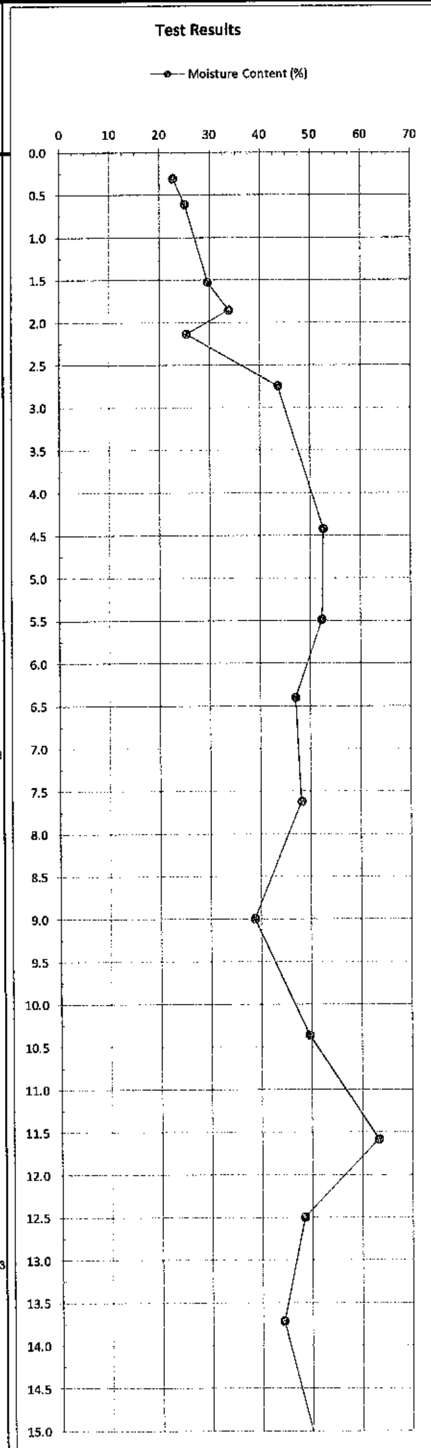
2. Test hole backfilled with cement/bentonite grout and upper 300 mm filled with bentonite chips



[redacted]	PROJECT # 113339	TEST HOLE NO. 4

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	South Central Area of Parking Lot	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	May 7, 2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	24.5

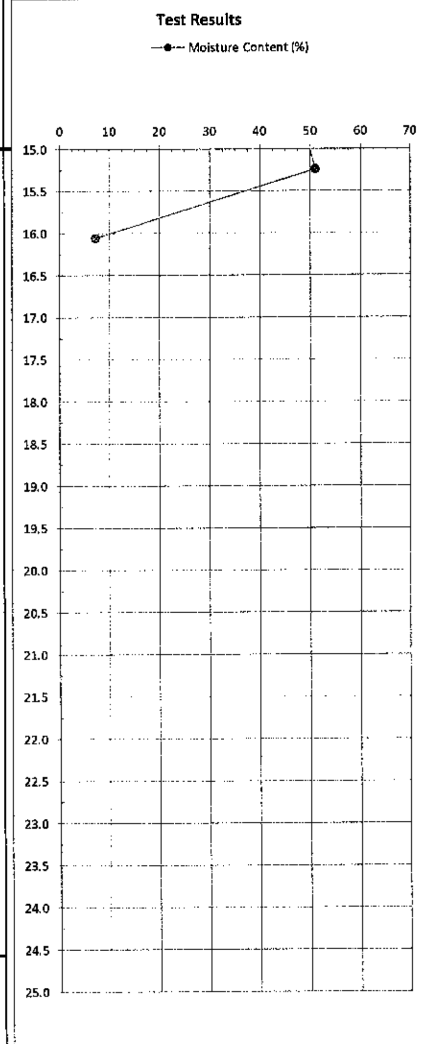
DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
0.00	100.00		0 - 0.03 m ASPHALT (30 mm)		
			0.03 - 1.5 m CLAY (Fill) - silty		
0.50	99.50		- some sand, some gravel		
			- black		
1.00	99.00		- stiff, dry to moist, intermediate plasticity		
1.50	98.50		1.5 - 2.0 m CLAY - silty		
			- dark grey, stiff, moist, high plasticity		
2.00	98.00		2.0 - 2.6 m SILT		
			- light grey		
2.50	97.50		- loose, moist		
			2.6 - 15.2 m CLAY - silty		
3.00	97.00		- grey/brown mottled		
			- stiff, moist, high plasticity		
3.50	96.50				
4.00	96.00				
			- trace silt inclusions (< 6 mm dia.) below 4.4 m		
4.50	95.50				
5.00	95.00				
5.50	94.50				
6.00	94.00				
					Qu = 53 kPa
6.50	93.50				TV = 51 kPa
					PP = 59 kPa
7.00	93.00				γbulk = 16.8 kN/m^3
7.50	92.50				
			- grey below 7.6 m		
8.00	92.00				
8.50	91.50				
9.00	91.00				
9.50	90.50				
10.00	90.00				
10.50	89.50				
11.00	89.00				
11.50	88.50				
12.00	88.00				
					Qu = 36 kPa
12.50	87.50				TV = 52 kPa
					PP = 47 kPa
13.00	87.00		- soft between 13.1 and 13.7 m		γbulk = 17.4 kN/m^3
13.50	86.50				
14.00	86.00				
14.50	85.50				
15.00	85.00				



[redacted]	PROJECT # 113339	TEST HOLE NO. 4

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	South Central Area of Parking Lot	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	May 7,2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	24.5

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
15.00	85.00				
15.50	69.50		15.2 - 16.7 m SILT (Till) - trace sand, trace gravel - 810 mm diameter boulder at 15.2 m depth - light grey - dense, dry		
16.00	69.00				
16.50	68.50				
17.00	68.00		16.7 - 24.5 m LIMESTONE BEDROCK whitish / light brown color medium strong to strong close to moderately close discontinuity spacing closed to gapped apertures evidence of water flow nvidence of silt filling		
17.50	67.50				
18.00	67.00				
18.50	66.50		16.0 to 17.1 m) L recovered 97%		
19.00	66.00		RQD (16.5 to 17.1 m) 40% 17.1 to 18.6 m) L recovered 87%		
19.50	65.50		RQD 38% 18.6 to 20.1 m) L recovered 40%		
20.00	65.00		RQD 17% 20.1 to 21.6 m) L recovered 30%		
20.50	64.50		RQD 32%		
21.00	64.00				
21.50	63.50		21.6 to 23.2 m) L recovered 60%		
22.00	63.00		RQD 43%		
22.50	62.50				
23.00	62.00				
23.50	61.50				
24.00	61.00				
24.50	60.50		24.5 m END OF TEST HOLE AT 24.5 m IN LIMESTONE BEDROCK		



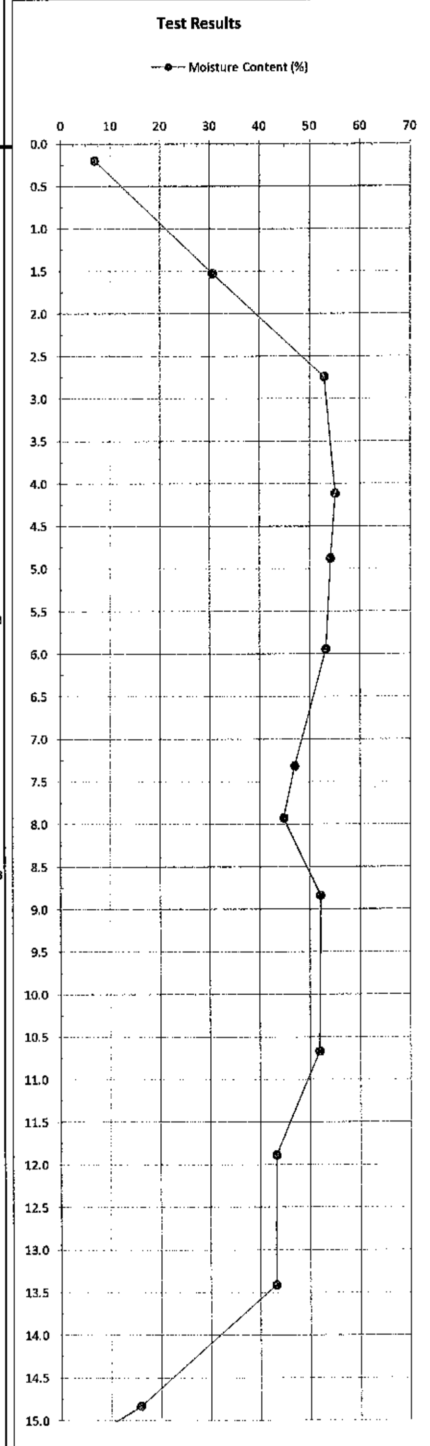
NOTES:

1. Auger refusal at 16.7 m, switched to HQ coring.
2. Test hole backfilled with cement/bentonite grout and capped with bentonite chips

[redacted]	PROJECT #	TEST HOLE NO.
	113339	5

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	North East Corner of Site [redacted]	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	May 8,2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	18.6

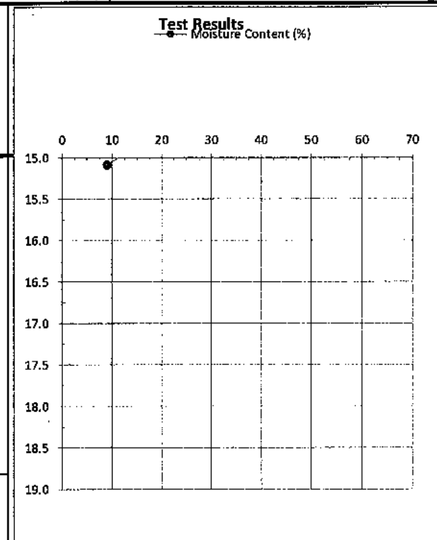
DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
0.00	100.00		0 - 0.09 m CONCRETE (90 mm thick)		
			0.09 - 0.3 m GRAVEL (Base) - trace sand, brown, loose, moist		
0.50	99.50		0.3 - 1.5 m CLAY (Fill) - silty - trace sand, trave gravel, pieces of brick		
1.00	99.00		- black, stiff, moist, high plasticity		
1.50	98.50		1.5 - 14.2 m CLAY - silty - mottled grey/ brown		
2.00	98.00		- stiff, moist, high plasticity - trace silt inclusions (< 15 mm dia.)		
2.50	97.50				
3.00	97.00				
3.50	96.50				
4.00	96.00				
4.50	95.50				
5.00	95.00				Qu = 27 kPa TV = 54 kPa
5.50	94.50				PP = 75 kPa γbulk = 16.6 kN/m^3
6.00	94.00		- grey below 6.1 m		
6.50	93.50				
7.00	93.00				
7.50	92.50				
8.00	92.00				Qu = 51 kPa TV = 54 kPa
8.50	91.50				PP = 71 kPa γbulk = 17.1 kN/m^3
9.00	91.00				
9.50	90.50				
10.00	90.00				
10.50	89.50				
11.00	89.00				
11.50	88.50				
12.00	88.00				
12.50	87.50				
13.00	87.00				
13.50	86.50				
14.00	86.00				
14.50	85.50		14.2 - 15.5 m SILT (Till) - trace sand, trace gravel - tan, loose, moist - dense, dry below 15.0 m		
15.00	85.00		- cobbles, boulders encountered from 14.2 to 14.8 m	60	blows for 100 mm spoon bouncing on cobble



[redacted]		PROJECT # 113339	TEST HOLE NO. 5

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	North East Corner of Site [redacted]	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	May 8,2012
METHOD:	Acker Soil Sentry - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	18.6

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
15.00	85.00				
15.50	69.50		15.5 - 18.6 m LIMESTONE BEDROCK whitish / light brown color medium strong to strong close to moderately close discontinuity spacing closed to gapped apertures evidence of water flow evidence of silt filling 15.5 to 17.1 m) L recovered 97% RQD 77% 17.1 to 18.6 m) L recovered 97% RQD 87%		
16.00	69.00				
16.50	68.50				
17.00	68.00				
17.50	67.50				
18.00	67.00				
18.50	66.50				
19.00	66.00		18.6 m END OF TEST HOLE AT 18.6 m LIMESTONE BEDROCK		



Notes:

1. Seepage below 14.2 m from cobbly, bouldery silt till. Water level at 11.6 m below ground surface after drilling.

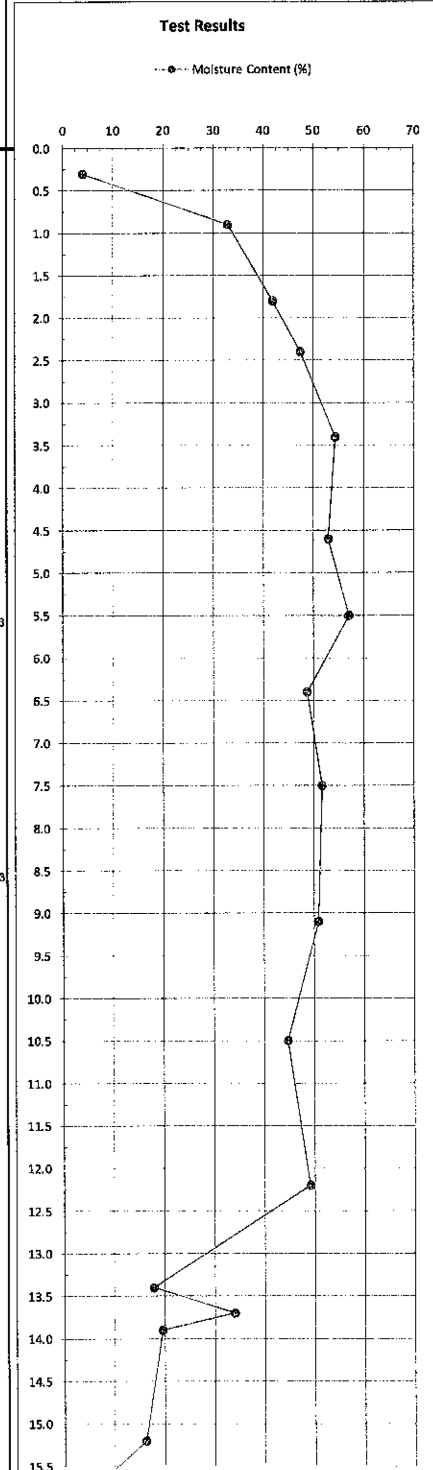
2. Auger refusal at 15.5 m, switched to HQ coring.

3. Test hole backfilled with cement/bentonite grout and capped with bentonite chips

[redacted]	PROJECT #	TEST HOLE NO.
	113339	6

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	South West Corner [redacted]	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	June 5,2012
METHOD:	Acker SS - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	23.2

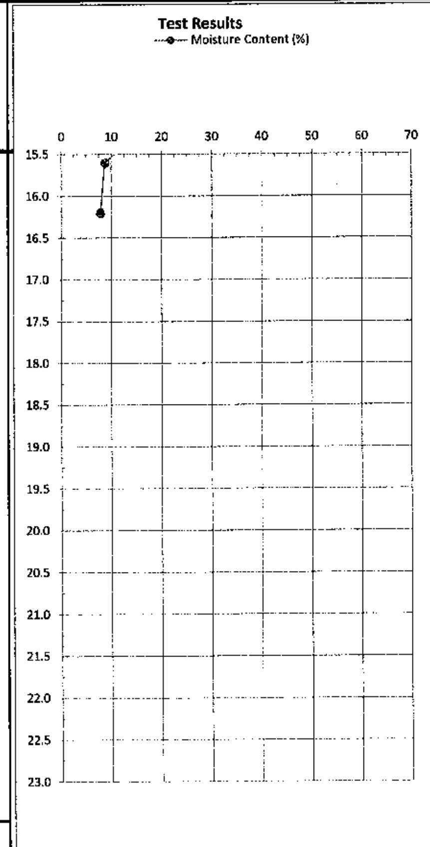
DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION		SPT (N) (blows / 300 mm)	COMMENTS
0.00	100.00		0-0.05 m	ASPHALT - 50 mm thick		
			0.05 - 0.5 m	SAND AND GRAVEL (Fill)		
0.50	99.50			-light brown, loose, dry to moist		
			0.5 - 1.8 m	CLAY (Fill) silty		
1.00	99.00			-grey, stiff, moist		
1.50	98.50					
			1.8 - 2.0 m	SILT - clayey, light brown, soft, moist		
2.00	98.00		2.0 - 12.8 m	CLAY - silty		
2.50	97.50			-mottled brown / grey		
				-stiff, moist		
3.00	97.00			-high plasticity		
				-trace gypsum inclusions (<3mm)		
3.50	96.50					
4.00	96.00					
				-trace silt inclusions below 4.3 m		
4.50	95.50					
						Qu = 34 kPa
5.00	95.00					TV = 54 kPa
						PP = 70 kPa
5.50	94.50					γbulk = 16.1 kN/m³
6.00	94.00			-grey below 6.1m		
				-softer between 6.1 - 7.6m		
6.50	93.50					
7.00	93.00					
7.50	92.50					
						Qu = 32 kPa
8.00	92.00					TV = 56 kPa
						PP = 51 kPa
8.50	91.50					γbulk = 17.0 kN/m³
9.00	91.00					
9.50	90.50					
10.00	90.00					
10.50	89.50					
11.00	89.00					
11.50	88.50					
12.00	88.00					
12.50	87.50					
			12.8-15.4 m	SAND		
13.00	87.00			- trace gravel		
				- light grey		
13.50	86.50			- loose to compact		
				- wet		
14.00	86.00			- silty to and silt around 14 m	12	
14.50	85.50					
15.00	85.00					



[redacted]	PROJECT # 113339	TEST HOLE NO. 6

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	South West Corne[redacted]	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	June 5,2012
METHOD:	Acker Soil Sentry - 125 mm Solid Stem Augers, HQ Coring	DRILL DEPTH (m):	23.2

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N) (blows / 300 mm)	COMMENTS
15.50	84.50		15.4 - 16.2 m SILT (till) -trace: sand, gravel, cobbles, boulders	58	
16.00	68.50		- light grey, dry to moist - dense to very dense		
16.50	68.00		16.2 - 23.2 m LIMESTONE BEDROCK		
17.00	67.50		whitish / light brown color; medium strong to strong		
17.50	67.00		close to moderately close discontinuity spacing; gapped apertures		
18.00	66.50		evidence of water flow; occassional clay seam (Class 1 filling)		
18.50	66.00				
19.00	65.50				
19.50	65.00		16.2 to 17.1 m) L recovered 98%; RQD 81%		
20.00	64.50		17.1 to 18.6 m) L recovered 97%; RQD 73%		
20.50	64.00		18.6 to 20.1 m) L recovered 98%; RQD 72%		
21.00	63.50		20.1 to 21.6 m) L recovered 98%; RQD 62%		
21.50	63.00		21.6 to 23.2 m) L recovered 100%; RQD 88%		
22.00	62.50				
22.50	62.00				
23.00	61.50				



END OF TEST HOLE AT 23.2m IN LIMESTONE BEDROCK

NOTES:

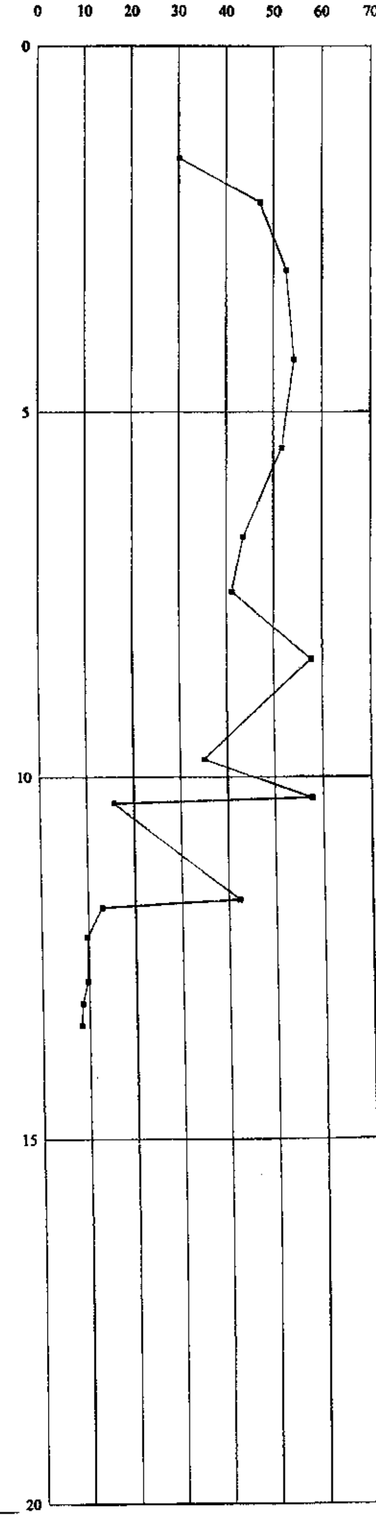
1. Seepage / sloughing observed from sand layer. Water level at 7.9m below ground surface 15 min. after drilling to 15.7 m
2. Solid stem augers to 15.7 m, HQ coring below 15.7 m
3. Test hole backfill with cement/bentonite grout and capped with bentonite chips.

Site 2

[redacted]	Logged/Dwn.: [redacted] Checked: [redacted]	Test Hole No. 10-1	Project No. [redacted]
[redacted]		DATE OF INVEST : MAY 17, 2010 DRILL : SUBTERRANEAN 405 mm AUGER	

SAMPLE NO.	DEPTH (M)	ELEV. (M)	SYM	SOIL DESCRIPTION	
	0.00	232.40		0.00-0.91 FILL 60 mm ASPHALT OVER 200 mm OF LIMESTONE BASE OVER PITRUN	
	0.50	231.90			
	1.00	231.40		0.91-11.74 CLAY BROWN, STIFF, HIGH PLASTIC, SILTY UPPER CLAY APPEARED WEATHERED, BLOCKY (LARGE), TRACE SILT INCLUSIONS FINELY LAYERED	
	1.50	230.90			
	2.00	230.40			Qu-35.9 kPa Pp-69.4 kPa Tv-78.5 kPa W -17.49 KN/M
	2.50	229.90			
	3.00	229.40			
	3.50	228.90			
	4.00	228.40		SAME, FEW FINE GREY LAYERINGS	Qu-36.4 kPa Pp-81.4 kPa Tv-68.9 kPa W -17.00 KN/M
	4.50	227.90			
	5.00	227.40			
	5.50	226.90			
	6.00	226.40			
	6.50	225.90			
	7.00	225.40		SAME, MORE SILTY	Qu-84.1 kPa Pp-64.6 kPa Tv-85.1 kPa W -17.68 KN/M
	7.50	224.90			
	8.00	224.40			
	8.50	223.90		GREY, FEW FINE TILL LAYERINGS FIRM TO STIFF	Qu-34.8 kPa Pp-47.9 kPa Tv-45.9 kPa W -16.46 KN/M
	9.00	223.40			
	9.50	222.90		LARGE TILL INCLUSION, PUTTY LIKE	
	10.00	222.40			
	10.50	221.90			Qu-43.6 kPa Pp-45.5 kPa Tv-38.3 kPa W -16.86 KN/M
	11.00	221.40			
	11.50	220.90			
	12.00	220.40		11.74-13.64 GLACIAL SILT TILL SILT MATRIX WITH SAND AND GRAVEL DAMP, FEW COBBLES AND BOULDERS AT 13.0 LARGER BOULDER	
	12.50	219.90			
	13.00	219.40			
	13.50	218.90		DENSE	
	14.00	218.40		END OF TEST HOLE AT 14.64 AUGER REFUSAL ON PROBABLE BOULDERS	
	14.50	217.90			
	15.00	217.40		NOTE: 900 mm WATER IN TEST HOLE AT COMPLETION OF DRILLING. THE SEEPAGE WAS ENCOUNTERED IN THE UPPER AREA OF THE TILL AT +/- 12.0 BELOW GRADE THE TEST HOLE WAS BACKFILLED WITH AUGER CUTTINGS	

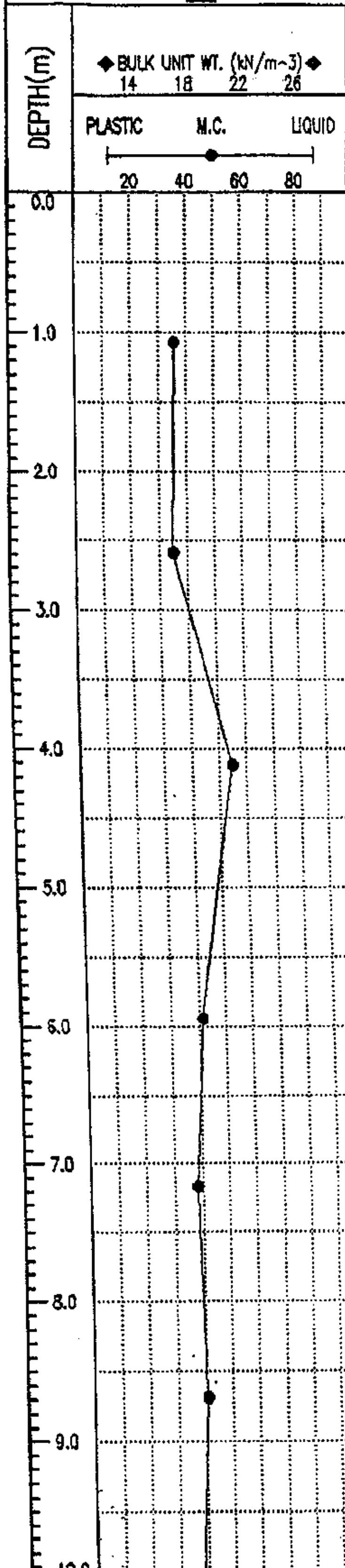
MOISTURE CONTENT (%)



PROJECT: [redacted]	DRILLED BY: PADDOCK DRILLING LTD.	TEST HOLE NO: TH-3
CLIENT: [redacted]	DRILL TYPE: ACKER MP5, 125 SS AUGERS	PROJECT NO: D534-001-01-02
PROJECT ENGINEER: [redacted]	LOCATION: SE CORNER OF PROP. BLDG.	ELEVATION: 231.404 (m)

SAMPLE TYPE: Grab Sample | Shelby Tube | Lab Submitted | No Recovery | Split Spoon | Wire Line-Type

BACKFILL TYPE: BENTONITE | PEA GRAVEL | SLOUGH | GROUT | DRILL CUTTINGS | SAND



SOIL SYMBOL

Soil Description

ASHPALT

CONCRETE

CLAY (Fill) – trace to some gravel, trace silt inclusions, trace pieces of brick
- brown mottled with light brown and grey
- firm to stiff, moist
- medium to high plasticity

CLAY (Fill) – trace to some gravel, trace pieces of brick
- black
- soft to firm, moist
- medium to high plasticity
- slight Creosote like odour

CLAY – trace silt inclusions/nodules, trace oxidation inclusions
- brown
- firm to stiff, moist
- high plasticity

– becoming grey and softer below 6.7m

SAMPLE TYPE	SAMPLE NO
Grab Sample	30
Grab Sample	31
Grab Sample	32
Grab Sample	33
Grab Sample	34
Grab Sample	35

INSTRUMENTATION DATA

▲ HV Shear Strength (kPa) ▲ 60 100 140 180

■ PP Shear Strength (kPa) ■ 60 100 140 180

◆ TV Shear Strength (kPa) ◆ 60 100 140 180

● QU Shear Strength (kPa) ● 60 100 140 180

ELEVATION(m): 231.0, 230.0, 229.0, 228.0, 227.0, 226.0, 225.0, 224.0, 223.0, 222.0

[redacted] Winnipeg, Manitoba	LOGGED BY: [redacted]	COMPLETION DEPTH: 18.0 m
	REVIEWED BY: [redacted]	COMPLETE: 20/08/98
	Fig. No:	Page 1 of 2

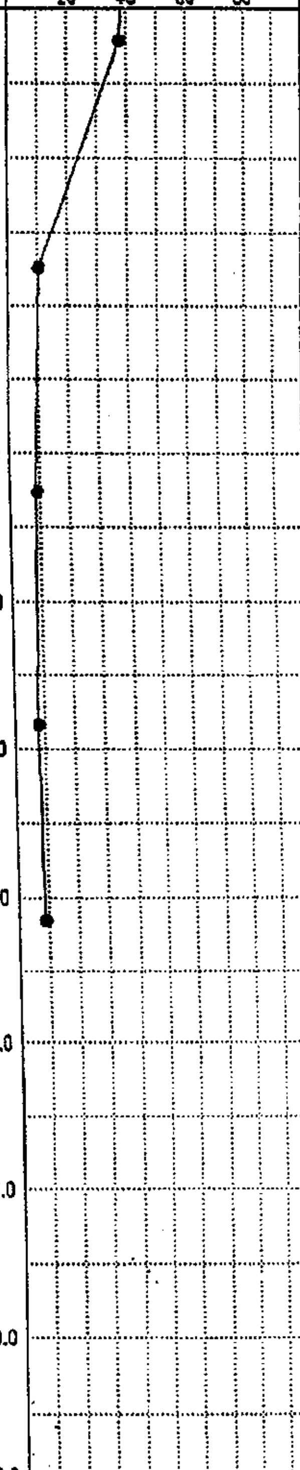
98/10/28 09:38AM (CSRP-HSC)

PROJECT: [redacted]	DRILLED BY: PADDOCK DRILLING LTD.	TEST HOLE NO: TH-3
CLIENT: [redacted]	DRILL TYPE: ACKER MP5, 125 SS AUGERS	PROJECT NO: D534-001-01-02
PROJECT ENGINEER: [redacted]	LOCATION: SE CORNER OF PROP. BLDG.	ELEVATION: 231.404 (m)

SAMPLE TYPE: Grab Sample | Shelby Tube | Lab Submitted | No Recovery | Split Spoon | Wire Line-Type

BACKFILL TYPE: BENTONITE | PEA GRAVEL | SLOUGH | GROUT | DRILL CUTTINGS | SAND

DEPTH(m) | ◆ BULK UNIT WT. (kN/m-3) ◆ 14 18 22 26 | PLASTIC M.C. LIQUID 20 40 60 80 | SOIL SYMBOL | Soil Description | SAMPLE TYPE | SAMPLE NO | INSTRUMENTATION DATA | ▲ HV Shear Strength (kPa) ▲ 60 100 140 180 | ■ PP Shear Strength (kPa) ■ 60 100 140 180 | ◆ TV Shear Strength (kPa) ◆ 60 100 140 180 | ● QU Shear Strength (kPa) ● 60 100 140 180 | ELEVATION(m)



Depth (m)	Soil Description	Sample No	Elevation (m)
10.0 – 11.3		36	221.0
11.3 – 18.1	SILT (Till) – some gravel and sand, trace clay – light brown to reddish brown – loose to dense, moist – becoming grey and very dense below 14m – water seepage observed after completion of drilling to 14m – cobbles/boulders encountered from 15.5m to 15.9m	37, 38, 40	220.0 – 213.0
18.1	AUGER REFUSAL AT 18.1m ON INFERRED BEDROCK		

Depth scale: 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0

Elevation scale: 221.0, 220.0, 219.0, 218.0, 217.0, 216.0, 215.0, 214.0, 213.0, 212.0

[redacted] Winnipeg, Manitoba	LOGGED BY: [redacted]	COMPLETION DEPTH: 18.0 m
	REVIEWED BY: [redacted]	COMPLETE: 20/08/98
	Fig. No:	Page 2 of 2

98/10/28 09:38AM (CSRP-HSC)

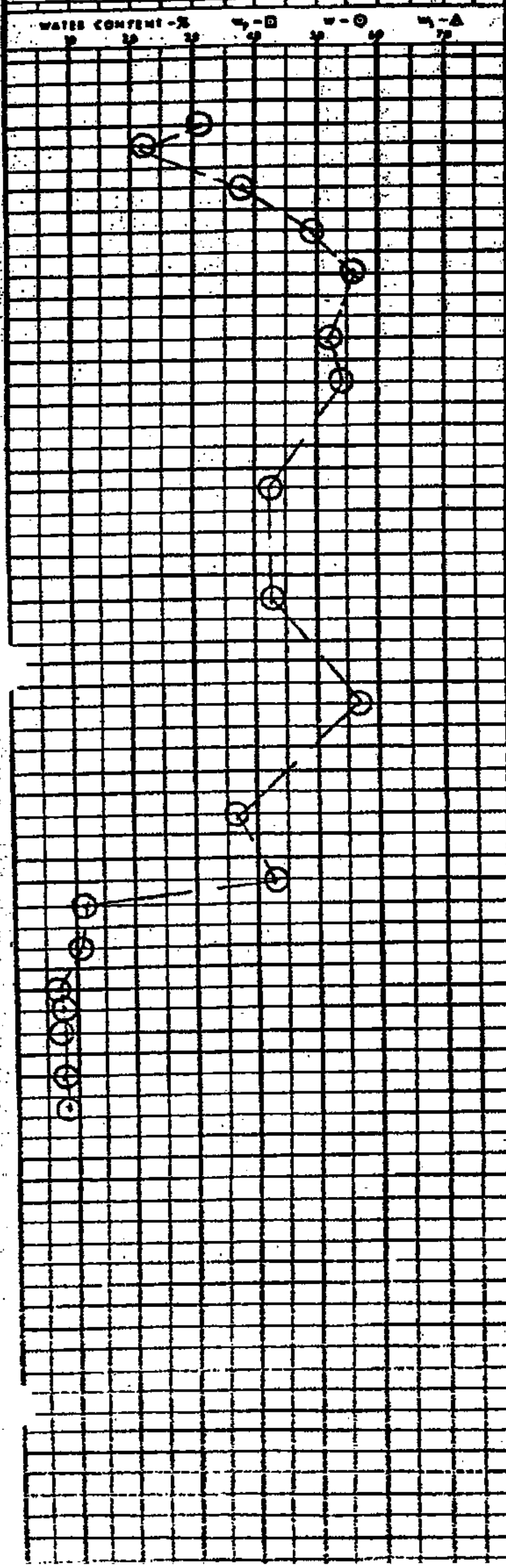
[redacted]

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT [redacted]

W.M.	CKD	DATE OF INVESTIGATION	JOB NO.	HOLE
[redacted]	[redacted]	Apr 8/81	W0854	TH 4

WATER CONTENT



SOIL PROFILE / **SOIL SAMPLES**

DEPTH FEET	SOIL DESCRIPTION (DATUM / SURFACE ELEVATION)	SAMPLE CONDITION / TYPE	OTHER TESTS: 36 inch Auger
2	FILL - asphalt - sand & gravel, clay		
4–6	SILT - grey - clayey, frozen		
6–38	CLAY - stiff - firm below 30 ft. - fissured - brown - grey and till inclusions @ 28 ft. - silty - highly plastic	U (≈11 ft)	qu= 1530 psf pp= 2300 psf Tv= 1700 psf γ_d= 69.7 pcf
		U (≈16 ft)	qu= 1990 psf pp= 3630 psf Tv= 2000 psf γ_d= 70.5 pcf
		U (≈26 ft)	qu= 2385 psf pp= 2650 psf Tv= 1485 psf γ_d= 79.3 pcf
		U (≈36 ft)	qu= 1165 psf pp= 1700 psf Tv= 900 psf γ_d= 87.09 pcf
40–50	SILT TILL - firm - hard @ 43 ft. - slight seepage @ 49 ft - heavy seepage @ 50 ft. - frequent cobbles & boulders - silty		
52	End Hole @ 50 ft.		

NOTES
- test hole terminated @ 50 ft due to heavy water seepage and boulders
- water rose to 43 ft in 2 minutes
- top 10 ft of test hole backfilled with lean concrete

Depth scale (feet): 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68 70

PLATE 5

Site 3

[redacted]		PROJECT #	TEST HOLE NO.
		[redacted]	12-1

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	Grid Lines 7 / E: 1.5m north of pile #389R	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	April 2, 2012
METHOD:	Acker SS: 125 mm SS augers, HQ coring	DRILL DEPTH (m):	17.1

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N)	COMMENTS
0.00	228.44		0 to 0.3m: Crushed Limestone Access Pad		
			0.3 to 7.6m: Clay - silty grey, moist, stiff becoming firm with depth high plasticity		
1.00	227.44				
2.00	226.44				
3.00	225.44				
4.00	224.44				
5.00	223.44				
6.00	222.44				
7.00	221.44				
			7.6 to 9.2m: Clay / till transition zone		
8.00	220.44				
9.00	219.44		9.2 to 15.6m: Glacial Silt Till compact, some sand to sandy, light brown some cobbles, some boulders		
10.00	218.44		10.7m: 750mm diam. boulder		
11.00	217.44		11.3 m: sand in split spoon (SPT)	27	
12.00	216.44		12.5m: 900 mm diam. boulder		
				36	penetration 260 mm bouncing on rock
13.00	215.44				
14.00	214.44		14m: 200 mm diam. cobble dense till around 15 m	29	penetration 200 mm bouncing on rock
15.00	213.44				
			15.6 to 17.1m: Limestone Bedrock RQD = 45% 100mm red clay seam at 16.5m		
16.00	212.44				
17.00	211.44				
			17.1 m END OF TEST HOLE IN LIMESTONE BEDROCK		

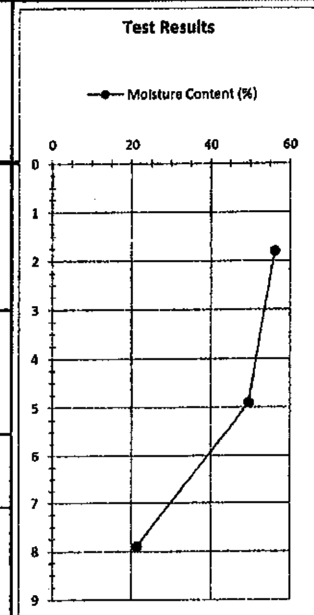
NOTES:

1) 125mm SS augers to 10.7 m
2) HQ casing advancer & coring below 10.7 m
3) bedrock water level 0.4 m above grade
4) test hole backfill with benonite / cement grout mix
   1 bag of bentonite chips at top of test hole

[redacted]		PROJECT # [redacted]	TEST HOLE NO. 12-2

PROJECT:	[redacted]	LOGGED BY:	[redacted]
LOCATION:	Grid Lines 7 / E: 1.8m south of pile #392	REVIEWED BY:	[redacted]
CONTRACTOR:	Paddock Drilling Ltd.	DRILL DATE:	April 3, 2012
METHOD:	Acker SS: 125 mm SS augers, HQ coring	DRILL DEPTH (m):	16.7

DEPTH (m)	ELEVATION (m)	SOIL SYMBOL	SOIL DESCRIPTION	SPT (N)	COMMENTS
0.00	228.44		0 to 0.3m: Crushed Limestone Access Pad		
1.00	227.44		0.3 to 7.6m: Clay - silty grey, moist stiff becoming firm to soft with depth high plasticity		Depth 1.5-2.1 m Qu = 42 kPa TV = 74 kPa PP = 62 kPa γbulk = 16.9 kN/m^3
2.00	226.44				
3.00	225.44				
4.00	224.44				Depth 4.6-5.2 m Qu = 24 kPa TV = 48 kPa PP = 32 kPa γbulk = 17.1 kN/m^3
5.00	223.44				
6.00	222.44				
7.00	221.44				Depth 7.6-8.2 m PP = 18 kPa γbulk = 16.9 kN/m^3
8.00	220.44		7.6 to 10.1m: Clay / till transition zone - soft wet clay with some till inclusions		
9.00	219.44				
10.00	218.44		10.1 to 15.2m Glacial Silt Till compact, some sand to sandy, light brown some cobbles, some boulders		
11.00	217.44			5	sand at top of SPT sample, compact till at bottom of sample
12.00	216.44		10.7 to 12.4m 150mm diam. granite cobble 200mm diam. limestone cobble	22	no sample recovery, likely wet sand
13.00	215.44				
14.00	214.44		14 to 14.9m: 900 mm diam. limestone boulder		
15.00	213.44		very dense till recovered from core barrel at 15.2m		
16.00	212.44		15.2 to 16.7m Limestone Bedrock RQD = 50% no recovery from 16.4 to 16.7 m.		
17.00	211.44		16.7m END OF TEST HOLE IN LIMESTONE BEDROCK		



NOTES:

1) 125mm SS augers to 10.7 m
2) HQ casing advancer & coring below 10.7 m
3) bedrock water level 0.6 m above grade
4) test hole backfill with benonite / cement grout mix
   1 bag of bentonite chips at top of test hole

Site 4

[redacted] [redacted]	Logged/Dwn.: [redacted] Checked: [redacted]	Test Hole No. 1	Project No. [redacted]
PROJECT: [redacted] CLIENT: [redacted]		DATE OF INVEST : MAY 6, 2009 DRILL : SUBTERRANEAN 610 mm AUGER	

SAMPLE NO.	DEPTH (M)	ELEV. (M)	SYM	SOIL DESCRIPTION	
	0.00	233.32		0.00-0.50 TOPSOIL OVER FILL	
	0.75	232.57		0.50-1.25 CLAY	
	1.50	231.82		1.25-2.74 SILT TAN, DAMP,	
	2.25	231.07			
	3.00	230.32		2.74-17.23 CLAY BROWN, STIFF, SILTY, HIGH-PLASTIC TRACE SULPHATES, FINE LAYERINGS SILT AND CLAY ONE TAN LAYER SILTY SANDY WITH FINE GRAVEL	Qu-42.2 kPa Pp-77.6 kPa Tv-74.4 kPa W -17.64 KN/M
	3.75	229.57			
	4.50	228.82			
	5.25	228.07			
	6.00	227.32			
	6.75	226.57			Qu-31.5 kPa Pp-71.8 kPa Tv-62.2 kPa W -16.88 KN/M
	7.50	225.82			
	8.25	225.07		GREY	
	9.00	224.32		TRACE SILT, SAND AND FINE GRAVEL INCLUSIONS, STIFF,	Qu-63.8 kPa Pp-62.5 kPa Tv-63.2 kPa W -17.14 KN/M
	9.75	223.57			
	10.50	222.82			
	11.25	222.07			
	12.00	221.32			
	12.75	220.57			Qu-56.3 kPa Pp-52.6 kPa Tv-52.6 kPa W -16.57 KN/M
	13.50	219.82			
	14.25	219.07			
	15.00	218.32			
	15.75	217.57			
	16.50	216.82			
	17.25	216.07			
	18.00	215.32		END OF TEST HOLE AT 17.23 IN GLACIAL SILT TILL AT AUGER REFUSAL NOTE: VERY LITTLE DEPTH OF SILT TILL.	

MOISTURE CONTENT (%)

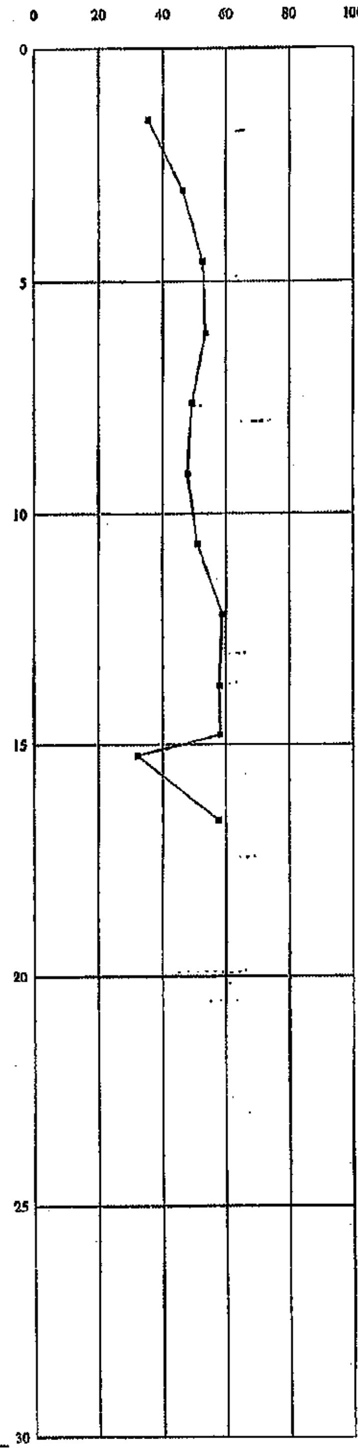


FIGURE 2

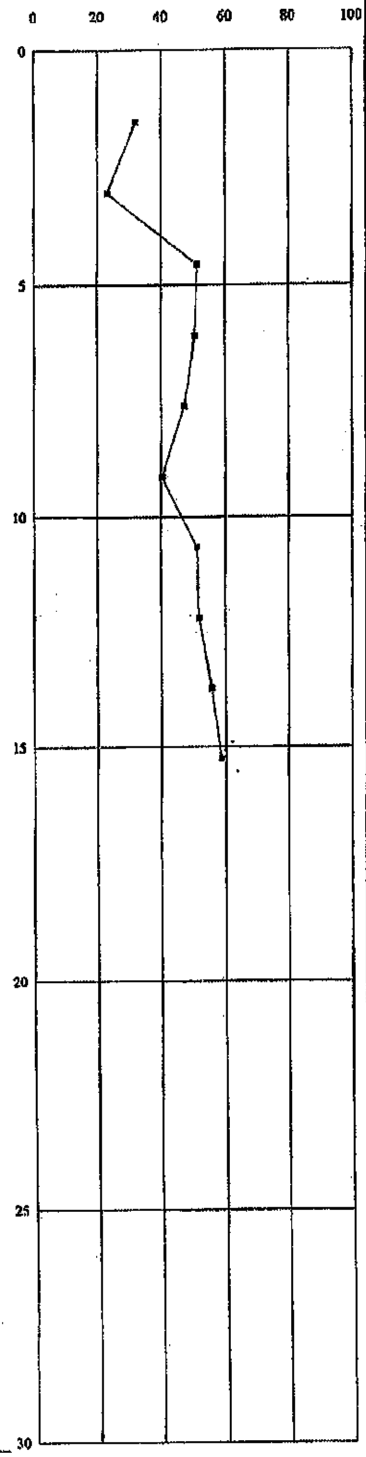
[redacted]	Logged/Dwn.: [redacted] Checked: [redacted]	Test Hole No. 2	Project No. [redacted]
PROJECT: [redacted] CLIENT: [redacted]		DATE OF INVEST : MAY 8,9,11,12/-2009 DRILL : SUBTERRANEAN 812 & 715 mm AUGERS	

SAMPLE NO.	DEPTH (M)	ELEV. (M)	SYM	SOIL DESCRIPTION	
	0.00	233.19			
	0.75	232.44		0.00-0.76 TOPSOIL OVER CLAY FILL FILL WAS A CLAY MATRIX WITH SOME GRAVEL	
	1.50	231.69		0.76-2.13 CLAY BROWN, STIFF, SILTY, HIGH PLASTIC	
	2.25	230.94		2.13-3.05 SILT TAN, DAMP TO DRY	
	3.00	230.19			
	3.75	229.44		3.05-16.77 CLAY BROWN, STIFF, SILTY, HIGH PLASTIC	
	4.50	228.69			
	5.25	227.94			
	6.00	227.19			
	6.75	226.44			
	7.50	225.69			
	8.25	224.94		BROWN TO GREY, STIFF, SILTY HIGH PKASTIC, TRACE SILT, SAND AND FINE GRAVEL INCLUSIONS	Qu-68.8 kPa Pp-105.3 kPa Tv-86.2 kPa W -17.63 KN/M
	9.00	224.19		SILT LAYER, DENSE, DARK BROWN	
	9.75	223.44			
	10.50	222.69			
	11.25	221.94		SAME, SMALL TILL POCKETS	Qu-51.1 kPa Pp-64.6 kPa Tv-53.6 kPa W.-18.85 KN/M
	12.00	221.19			
	12.75	220.44			
	13.50	219.69			
	14.25	218.94		GREY, STIFF, HIGH PLASTIC, SILTY	Qu-63.2 kPa Pp-55.0 kPa Tv-45.9 kPa W -16.66 KN/M
	15.00	218.19			
	15.75	217.44			
	16.50	216.69			
	17.25	215.94		16.77-17.00 GLACIAL SILT TILL WET, SANDY, GRAVELLY	
	18.00	215.19		17.00-25.45 BEDROCK LIMESTONE	
	18.75	214.44			
	19.50	213.69		AT 19.86 +/- 25 TO 50 mm CLAY SEAM SOME VERTICAL FRACTURES AT 19.7	
	20.25	212.94		RED CLAY SEAM AT 20.44, +/- 20mm THICK AT 20.53 A 15 mm THIN RED CLAY SEAM	
	21.00	212.19		THE ROCK UNLEVEL LAYERING WITH	
	21.75	211.44		MASSIVE ROCK, RED TO YELLOW TAN	
	22.50	210.69			
	23.25	209.94		FROM 22.7 TO 25.45 MASSIVE ROCK WITH WITH FINE CLAY SEAM AT 23.3 & 23.9 (+/- 6 mm)	
	24.00	209.19			
	24.75	208.44		FROM 24.3 TO 25.53 UNABLE TO PULL CORES THIS ROCK WAS CHOPPED AND SOUPED OUT IT APPEARED MASSIVE WITH NO VISIBLE CLAY SEAMS	
	25.50	207.69			
	26.25	206.94		END OF TEST HOLE AT 25.45 IN LIMESTONE BEDROCK	
	27.00	206.19			

NOTE: AT 16.77 WATER ROSE INTO TEST HOLE TO 5.8 BELOW SURFACE IN 5 TO 7 MINUTES, AND TO 5.5 BELOW SURFACE IN 20 MINUTES.

STATIC WATER LEVEL WAS AT ELEVATION 227.9 (MEASURED MAY 9, MAY 11 AND MAY 12 PRIOR TO DRILLING)

MOISTURE CONTENT (%)



NOTE: USING 250 GPM PUMP, THE WATER COULD ONLY BE PUMPED DOWN TO 13.1 BELOW GRADE.

FIGURE 3

[redacted]

BOREHOLE LOG

PROJECT [redacted]

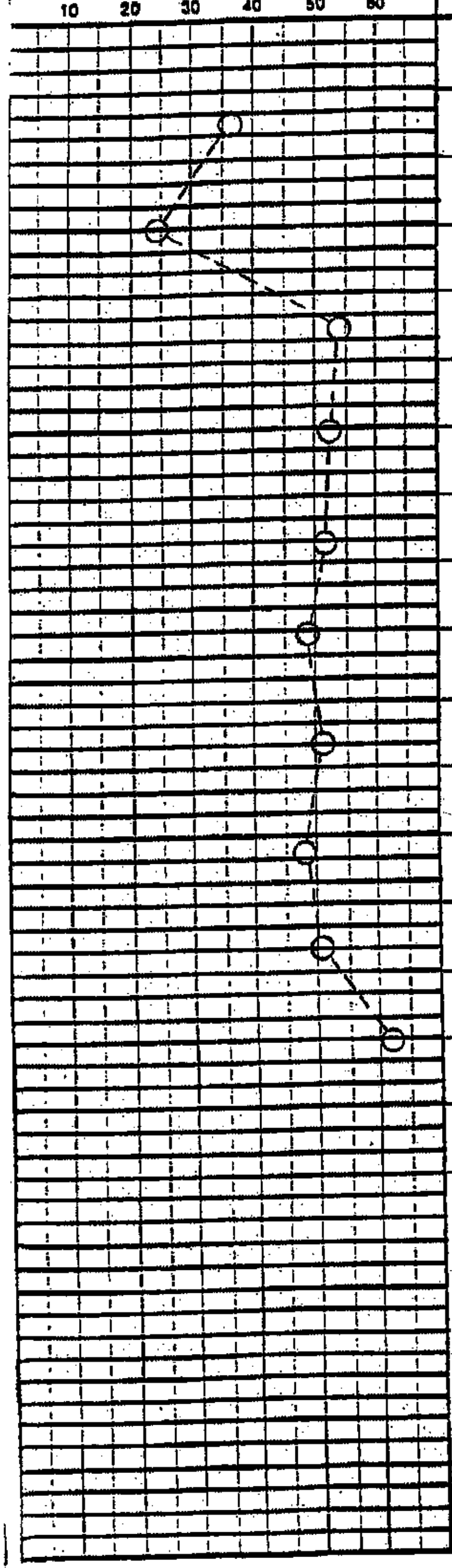
LOGGED/DWN. SDG	CKD. AOD	DATE OF INVEST. 6/04/90	JOB NO. 89819	HOLE NO. 3

WATER CONTENT

Wp - □ W - ○ W_L - △

PERCENT %

10 20 30 40 50 60



DEPTH (M)	SOIL SYMBOL	SOIL DESCRIPTION (DATUM / SURFACE ELEVATION)	SOIL SAMPLE: CONDITION	TYPE	PENETRATION RESISTANCE	DRILL TYPE: 400 mm Auger / OTHER TESTS
0–1		Fill -clay, silt -building rubble				
1–2		Clay -silty -brown				
2–3		Silt -tan -moist to wet				
3–15		Clay -mottled brown -highly plastic -stiff to firm				
~4.8				U		qu=89.0kpa γ_W=16.7kn/m^3 pp=172.4kpa Tv=90.5kpa
~8				U		qu=95.5kpa γ_W=17.1kn/m^3 pp=153.2kpa Tv=81.9kpa
~8.5		---grey				
~11				U		qu=133.9kpa γ_W=16.7kn/m^3 pp=177.1kpa Tv=80.9kpa
~14				U		qu=79.9kpa γ_W=16.2kn/m^3 pp=79.0kpa Tv=55.1kpa
15–16.8		Glacial Till -silty, sandy, gravelly, some clay seams and sand seams -tan, loose				

Notes:
1. Auger refusal at 16.8 m.
2. Water level at 11.9 m from grade upon completion of drilling.

Plate 5

Site 6

[redacted]

TESTHOLE TH3

Project Name: [redacted]
Client: [redacted]
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger and 50 mm Core Bit

Date Drilled: July 15, 2010
Depth of Testhole: 13.0 m
Testhole Elevation: 234.13 m
Logged By: [redacted]

[redacted]

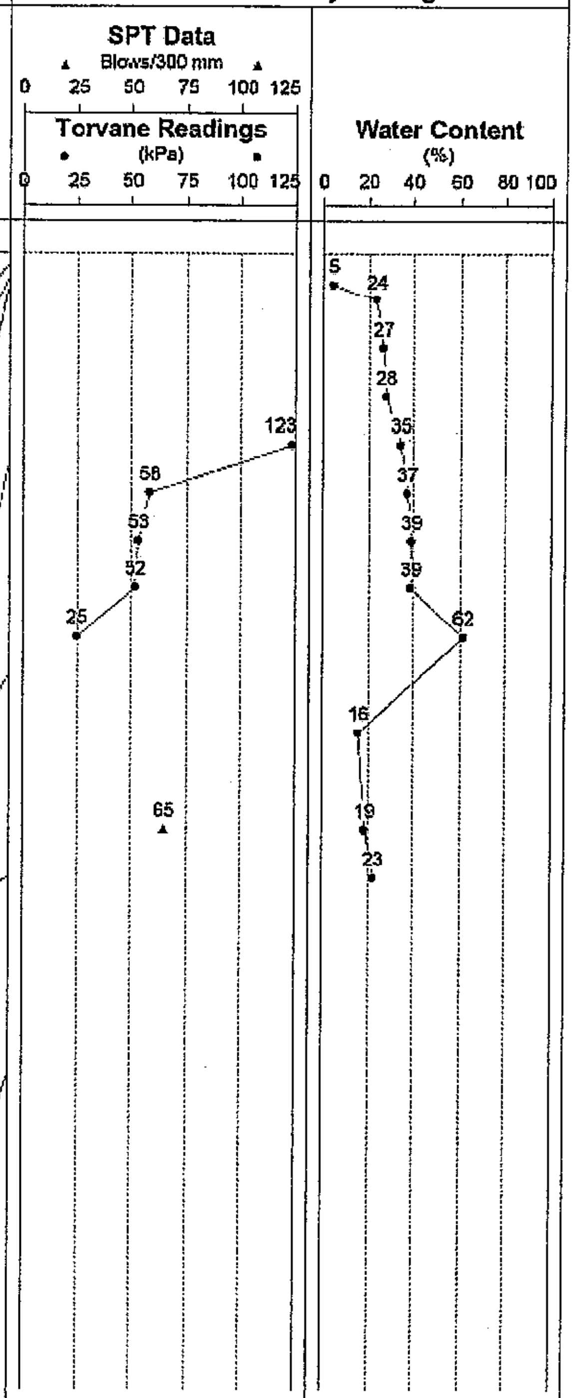
Subsurface Profile

Elev. (m)	Description
234.13	Ground Surface
	Asphalt
	Concrete
	Granular Fill - 20 mm maximum aggregate size
230.78	**Clay Fill** - black, stiff to very stiff, moist, high plasticity - trace fine to coarse sand - trace fine gravel below 2.4 m - trace organic material
	Clay - brown, firm, moist, high plasticity - grey below 3.8 m - brown, soft below 5.6 m
227.42	**Silt Till** - tan, loose, moist to saturated, low plasticity - some coarse sand to fine gravel - red below 7.8 m - trace coarse gravel below 9.1 m
224.22	**Carbonate Bedrock** - mottled, white and red - fracture zones to 12.4 m - 59% RQD (fair quality) from 9.9 m to 11.5 m - 83% RQD (good quality) from 11.5 m to 13.0 m - close to moderately close discontinuities
221.13	

Depth (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18

- Heavy groundwater seepage was observed at a depth of 6.7 m from the silt till layer.
- Groundwater level at 4.7 m upon completion of drilling.
- Soil sloughing was observed below a depth of 7.3 m.
- Auger refusal at 9.9 m on bedrock.

Field / Laboratory Testing



TESTHOLE TH4

[redacted]

Project Name: [redacted]
Client: [redacted]
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 15, 2010
Depth of Testhole: 10.8 m
Testhole Elevation: 234.22 m
Logged By: [redacted]

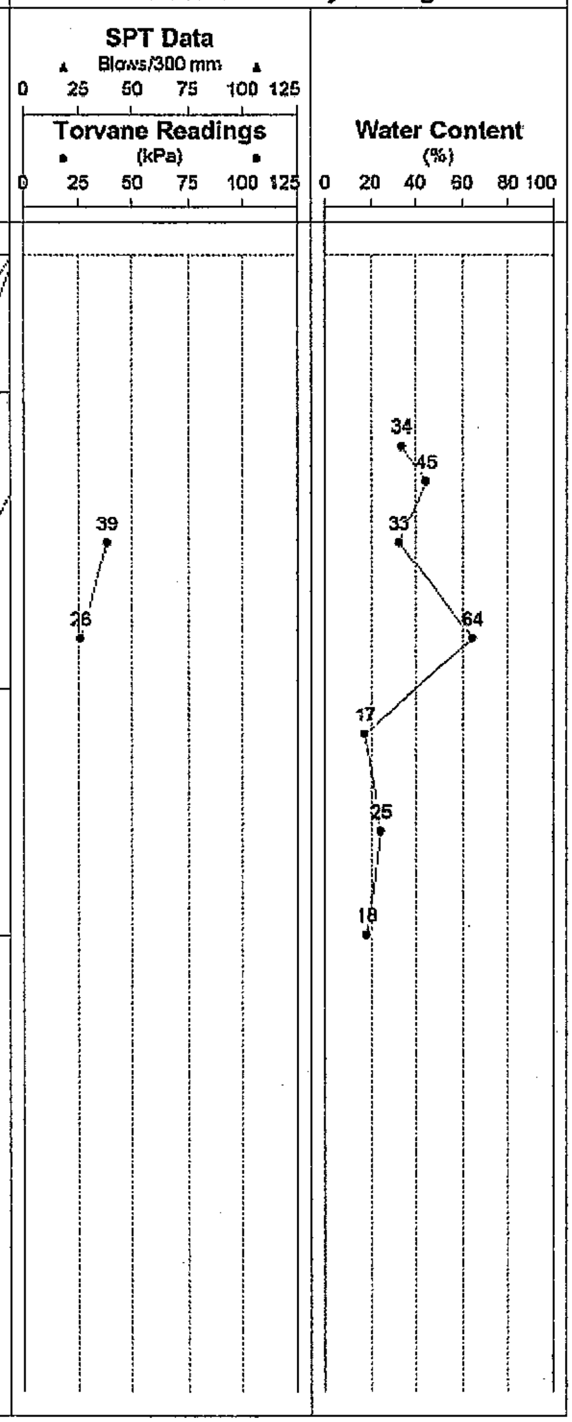
Subsurface Profile

Elev. (m)	Depth (m)	Symbol	Description
234.22	0		Ground Surface
			Asphalt
			Concrete Bridge Deck
232.02			**Clay Fill** - black, stiff to very stiff, moist, high plasticity - trace fine to coarse sand - trace organic material
230.42			**Clay** - brown, firm, moist, high plasticity
227.32			**Silt Till** - tan, loose, moist to saturated, low plasticity - some coarse sand to fine gravel - trace coarse gravel below 7.8 m - red below 7.9 m
223.42			

[redacted]

• Testhole drilled from surface of existing bridge deck.
• Heavy groundwater seepage was observed from the silt till layer.
• Groundwater level at 4.9 m upon completion of drilling.
• Soil sloughing was observed below a depth of 7.9 m.
• Auger refusal at 10.8 m on bedrock.

Field / Laboratory Testing



TESTHOLE TH5

[redacted]

Project Name: [redacted]
Client: [redacted]
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger and 25 mm Core Bit

Date Drilled: July 15, 2010
Depth of Testhole: 12.6 m
Testhole Elevation: 234.20 m
Logged By: [redacted]

Subsurface Profile

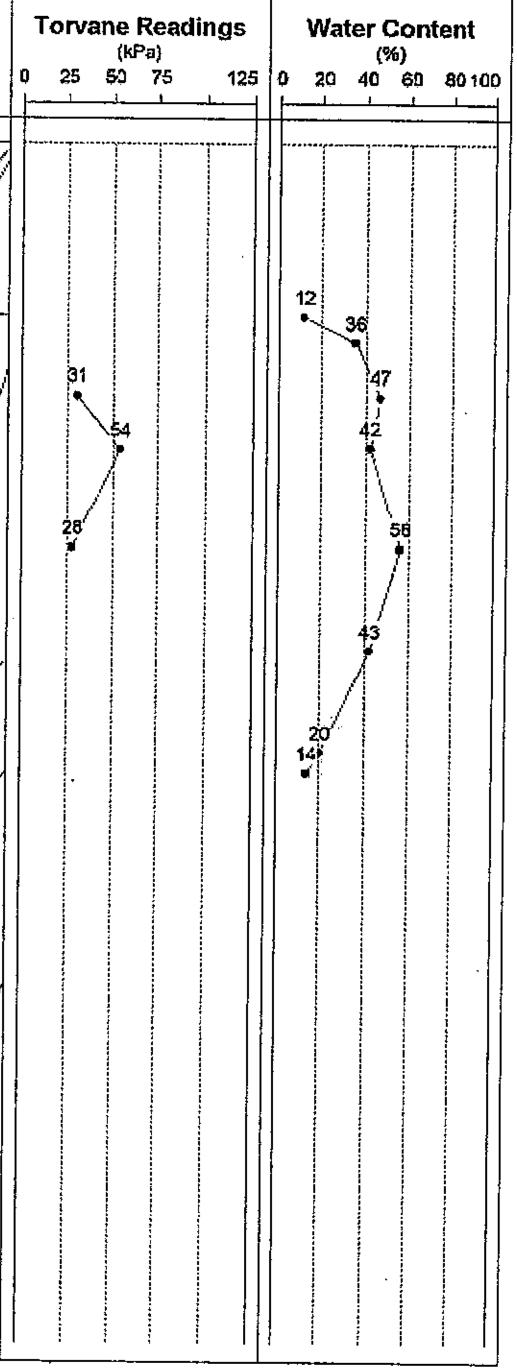
Elev. (m)	Description
234.20	Ground Surface
	Asphalt
	Concrete Bridge Deck
231.60	**Clay Fill** - black, stiff to very stiff, moist, high plasticity - trace fine to coarse sand - trace organic material
230.80	**Clay** - black, firm, moist, high plasticity - brown below 4.7 m - grey below 5.0 m - trace coarse sand to fine gravel below 5.5 m - soft below 6.7 m - some silt till inclusions below 7.6 m
226.40	**Silt Till** - tan, loose, moist to saturated, low plasticity - some coarse sand to fine gravel - red below 8.0 m
224.75	**Carbonate Bedrock** - mottle, white and red - horizontal fracture in filled with clay at 12.0 m - 45% RQD (poor quality) from 9.5 m to 11.0 m - 93% RQD (excellent quality) from 11.0 m to 12.6 m
221.60	

Depth (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18

[redacted]

- Testhole drilled from surface of existing bridge.
- Heavy groundwater seepage was observed from the silt till.
- Groundwater level at 5.5 m upon completion of drilling.
- Soil sloughing was observed below a depth of 7.3 m.
- Auger refusal at 9.5 m on bedrock.

Field / Laboratory Testing



TESTHOLE TH6

[redacted]

Project Name [redacted]
Client: [redacted]
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 15, 2010
Depth of Testhole: 10.1 m
Testhole Elevation: 234.12
Logged By: [redacted]

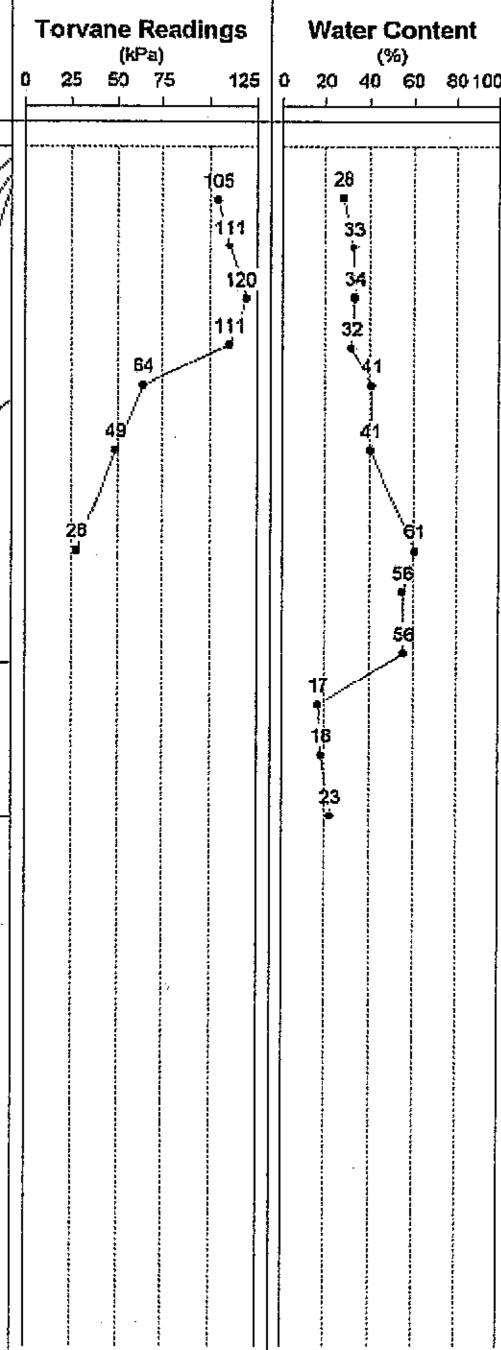
Subsurface Profile

Elev. (m)	Depth (m)	Description
234.12	0	Ground Surface
		Asphalt
		Concrete
		Granular Fill - 20 mm maximum aggregate size
		Clay Fill - black, stiff to very stiff, moist, high plasticity - trace fine to coarse sand - trace organic material
230.32		**Clay** - brown, firm, moist, high plasticity - grey below 5.2 m - trace coarse gravel below 7.7 m
226.35		**Silt Till** - tan, loose, moist to saturated, low plasticity - some coarse sand to fine gravel - red below 8.4 m
224.02	10	

[redacted]

- Heavy groundwater seepage was observed at a depth of 7.8 m from the silt till layer.
- Groundwater level was observed at 4.6 m upon completion of drilling.
- Soil sloughing was observed below a depth of 8.1 m.
- Auger refusal at 10.1 m on bedrock.

Field / Laboratory Testing



Site 7

[redacted]	Logged/Dwn.: [redacted] Checked: [redacted]	Test Hole No. 3	Project No. [redacted]
PROJECT: [redacted] CLIENT: [redacted]		DATE OF INVEST : MARCH 10 - 12, 2009 DRILL : SUBTERRANEAN	

SAMPLE NO.	DEPTH (M)	ELEV. (M)	SYM	SOIL DESCRIPTION
				TEST HOLE LOCATED OFF HENDERSON ON SOUTH SIDE OF MIDWINTER AVE. WEST OF BRIDGE
	0.00	229.40		
	1.25	228.15		0.00-0.15 TOPSOIL 0.15-0.91 CLAY 0.91-2.13 SILT
	2.50	226.90		
	3.75	225.65		2.13-11.43 CLAY DARK BROWN, STIFF, SILTY, HIGH PLASTIC
[redacted]	5.00	224.40		[redacted]
	6.25	223.15		
	7.50	221.90		
	8.75	220.65		
	10.00	219.40		
	11.25	218.15		
	12.50	216.90		11.43-17.20 SANDY, GRAVELLY, SILTY SAND AND GRAVEL MATRIX WITH SOME COBBLES LIMESTONE AND GRANITIC GRAVEL AND COBBLES
	13.75	215.65		
	15.00	214.40		
	16.25	213.15		
	17.50	211.90		17.20-24.60 GLACIAL SILT TILL SILT MATRIX, WITH SAND AND GRAVEL, SOME COBBLES, DENSE TO VERY DENSE
	18.75	210.65		18.3 TO 18.9 SOFTER DRILLING SANDY LAYERS, (WASHED OUT DURING CORING)
	20.00	209.40		
	21.25	208.15		
[redacted]	22.50	206.90		
	23.75	205.65		
[redacted]	25.00	204.40		24.60-37.35 SANDY, GRAVELLY, AND AND GRAVEL MATRIX WITH COBBLES AND SOME BOULDERS, LIMESTONE AND GRANITIC APPEARED DENSE FROM THE CORING WITH ONLY COARSE GRAVEL IN CORE BARREL
	26.25	203.15		
	27.50	201.90		
	28.75	200.65		
	30.00	199.40		29.9 TO 34.6 FINE SAND ODD COBBLE AND GRAVEL STONES
	31.25	198.15		
	32.50	196.90		
[redacted]	33.75	195.65		
	35.00	194.40		34.6 TO 35.3 LAYER SHALE (0.7m) FINE SAND
	36.25	193.15		36.6 TO 37.0 LAYER SHALE (0.4m) 37.0 TO 37.35 GLACIAL SILT TILL
[redacted]	37.50	191.90		37.35-43.45 BEDROCK UPPER 1.50 FEW FRACTURES
	38.75	190.65		
	40.00	189.40		LOWER 4.57 SOUND, VERY FEW FRACTURES
	41.25	188.15		
	42.50	186.90		
	43.75	185.65		
	45.00	184.40		END OF TEST HOLE AT 43.45 IN LIMESTONE BEDROCK

RQD (%)	DEPTH (M)
88	37.35-38.87
89	38.87-40.40
100	40.40-41.92
94	41.92-43.45

MOISTURE CONTENT (%)

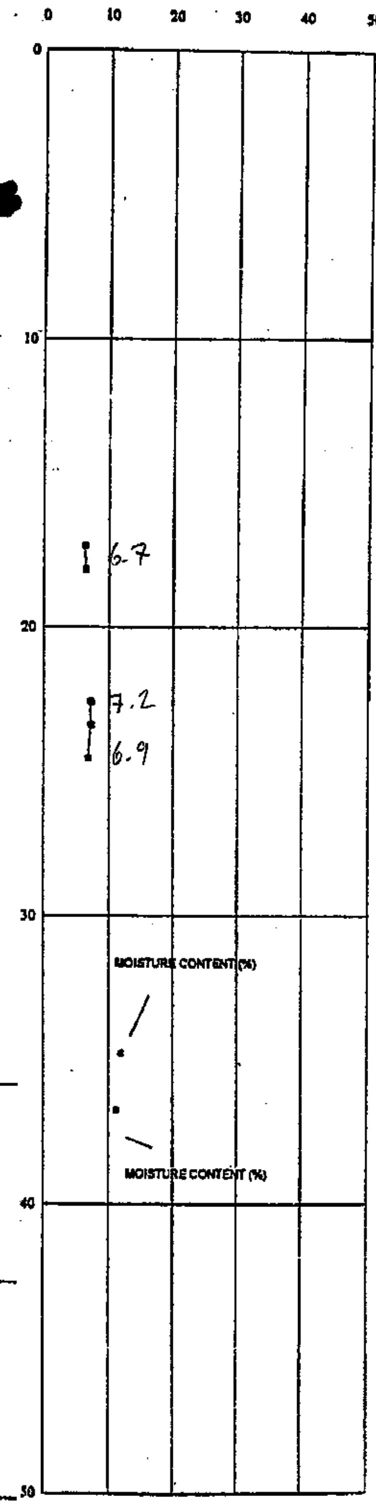
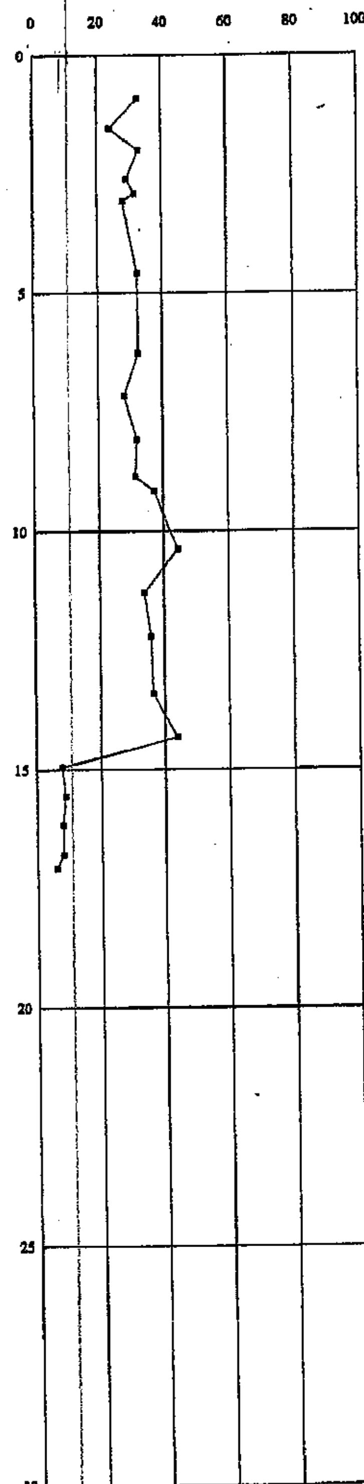


FIGURE 4

[redacted]	Logged/Dwn.: [redacted] Checked: [redacted]	Test Hole No. 5	Project No. [redacted]
PROJECT: [redacted] CLIENT: [redacted]		DATE OF INVEST : APRIL 1, 2009 DRILL : SUBTERRANEAN, 610 & 710 mm AUGER	

SAMPLE NO.	DEPTH (M)	ELEV. (M)	SYM	SOIL DESCRIPTION	
				LOCATION GT4 (NORTH OF GLADSTONE BTWN EXISTING PIERS 2 & 3)	
	0.00	229.19		0.00-0.15 TOPSOIL	
	0.75	228.44		0.15-1.22 FILL CLAY MATRIX WITH TRACE METAL, WOOD,	
	1.50	227.69		1.22-14.48 ALLUVIAL CLAYS, SILTS AND SANDS BROWN, SILTY, CLAYEY, FINE SAND	
	2.25	226.94		LAYERED, STIFF, MEDIUM PLASTIC	
	3.00	226.19		FINE SAND, CLEAN,	
	3.75	225.44			
	4.50	224.69			
	5.25	223.94			
	6.00	223.19		TEST HOLE SLOUGHING FROM WET SILT AND SAND, A 710 mm x 8.64 M STEEL SLEEVE WAS USED TO CONSTRUCT THE TEST HOLE	
	6.75	222.44			
	7.50	221.69		THE SILTY SAND WAS STAINED	
	8.25	220.94		GREY, MOIST, TRACE BLACK INCLUSIONS, SAND MEDIUM TO FINE, LAYERING, TRACE FINE ROOTLETS WET TO PAT TEST,	Pp-67.0 kPa Tv-23.94 kPa
	9.00	220.19			
	9.75	219.44			
	10.50	218.69			
	11.25	217.94			
	12.00	217.19		CLAYEY, SILTY, STIFF, MEDIUM PLASTIC, DARK 'ORGANIC' INCLUSIONS	Qu-46.5 kPa Pp-79.0 kPa Tv-68.0 kPa W -18.66 KN/M
	12.75	216.44			
	13.50	215.69			
	14.25	214.94			
	15.00	214.19		14.48-17.38 GLACIAL SILT TILL SILTMATRIX WITH SAND AND GRAVEL DENSE, COBBLES AND BOULDERS, DENSE TO VERY DENSE AT 16.8	
	15.75	213.44			
	16.50	212.69			
	17.25	211.94			
	18.00	211.19			
	18.75	210.44		END OF TEST HOLE AT 17.38 IN VERY DENSE GLACIAL SILT TILL	

MOISTURE CONTENT (%)



Site 8

[redacted] **SUMMARY LOG**

REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	230.85
SITE	[redacted]	TOP OF PVC ELEV.	231.92
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	15 Jan 09

Column headings: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25, 50, 75) | Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20, 40, 60, 80) / PL — MC — LL % (20, 40, 60, 80)

Elevation (m)	Depth (m)	Description and Classification
230.85	0.0	CLAY (Fill) - silty, trace sand, trace gravel, trace organics - black, dark grey - stiff to very stiff, moist - medium to high plasticity
229.64		CLAY (Glaciolacustrine) - silty, trace sand, trace gravel - brown - stiff, moist - high plasticity - dark grey below 4.9 m - firm below 6.7 m
223.84	7	SILT (Glacial Till) - trace clay, trace sand, trace gravel, trace cobbles - light brown - compact becoming dense with depth, moist - low to nonplastic

Piezometer log depth: 0.3

Sample Number	Recovery %	SPT (N)
G1		
G2		
T3		
G4		
T5		
G6		
S7	58	▲34 bouncing at 0.3 m
S8	100	▲50 blows over 89mm
C9	26	
S10	72	▲36
C11	16	

Elevation scale: 230, 229.64, 229, 228, 227, 226, 225, 224, 223.84, 223, 222, 221
Depth scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9; (ft): 5, 10, 15, 20, 25, 30

SAMPLE TYPE: Auger Cuttings | Shelby Tube | Split Spoon | Core Barrel

CONTRACTOR Paddock Drilling Ltd. | INSPECTOR [redacted] | APPROVED [redacted] | DATE 16/1/09

[redacted] **SUMMARY LOG**

REFERENCE NO.

HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75)	PL MC LL % (20 40 60 80) / Cu Uncon. (kPa) ◇, Cu Lab Vane. (kPa) □, Cu TORVANE (kPa) ◆, Cu Pocket Pen (kPa) ★ (20 40 60 80)
220	11 (35)						S12	28	▲36	●
							C13	31		
219 / 218.85	12 (40)		LIMESTONE (Bedrock) - mottled yellow and white - R3 medium strong - close spacing - class 2 aperture - class 1 filling - RQD = 16% between 12.2 and 13.7 m				S14	36	▲75 - Blows over 200 mm	●
218	13						C15	17		
							C16			
217	14 (45)		- RQD = 100% between 13.7 and 15.2 m		14.3		C17	100		
216	15				14.9					
215.61	(50)				15.2					

END OF TEST HOLE AT 15.2 m IN BEDROCK

Notes:

1. Power auger refusal at 8.5 m, coring below 8.5m.
2. No sloughing.
3. No seepage.
4. Installed standpipe at 15.2 m complete with Casagrande tip and above ground cover. Sand backfill up to 14.3 m, bentonite grout to surface.

June 15, 2006: Water level in piezometer = 221.05 m

Elevation scale: 215, 214, 213, 212, 211, 210; Depth scale (m): 16, 17, 18, 19, 20, 21; (ft): 55, 60, 65, 70

SAMPLE TYPE: Auger Cuttings; Shelby Tube; Split Spoon; Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted]

DATE 16/1/09

[redacted]UMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	230.60
SITE	[redacted]	TOP OF PVC ELEV.	231.58
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	15 Jan 09

Column headings: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ (25 50 75) | PL MC LL % (20 40 60 80)

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

Elevation (m)	Depth (m)	Description and Classification	Piez. Depth (m)	Sample	Recovery %	Notes
230.45		CLAY - trace organics, trace rootlets, black, dry to moist	0.3			
		CLAY (Fill) - silty, trace sand, trace gravel - varies from black to dark grey to brown below 1.8 m - stiff, dry to moist - high plasticity - trace silt inclusions		G73		
				G74		
				T75		177
		- firm to stiff below 3.7 m		G76		
226.03		CLAY (Glaciolacustrine) - silty - brown - firm to stiff, moist - high plasticity - trace silt inclusion (<5 mm dia.)		G77		
				T78		
		- clay and silt to till transition below 6.7 m - trace sand, brown, soft to firm, moist		G79		
222.08		SILT (Glacial Till) - trace clay, trace sand, trace gravel, trace cobbles - light brown - compact becoming dense with depth, moist - low to nonplastic		G80		
				S81	33	33 rock pieces in spoon
				C82	17	

SAMPLE TYPE: Auger Cuttings | Shelby Tube | Split Spoon | Core Barrel

CONTRACTOR	Paddock Drilling Ltd.	INSPECTOR	[redacted]
APPROVED	[redacted]	DATE	16/1/09

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	DEPTH (m)	NUMBER	RECOVERY %
220	11	35			C83	69
219.02			- 25 mm silt and sand seam at 11.4 m		C84	
219	12	40	LIMESTONE (Bedrock) - mottled orange and white to yellow and white - R3 medium strong - moderately close spacing - class 2 aperture - class 1 filling - RQD = 13% between 12.2 and 13.7 m		C85	
218	13				C86	18
217	14	45	- RQD = 30% between 13.7 and 15.2 m		C87	72
216, 215	15, 16	50	- RQD = 66% between 15.2 and 16.8 m		C88	100
214, 213	17, 18	55	- RQD = 91% between 16.8 and 18.3 m	17.4 18.0 18.3	C60	100
212.31		60	END OF TEST HOLE AT 18.3 m IN BEDROCK			

Notes:
1. Near power auger refusal at 9.1 m, coring below 9.1 m.
2. No sloughing.
3. No seepage.
4. Installed standpipe at 18.3 m, complete with Casagrande tip and above ground cover.
5. Backfilled with sand up to 17.4 m, bentonite grout to surface, and capped with bentonite chips.

June 15, 2008: Groundwater level in piezometer = 221.57 m

Elevation scale: 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209
Depth scale (m): 11–21; (ft): 35, 40, 45, 50, 55, 60, 65, 70

Column headings: GRAPHICS; PIEZ. LOG; SAMPLE TYPE; SPT (N) blows/0.30 m ▲; CONE blows/0.15 m △ (25, 50, 75); Cu Uncon. (kPa) ◇; Cu Lab Vane. (kPa) □; Cu TORVANE (kPa) ◆; Cu Pocket Pen (kPa) ★ (20, 40, 60, 80); PL, MC, LL % (20, 40, 60, 80)

SAMPLE TYPE: Auger Cuttings; Shelby Tube; Split Spoon; Core Barrel

CONTRACTOR Paddock Drilling Ltd. INSPECTOR [redacted] APPROVED [redacted] DATE 16/1/09

Site 9

[redacted] **SUMMARY LOG**

REFERENCE NO.

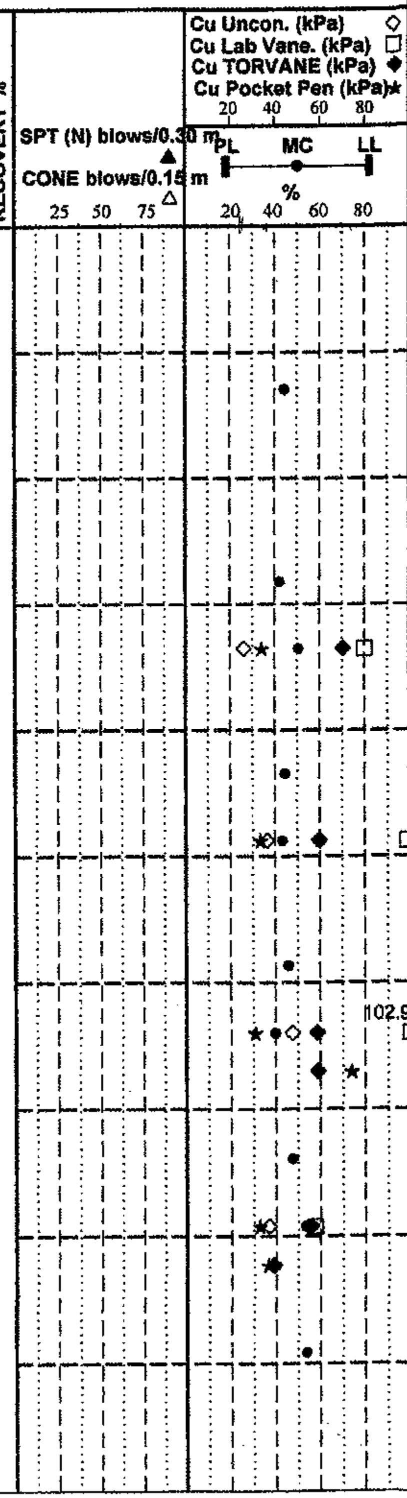
HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	235.58
SITE	[redacted]	TOP OF PVC ELEV.	236.51
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø SS Auger, HQ Coring, Mobile S61	DATE DRILLED	Jun 8, 06

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %
	0		CLAY - silty, trace sand, trace gravel - varies from black to brown - firm, moist - medium to high plasticity		0.3			
						Auger Cuttings	G23	
233.75			CLAY (Glaciolacustrine) - silty - brown - stiff becoming firm with depth, moist - high plasticity - trace silt inclusions (<3 mm dia.)					
						Auger Cuttings	G24	
						Shelby Tube	T25	
						Auger Cuttings	G26	
						Shelby Tube	T27	
						Auger Cuttings	G28	
						Shelby Tube	T29	
						Auger Cuttings	G30	
			- grey below 7.6 m			Shelby Tube	T31	
						Auger Cuttings	G32	
226.43			SILT (Glacial Till) - trace clay, trace sand, trace gravel, trace cobbles - light grey - moist, compact becoming dry, dense with depth					

Elevation scale (m): 235, 234, 233, 232, 231, 230, 229, 228, 227, 226

Depth scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9; (ft): 5, 10, 15, 20, 25, 30



SAMPLE TYPE: Auger Cuttings | Shelby Tube | Split Spoon | Core Barrel

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 1/30/07

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	DEPTH (m)	NUMBER	RECOVERY %	SPT (N) blows/0.30 m / CONE blows/0.15 m
			- low to nonplastic		G33		
223	11	35			S34	106	15
222					G35		
	12	40			C36	43	
					S37	90	50 blows over 102 mm
221	13				C38	70	
220	14	45			G40		
					S39		
					C41	54	
220.98					G42		
	15	50	LIMESTONE (Bedrock) - mottled yellow and white - strength classification: R3 medium strong to R4 strong - spacing classification: moderately close spacing - class 2 aperture - class 1 filling - some chemical weathering - RQD = 69% between 15.2 and 16.8 m		C43	50	
218	16				C44	91	
217	17	55	- RQD = 89% between 16.8 and 18.3 m	17.4			
216					C45	93	
	18			18.0			
217.29		60	END OF TEST HOLE AT 18.3 m IN LIMESTONE BEDROCK Notes: 1. Solid stem auger refusal at 11.9 m, HQ coring below 11.9 m. 2. No sloughing. 3. Seepage from till and bedrock. 4. Installed standpipe at 18.3 m, complete with Casagrande tip and above ground cover.	18.3			
215	19						
214	20	65					
213	21						
212		70					

Other column headings: GRAPHICS; PIEZ. LOG; SAMPLE TYPE

SPT scale: 25 50 75

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

PL MC LL
%
20 40 60 80

SAMPLE TYPE: Auger Cuttings | Shelby Tube | Split Spoon | Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted]

DATE 1/30/07

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	231.00
SITE	[redacted]	WATER ELEV.	
LOCATION	[redacted]		
DRILLING METHOD	125 mm ø SS Augers, HQ Coring, ACKER SS Drill Rig	DATE DRILLED	Mar 23, 06

Column headings: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲; CONE blows/0.15 m △ (25 50 75) | Cu Uncon. (kPa) ◇; Cu Lab Vane. (kPa) □; Cu TORVANE (kPa) ◆; Cu Pocket Pen (kPa) ★ (20 40 60 80); PL — MC — LL % (20 40 60 80)

Elevation (m)	Depth (m)	Depth (ft)	Description and Classification	Number	Recovery %	SPT (N)
			CLAY (Glaciolacustrine) - silty - brown - frozen to 0.3 m -moist, stiff, high plasticity below 0.3 m - trace silt inclusions < 5 mm dia.			
230	1					
		5		G108		
229	2					
			- grey below 2.5 m			
228	3	10	- firm to stiff below 3m	T109	100	
227	4					
		15	- trace sand, trace gravel below 4.4 m	T110	83	
226	5					
225	6	20		G111		
224.66			- Clay to Silt till transition below 6.3 m: silt and clay, light grey, soft, low to medium plasticity			
224.20			SILT (Glacial Till) - trace to some sand, trace gravel - light brown - very dense, moist to dry with depth - non plastic	G112		
224	7					
		25		S113	67	▲74
223	8					
				S114	89	▲76
			- some gravel below 8.6 m	C115	6	
222	9	30		S116	47	▲54
			- light grey below 9.7 m	C117	35	
221						

SAMPLE TYPE: Auger Cuttings | Shelby Tube | Split Spoon | Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 1/30/07

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m / CONE blows/0.15 m	Cu / PL MC LL %
219		35		Split Spoon	S118	50	86 blows over 270 mm	
218	11			Core Barrel	C119	18		
217	12	40						
216.41			LIMESTONE (Bedrock) - rubbled bedrock between 12.3 and 13.4m - mottled brown and white - slight weathering - some rubble zones - strength classification: R3 medium strong - spacing classification: moderately close spacing - class 2 aperture - class 1 filling - RQD = 85 % between 13.5 and 15m	Core Barrel	C120	56		
216 / 217.85	13	45						
215	14			Core Barrel	C121	100		
214	15	50	- RQD = 100% between 15 and 16.55m					
213	16			Core Barrel	C122	100		
214.45			END TEST HOLE AT 16.55 m IN LIMESTONE BEDROCK Notes: 1. Solid stem auger refusal at 8.1 m, HQ core below 8.1 m. 2. No sloughing. 3. Seepage from bedrock. Circulation loss below 12.6 m. 4. Backfill with bentonite grout to surface.					
212	17	55						
211	18	60						
210	19							
209	20	65						
208	21	70						

Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80

SAMPLE TYPE: Auger Cuttings, Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 1/30/07

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	227.41
SITE	[redacted]	TOP OF PVC ELEV.	228.30
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø SS Augers, HQ Coring, ACKER SS Drill Rig	DATE DRILLED	Mar 21, 06

Column headings: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲, CONE blows/0.15 m △ (25, 50, 75) | Cu Uncon. (kPa) ◇, Cu Lab Vane. (kPa) □, Cu TORVANE (kPa) ◆, Cu Pocket Pen (kPa) ★ (20, 40, 60, 80); PL, MC, LL % (20, 40, 60, 80)

Elevation (m)	Description and Classification
227.41	CLAY (Glaciolacustrine) - silty - brown - frozen to 0.9m - trace silt inclusions <10 mm dia. - moist, firm, high plasticity below 0.9m - grey below 2.1m
223.80	SILT (Glacial till) - clayey, trace sand, trace gravel, trace cobbles - light grey - moist, loose to very loose - low plasticity - trace clay below 4.4m - some gravel below 5.8m
221.01	LIMESTONE (Bedrock) - mottled yellow and brown - massive - strength classification: R4 strong - spacing classification: moderately close spacing - class 1 aperture - class 1 filling - RQD = 57% between 6.9 and 8.4m - RQD = 82% between 8.4 and 9.9m
217.51	

Piezometer log depths (m): 0.6, 2.9, 4.4, 5.3, 5.6

Number	Recovery %	SPT (N)
G59		
G60		
G61		
S62	53	7
S63	75	3
C64	46	
S65		
C66	76	
C67	88	
C68	100	

SAMPLE TYPE: Auger Cuttings, Split Spoon, Core Barrel

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 1/30/07

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT [redacted]

PROJECT [redacted]

SITE [redacted]

LOCATION [redacted]

DRILLING METHOD 125 mm ø SS Auger, HQ Coring, Mobile S61

JOB NO. [redacted]

GROUND ELEV. 230.50

TOP OF PVC ELEV. 231.46

WATER ELEV.

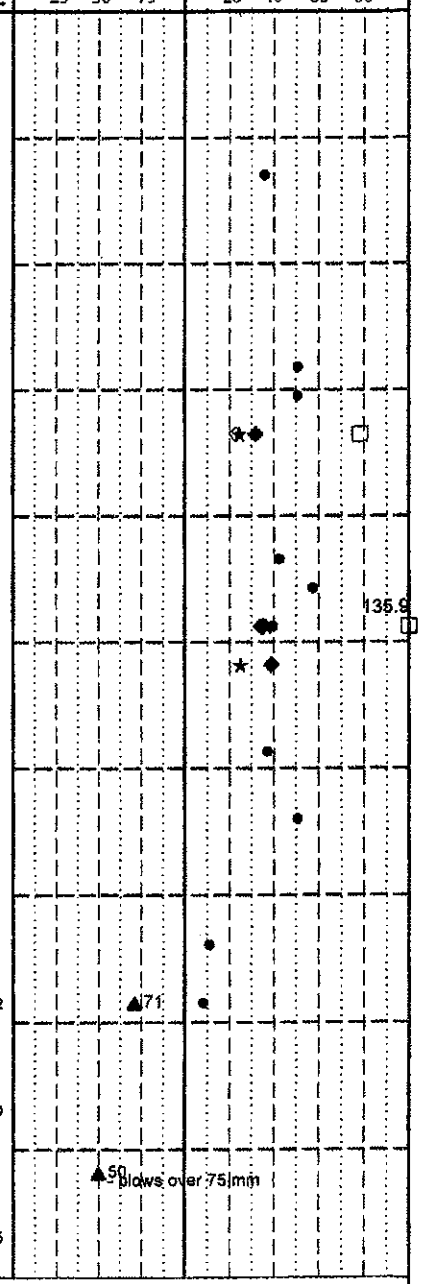
DATE DRILLED Jun 8, 06

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %
230				CLAY - silty, trace sand, trace gravel - varies from grey to brown - stiff, dry to moist - medium to high plasticity		0.3			
229	1	5					Auger Cuttings	G8	
228.67									
228	2			CLAY (Glaciolacustrine) - silty - grey - firm to stiff, moist - high plasticity - trace silt inclusions (<3 mm dia.)			Auger Cuttings	G9	
227	3	10					Shelby Tube	T10	
226	4	15		- firm below 4 m			Auger Cuttings	G11	
	5						Shelby Tube	T12	
225	6	20					Auger Cuttings	G13	
224.10 224				SILT (Glacial Till) - trace clay, trace sand, trace gravel, trace cobbles - light grey - moist, compact becoming dry, dense to very dense with depth - low to nonplastic			Shelby Tube	T14	
223	7	25					Auger Cuttings	G15	
	8						Split Spoon	S16	72
222							Core Barrel	C17	50
221.20	9	30				9.1		S18	0
221				LIMESTONE (Bedrock) - mottled yellow and white - strength clasification: R3 medium strong - spacing classification: close spacing		9.4 9.8	Core Barrel	C19	95

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★

SPT (N) blows/0.30 m ▲
CONE blows/0.15 m △

PL — MC — LL (%)



SAMPLE TYPE: Auger Cuttings, Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted]

DATE 1/30/07

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %
218		35		- class 2 aperture - class 1 filling - solution pitting between 11.9 and 12.5 m - RQD = 63% between 9.1 and 10.7 m - RQD = 60% between 10.7 and 12.2 m		10.1	Core Barrel	C20	78
217	11							C21	83
216	12	40		- RQD = 96% between 12.2 and 13.7 m				C22	97
[illegible]	13	45		END OF TEST HOLE AT 13.7 m IN LIMESTONE BEDROCK Notes: 1. Solid stem auger refusal at 8.2 m, HQ coring below 8.2 m. 2. Installed standpipe at 9.8 m, complete with Casagrande tip and above ground cover. 3. Backfilled with bentonite chips to 10.1 m, sand to 9.1 m, bentonite grout to surface.		13.7			
214	14								
213	15	50							
212	16	55							
211	17								
210	18	60							
209	19	65							
208	20								
207	21	70							

SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75

Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80

PL MC LL % 20 40 60 80

SAMPLE TYPE: Auger Cuttings, Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR Paddock Drilling Ltd. INSPECTOR [redacted] APPROVED [redacted] DATE 1/30/07

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	235.88
SITE	[redacted]	TOP OF PVC ELEV.	236.85
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø SS Augers, HQ Coring, ACKER SS Drill Rig	DATE DRILLED	Mar 22, 06

Columns: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75) | Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80) / PL MC LL % (20 40 60 80)

Elevation (m)	Depth (m)	Description and Classification	Sample No.	Recovery %
	0	CLAY (Glaciolacustrine) - silty, trace sand, trace organics to 1.5 m - brown - frozen to 0.9 m	G69	
235	1	- moist, stiff, high plasticity below 0.9 m - silt laminations < 1mm thick, between 0.9 and 3 m - trace silt inclusions <8 mm dia.	G70	
234	2			
233	3		T71	83
232	4	- grey, trace oxidation below 3.9 m	T72	100
231	5	- trace fine sand inclusions <10 mm dia. between 5.1 and 6.6 m		
230	6	- trace gravel below 6.2 m	T73	
229	7		T74	
228	8	- firm below 8m		
227	9		G75	
226	10			

101.3

SAMPLE TYPE: Auger Cuttings, Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR: Paddock Drilling Ltd.   INSPECTOR: [redacted]   APPROVED: [redacted]   DATE: 1/30/07

[redacted] **SUMMARY LOG** REFERENCE NO. HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION
224			
225.29		35	**SILT (Glacial Till)** - trace clay, trace sand, trace gravel, trace cobbles - light brown - moist to dry with depth, dense to very dense - low plastic - trace to some sand, trace to some gravel, dense below 11.1m
223	11		
222	12	40	- very dense below 12.6 m
221	13		
220	14	45	
219	15	50	- some gravel, light grey below 15 m
218	16		
217	17	55	
216	18	60	
215	19		
216.30		65	**LIMESTONE (Bedrock)** - mottled brown and white - some weathering - strength classification: R4 strong - spacing classification: moderately close spacing - class 2 aperture - class 1 filling - RQD = 76% between 19.6 and 21.1m
214	20		
213	21	70	- RQD = 84% between 21.1 and 22.6m

PIEZ. LOG — DEPTH (m): 21.1

NUMBER	RECOVERY %	SPT (N) blows/0.30 m
G76		
G77		
S78		53
S79		22
C80		
S81	67	
C82	0	
S83	50	78
C84	13	
S85	83	63
C86		
C87	90	
S88	100	100 - blows over 255 mm
C89	75	
S90	63	97 blows over 238 mm
C91	21	
C92	100	

SPT (N) blows/0.30 m ▲ — CONE blows/0.15 m △ — 25 50 75

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ✶
20 40 60 80

PL MC LL
%
20 40 60 80

SAMPLE TYPE: Auger Cuttings, Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR Paddock Drilling Ltd. INSPECTOR [redacted]

APPROVED [redacted] DATE 1/30/07

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75	Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa)★ 20 40 60 80; PL MC LL % 20 40 60 80
212	22					22.3		C93	100		
213.25		75				22.6					
211	23			END TEST HOLE AT 22.6 m IN LIMESTONE BEDROCK Notes: 1. Approaching auger refusal at 12.1 m, HQ core below 12.1 m. 2. No sloughing. 3. Seepage from till. Circulation loss at 19.5 m 4. Install standpipe piezometer at 22.6 m in Limestone bedrock, complete with above ground cover and Casagrande tip. Water level at 225.97 at time of installation.							
210	24	80									
209	25										
208	26	85									
207	27	90									
206	28										
205	29	95									
204	30	100									
203	31										
202	32	105									
201	33	110									

SAMPLE TYPE: Auger Cuttings; Shelby Tube; Split Spoon; Core Barrel

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 1/30/07

Site 10

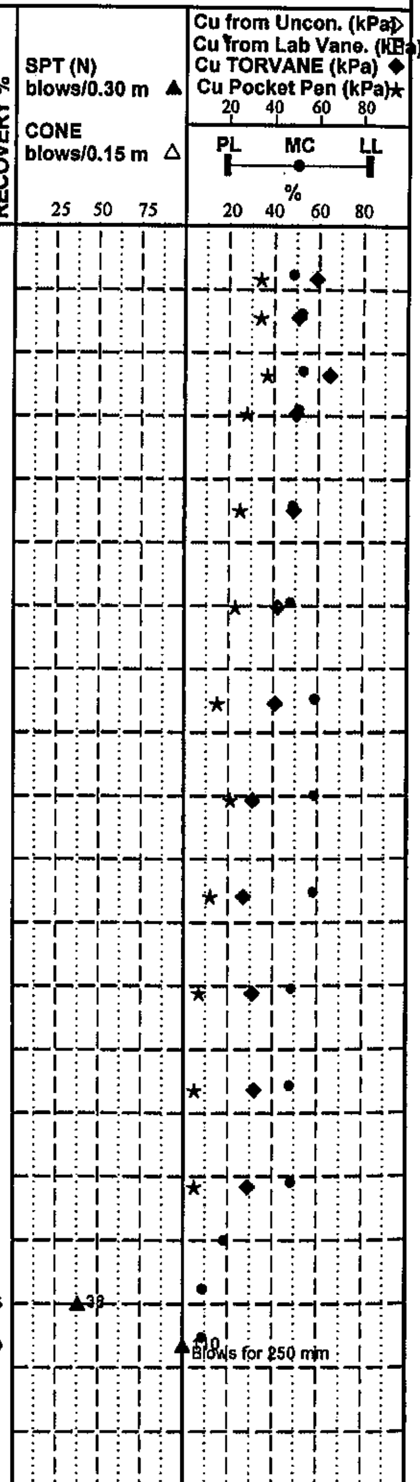
[redacted] **SUMMARY LOG**

REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	231.45
SITE	Foundation Test-Holes		
LOCATION	South of existing bridge, east of floodway channel on slope.	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger, Nodwell	DATE DRILLED	19 May 05

ELEVATION m (ft)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
231.45	0	CLAY (GLACIOLACUSTRINE) - silty - brown trace grey mottling - moist, stiff, high plastic - trace silt inclusions	Auger Cuttings	G107	
			Auger Cuttings	G108	
		- grey below 2.1 m	Auger Cuttings	G109	
			Auger Cuttings	G110	
			Auger Cuttings	G111	
		- firm below 5.8 m	Auger Cuttings	G112	
		- trace gravel below 7.0 m	Auger Cuttings	G113	
			Auger Cuttings	G114	
			Auger Cuttings	G115	
		- trace sand below 12.5 m	Auger Cuttings	G116	
			Auger Cuttings	G117	
			Auger Cuttings	G118	
215.78		SILT (TILL) - some clay, trace to some sand, trace gravel - light brown - moist, loose, low plastic	Auger Cuttings	G119	
		- trace clay, dense below 16.5 m	Split Spoon	S120	65
		- some gravel (limestone) below 17.5 m	Split Spoon	S121	60
213.38	18	END TEST HOLE AT 18.0 m IN SILT TILL. Notes: 1. Power auger refusal at 18.0 m. 2. Seepage from till layer. 3. Sloughing from loose till layer.			



SAMPLE TYPE: Auger Cuttings; Split Spoon

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED ______

DATE 2/8/05

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	236.27
SITE	[redacted]	TOP OF PVC ELEV.	237.29
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	Mobile S61, 125 mm dia. Solid Stem Augers and HQ Coring	DATE DRILLED	14 Dec 05

ELEVATION m (ft)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %
		CLAY (Fill) - silty, some organics - mottled black and grey - moist, very stiff - medium plasticity	0.0 0.3	Auger Cuttings	G168	
235.20	1	CLAY (Glaciolacustrine) - silty - brown, moist, very stiff - high plasticity, trace silt inclusions		Auger Cuttings	G169	
234.44		SILT and CLAY - some sand - some silt inclusions - trace oxidation				
233.22	3	CLAY (Glaciolacustrine) - silty - brown - moist, very stiff - high plasticity, trace silt inclusions - brownish grey between 3.0 m and 4.9 m		Shelby Tube	T170	100
				Auger Cuttings	G171	
		- brown below 4.9 m				
		- grey, firm below 5.8 m				
229.87		- 0.3 m thick sand layer		Shelby Tube	T172	100
229.50						
				Auger Cuttings	G173	
		- trace sand, trace gravel below 7.8 m - trace sulphates between 7.9 m and 8.5 m				
				Shelby Tube	T174	100
				Auger Cuttings	G175	

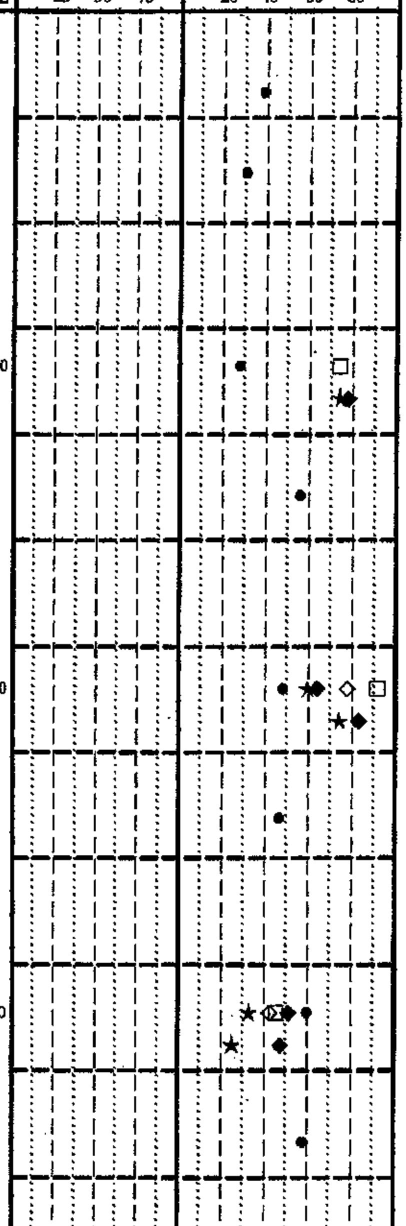
Elevation scale: 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225
Depth scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11; (ft): 5, 10, 15, 20, 25, 30, 35

GRAPHICS | PIEZ. LOG

SPT (N) blows/0.30 m ▲
CONE blows/0.15 m △
25 50 75

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

PL MC LL
%
20 40 60 80



SAMPLE TYPE: Auger Cuttings | Shelby Tube | Split Spoon | Core Barrel

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR
[redacted]

APPROVED [redacted] DATE 28/4/06

[redacted] **SUMMARY LOG** REFERENCE NO. HOLE NO [redacted]

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %
			- clay to till transition zone below 14.6 m				T176	100
							G177	
220.88							G178	
			SILT (Till) - some sand, trace to some gravel, trace clay - moist, loose to compact - low plasticity				T179	100
							S180	39
			- very dense below 16.8 m				S181	83
218.44			LIMESTONE (Bedrock) - mottled brownish yellow and white, R4 strong, moderately close spacing, class 3 apature, class 1 filling - broken to 21.6 m, clay on some joints, evidence of water flow - RQD 43% between 18.3 m and 19.8 m				C182	68
							S183	
							C184	90
			- RQD 29% between 19.8 m and 21.3 m				C185	66
			- RQD 81% between 21.3 m and 22.9 m		21.3			
					22.6		C186	100
213.41			END TEST HOLE AT 22.9 m IN LIMESTONE BEDROCK Note: 1. Auger refusal at 17.4 m, HQ coring below 17.4 m.. 2. Seepage from silt. Sloughing of sand from silt and clay layer. 3. Circulation loss below 18.3 m. 4. Installed standpipe piezometer at 22.9 m in bedrock, complete with above ground cover and Casagrande tip. 5. Water elevation at 225.52 m at time of installation.					

Elevation scale (m): 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211

Depth scale (m): 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25

Depth scale (ft): 40, 45, 50, 55, 60, 65, 70, 75, 80

SPT (N) blows/0.30 m ▲
CONE blows/0.15 m △
25 50 75

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

PL MC LL
%
20 40 60 80

▲14

▲90 blows over 280 mm

▲50 blows over 121 mm

SAMPLE TYPE: Auger Cuttings, Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 28/4/06

Site 11

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT [redacted]

PROJECT [redacted]

SITE [redacted]

LOCATION Permanent Bridge - N 5528115.648 E 647151.268

DRILLING METHOD Mobile S61, 125 mm dia. Solid Stem Augers and HQ Coring

JOB NO. [redacted]

GROUND ELEV. 236.18

TOP OF PVC ELEV. 237.11

WATER ELEV.

DATE DRILLED 23 Nov 05

ELEVATION m (ft)	DEPTH (m)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION
236	0.0	0.0	CLAY (Fill) - silty, trace sand, trace gravel - mottled black and grey, moist, very stiff - medium to high plasticity - no gravel below 0.3 m
235.26	0.3	0.3	CLAY (Glaciolacustrine) - silty - brown - moist, very stiff - high plasticity - trace silt inclusions - some silt inclusions between 2.3 m and 2.6 m - stiff below 2.3 m
			- trace sand, trace gravel, grey below 7.6 m
			- becoming firm with depth below 9.1 m

Elevation scale: 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225

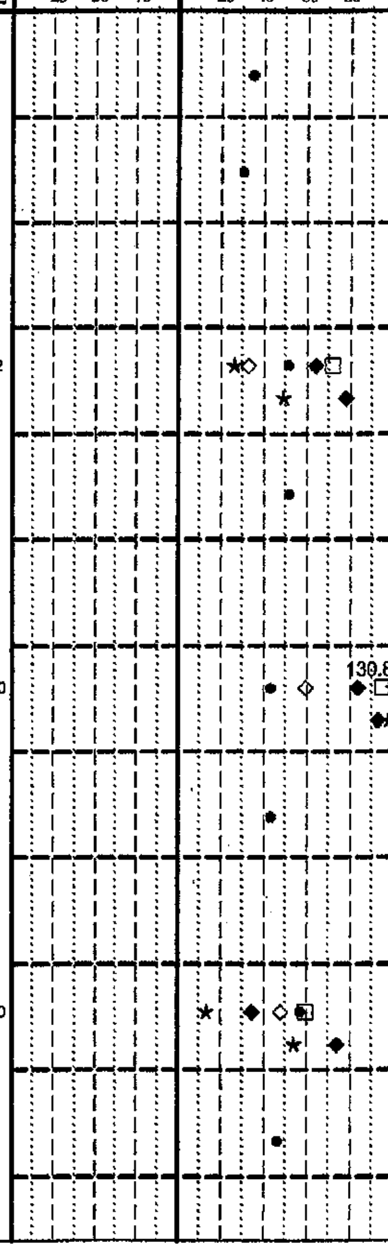
Depth scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11; (ft): 5, 10, 15, 20, 25, 30, 35

SAMPLE TYPE	NUMBER	RECOVERY %
Auger Cuttings	G78	
Auger Cuttings	G79	
Shelby Tube	T80	92
Auger Cuttings	G81	
Shelby Tube	T82	100
Auger Cuttings	G83	
Shelby Tube	T84	100
Auger Cuttings	G85	

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

SPT (N) blows/0.30 m ▲
CONE blows/0.15 m △
25 50 75

PL MC LL
%
20 40 60 80



SAMPLE TYPE: Auger Cuttings, Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted]

DATE 28/4/06

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75)	Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80); PL MC LL % (20 40 60 80)
224	12 / 40					Shelby Tube	T86	100		
223	13									
222.31	45		- clay to till transition zone below 13.6 m - light grey, low plasticity below 13.6 m			Auger Cuttings	G87			
222	14		SILT (Till) - clayey, trace sand, trace gravel - light brown - moist, loose to compact - low plasticity - some clay below 14.3 m			Auger Cuttings	G88			
221	15 / 50					Split Spoon	S89	78	36	
220	16		- trace clay below 15.8 m			Split Spoon	S90	44	25	
219	55 / 17		- dense to very dense below 16.8 m				S91	80	55 blows over 127 mm	
						Core Barrel	C92	18		
218	18									
217.68	60		LIMESTONE (Bedrock) - mottled brownish yellow and white, R3 medium strong, moderately close spacing, class 2 apature, class 1 filling				S93	100	60 blows over 89 mm	
217.13 / 217.10	19		- 25 mm clay lense - RQD 70% between 18.3 m and 19.8 m			Core Barrel	C94	85		
216	65 / 20		- RQD 100% between 19.8 m and 21.3 m							
						Core Barrel	C95	100		
215	21 / 70		- RQD 100% between 21.3 m and 22.9 m		21.3					
214	22				22.6	Core Barrel	C96	100		
213.32	75		END TEST HOLE AT 22.9 m IN LIMESTONE BEDROCK							
213	23		Notes: 1. Auger refusal at 16.9 m, HQ coring below 16.9 m. 2. Seepage and sloughing from till. 3. Loss of circulation at top of bedrock. 4. Installed standpipe piezometer at 22.9 m in bedrock, complete with above ground cover and Casagrande tip. 5. Water level at elevation 225.93 m at time of installation.							
212	24 / 80									
211	25									

SAMPLE TYPE: Auger Cuttings; Shelby Tube; Split Spoon; Core Barrel

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted]   DATE 28/4/06

Site 12

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT [redacted]

PROJECT [redacted]

SITE [redacted]

LOCATION [redacted]

DRILLING METHOD 125 mm ø Solid Stem Auger and HQ Coring, Mobile S61

JOB NO. [redacted]

GROUND ELEV. 232.86

WATER ELEV.

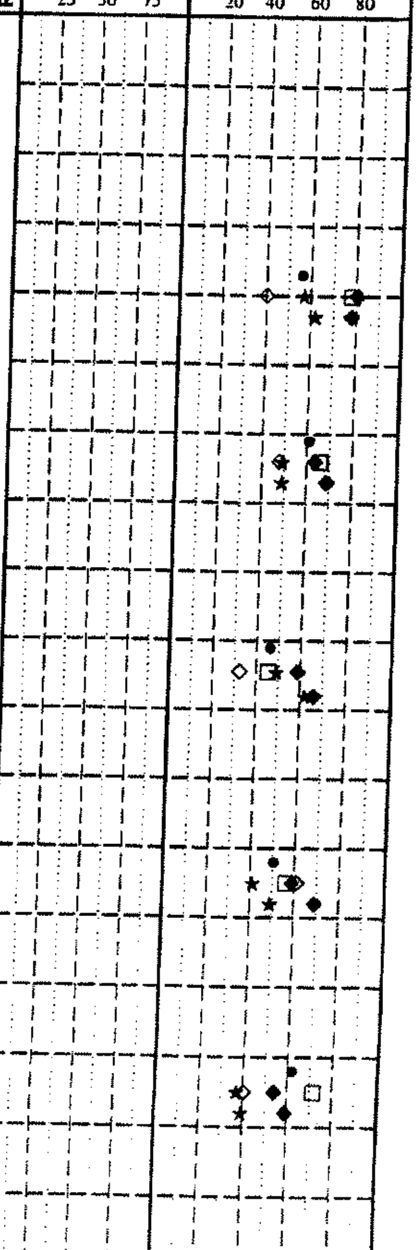
DATE DRILLED 25 Oct 05

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
232	1			CLAY (Glaciolacustrine) - silty - grey and black mottled - moist, very stiff, becoming stiff with depth - trace organics to 1.5 m - brown below 1.5 m, becoming grey with depth			
231	2	5					
230	3	10					
229	4				Shelby Tube	T19	
228	5	15					
227	6	20		- grey below 6.1 m	Shelby Tube	T20	
226	7						
225	8	25					
224	9	30		- numerous light brown silt inclusions at 9.1 m			
223	10				Shelby Tube	T21	
222	11	35					
221	12	40					
220	13				Shelby Tube	T22	
219	14	45					
218	15	50		- firm to stiff below 15 m			
217	16				Shelby Tube	T23	
216	17	55					
215	18						

SPT (N) blows/0.30 m ▲
CONE blows/0.15 m △
25 50 75

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

PL MC LL
%
20 40 60 80



SAMPLE TYPE Shelby Tube Split Spoon Core Barrel

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR
[redacted]

APPROVED [redacted] DATE 25/4/06

ELEVATION m (ft)	DEPTH (m) (ft)	DESCRIPTION AND CLASSIFICATION
	18 m – 21 m	- firm below 18 m
	21 m – 24.5 m	SILT (Till) - some sand to sandy, some gravel, trace cobbles - light brown - compact - moist, low to non plastic - dense to very dense below 22.9 m
208.48	24.5 m – 29.8 m	SAND (Till) - light brown - wet, very dense - fine grained - trace gravel, trace cobbles, and trace boulders below 27.4 m - sand is coarse, brown and multicolored below 27.7 m
203.16	29.8 m – 33.5 m	LIMESTONE (Bedrock) - mottled yellow and white - R3-R4 medium strong to strong - moderately close spacing - Class 2 to 3 aperture - RQD = 33% between 29 and 30.5 m - massive below 30.2 m - RQD = 93% between 30.5 and 32 m - RQD = 94% between 32 and 33.5 m
199.36	33.5 m	END TEST HOLE AT 33.5 m IN LIMESTONE BEDROCK

Notes:
1. Solid stem auger to
2. HQ coring below
3. Seepage from till and bedrock.
4. Squeezing of clay.
5. Loss of water circulation during coring at top of bedrock.
6. Backfilled with bentonite grout to 12.2 m and bentonite chips to surface.

SAMPLE NUMBER	RECOVERY %	SPT (N) blows/0.30 m
T24		
S25		39
C26	0	
S27		50 SPT blows over 125 mm
C28	24	
S29		113 SPT blows over 260 mm
C30	0	
S31		50 SPT blows over 100 mm
C32	0	
C33	52	
S34		50 SPT blows over 125 mm
C35	52	
S36		50 SPT blows over 80 mm
C37	100	
C38	100	
C39	100	

Chart legend: Cu Uncon. (kPa) ◇; Cu Lab Vane. (kPa) □; Cu TORVANE (kPa) ◆; Cu Pocket Pen (kPa) ★ (20 40 60 80); SPT (N) blows/0.30 m ▲; CONE blows/0.15 m △ (25 50 75); PL — MC — LL, % (20 40 60 80)

Elevations: 214, 213, 211, 210, 209, 208.48, 208, 207, 206, 205, 204, 203.16, 203, 202, 201, 200, 199.36, 199, 198, 197, 196, 195, 194
Depth (m): 19–39; Depth (ft): 60–125

SAMPLE TYPE: Shelby Tube, Split Spoon, Core Barrel

CONTRACTOR Paddock Drilling Ltd. INSPECTOR [redacted] APPROVED [redacted] DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT [redacted]

PROJECT [redacted]

SITE [redacted]

LOCATION [redacted]

DRILLING METHOD 125 mm ø Solid Stem Auger and HQ Coring, Mobile S61

JOB NO. [redacted]

GROUND ELEV. 232.45

WATER ELEV.

DATE DRILLED 24 Oct 05

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75)	Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80); PL MC LL % (20 40 60 80)
232			CLAY (Glaciolacustrine) - silty - grey - moist, very stiff - high plasticity - trace silt inclusions - trace organics to 1.5 m - stiff below 2.1 m					
231	1 / 5			Auger Cuttings	G1			
230	2							
229	3 / 10							
228	4 / 15			Auger Cuttings	G2			
227	5							
226	6 / 20							
225	7		- firm below 6.7 m					
224	8 / 25			Auger Cuttings	G3			
223	9 / 30							
222	10							
221	11 / 35			Auger Cuttings	G4			
220	12 / 40		- trace sand, trace gravel below 12.5 m					
219	13							
218	14 / 45			Auger Cuttings	G5			
217	15 / 50							
216	16							
215	17 / 55			Auger Cuttings	G6			
	18							

SAMPLE TYPE ▦ Auger Cuttings ☒ Split Spoon ▯ Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted]

DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

Legend: Cu Uncon. (kPa) ◇; Cu Lab Vane. (kPa) □; Cu TORVANE (kPa) ◆; Cu Pocket Pen (kPa) ★ (scale 20, 40, 60, 80)

SPT (N) blows/0.30 m ▲; CONE blows/0.15 m △ (scale 25, 50, 75)

PL, MC, LL (%) (scale 20, 40, 60, 80)

ELEVATION m (ft)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION
214 – 210.95	18.5 – 21.5	
210.95	21.5	SILT (Till) - some sand to sandy, trace to some gravel, trace cobbles - light brown - moist, compact - low to non-plastic - very dense below 22.9 m
208.35	24.0	SAND (Till) - trace gravel, trace cobbles, trace boulders - light brownish grey - wet, very dense - fine
203.35	29.0	LIMESTONE (Bedrock) - mottled white and yellow - R4-strong - close spacing - Class 2 aperture - RQD = 60% between 29.0 and 30.5 - RQD = 100% between 30.5 and 32.0 m
200.45	32.0	END TEST HOLE AT 32 m IN LIMESTONE BEDROCK

NUMBER	RECOVERY %	SPT (N)	Remarks
G7			
S8		19	
C9	0		
S10		74	
C11	0		
S12		55	
C12	20		
C13			
S14		50	SPT blows over 75 mm, through sand blov
C15	31		
S16		50	SPT blows over 50 mm
C17	93		
C18	100		

Notes:
1. Solid stem auger to 21.5 m.
2. HQ coring below 21.5 m
3. Seepage from clay between 6.0 m and 7.5 m and from till layer.
4. Squeezing of clay, blowup of sand.
5. Loss of water circulation during coring at top of bedrock.
6. Backfilled with bentonite grout to 6.4 m and bentonite chips to surface.

SAMPLE TYPE: Auger Cuttings; Split Spoon; Core Barrel

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED [redacted] DATE 25/4/06

[redacted] **SUMMARY LOG** REFERENCE NO. HOLE NO. [redacted]

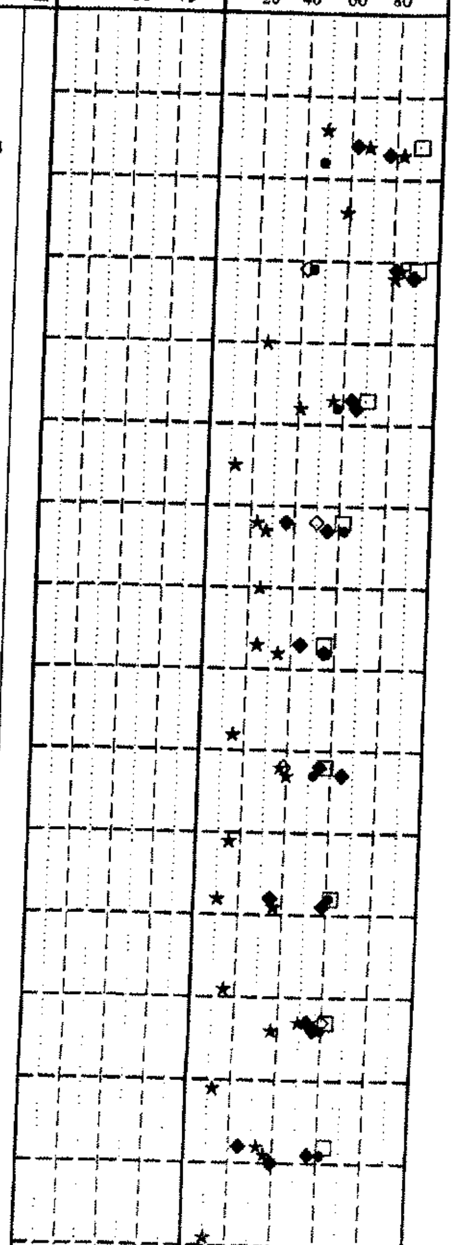
CLIENT	[redacted]	**JOB NO.**	[redacted]
PROJECT	[redacted]	**GROUND ELEV.**	230.24
SITE	[redacted]		
LOCATION	[redacted]	**WATER ELEV.**	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	**DATE DRILLED**	23 Mar 05

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
230	0		CLAY (Glaciolacustrine) - silty - mottled grey and brown - high plasticity - frozen to 1.2 m - trace sand between 0 and 1 m - moist, stiff below 1.2 m			
228	2			Shelby Tube	T74	
227	3 (10)			Shelby Tube	T75	
226	4		- grey, firm to stiff, trace silt inclusions <3mm dia below 3.7 m			
225	5			Shelby Tube	T76	
224	6 (20)		- firm below 6 m	Shelby Tube	T77	
222	8			Shelby Tube	T78	
221	9 (30)			Shelby Tube	T79	
219	11			Shelby Tube	T80	
217	13			Shelby Tube	T81	
216	14 (45)			Shelby Tube	T82	
	15					

SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75

Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80

PL MC LL % 20 40 60 80



SAMPLE TYPE: Shelby Tube, Auger Cuttings, Split Barrel, Core Barrel

CONTRACTOR Paddock Drilling Ltd. INSPECTOR [redacted]

APPROVED [redacted] DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △	Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ PL MC LL %
212.54					T83			
					T84			
			SILT (Till) - some clay, trace sand, trace gravel - light brown - moist to wet, loose to compact - low to non plastic - trace clay, moist, dense below 18.6 m - trace cobbles, some boulders below 19.8 m - moist to wet, very dense below 20.1		G85			
					S86		▲35	
					C87			
208.84					C88			
			SAND (Till) - trace silt - light brown - moist to wet, very dense - fine to medium grained - trace to some gravel, some cobbles, some boulders below 23.2 m		S89	60	▲122	
						0		
					S90	44	▲53 SPT blows over 75 mm	
					C91	45		
					C92	63		
204.04					C93	47		
			LIMESTONE (Bedrock) - mottled white and yellow - no weathering - one fracture 40° to core angle, all others 85 - 90° - no face alterations on non machine breaks - joints are discontinuous - RQD 47% between 25.2 and 26.7 - RQD 95% between 26.7 and 28.4 - mottled white and light brown below 27.1 m - RQD 100% between 28.4 and 29.5 m		C94	95		
200.74					C95	100		
			END TEST HOLE AT 29.5 m IN LIMESTONE BEDROCK. Notes: 1. Solid stem auger to 18.3 m. 2. HQ Coring below 18.3 m. 3. Some squeezing of clay and sloughing of till. 4. Backfilled with bentonite grout to surface.					

Elevation scale: 214, 213, 212, 211, 210, 209, 208, 207, 206, 205, 204, 203, 202, 201, 200, 199, 198

Depth scale (m): 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33; (ft): 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105

SPT scale: 25, 50, 75; Cu scale: 20, 40, 60, 80; % scale: 20, 40, 60, 80

SAMPLE TYPE: Shelby Tube; Auger Cuttings; Split Barrel; Core Barrel

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR
[redacted]

APPROVED [redacted]

DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT [redacted]

PROJECT [redacted]

SITE [redacted]

LOCATION [redacted]

DRILLING METHOD 125 mm ø Solid Stem Auger and HQ Coring, Mobile S61

JOB NO. [redacted]

GROUND ELEV. 227.35

WATER ELEV.

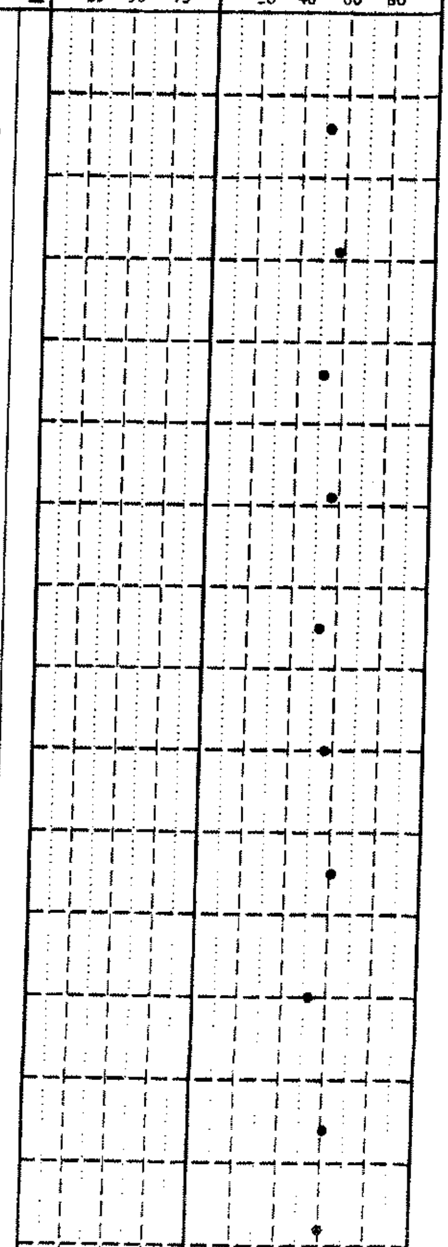
DATE DRILLED 22 Mar 05

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
227				CLAY (Glaciolacustrine) - silty - grey - stiff, moist - high plasticity			
	1						
226		5		- mottled grey with trace brown between 1.2 and 2.7 m	Auger Cuttings	G49	
	2						
225							
	3	10		- trace silt inclusions < 5 mm dia. below 2.9 m	Auger Cuttings	G50	
224							
	4						
223		15			Auger Cuttings	G51	
	5						
222							
	6	20			Auger Cuttings	G52	
221							
	7						
220							
		25			Auger Cuttings	G53	
	8						
219							
	9	30			Auger Cuttings	G54	
218							
	10						
217		35		- becoming firm below 10.4 m	Auger Cuttings	G55	
	11						
216							
	12	40			Auger Cuttings	G56	
215							
	13						
214		45			Auger Cuttings	G57	
	14						
213							
212.25	15				Auger Cuttings	G58	

SPT (N) blows/0.30 m ▲ — CONE blows/0.15 m △ (25, 50, 75)

Cu Uncon. (kPa) ◇ — Cu Lab Vane. (kPa) □ — Cu TORVANE (kPa) ◆ — Cu Pocket Pen (kPa) ★ (20, 40, 60, 80)

PL — MC — LL % (20, 40, 60, 80)



SAMPLE TYPE: Auger Cuttings — Core Barrel — Split Spoon

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 25/4/06

[redacted] **SUMMARY LOG** REFERENCE NO. HOLE NO. [redacted]

Column headings: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75) | Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80) / PL MC LL % (20 40 60 80)

Elevation (m)	Depth (m)	Description and Classification
211.85		SAND (Till) - trace silt, light brown, wet
		SILT (Till)
		- trace clay, trace sand, trace to some coarse gravel
		- brown to light brown
		- compact to very dense, moist to wet
		- low plasticity
		- some cobbles below 15.8 m
		- non plastic below 17.4 m
		- light brown below 17.4 m
208.75		SAND (Till)
		- trace silt, trace gravel, trace cobbles
		- light brown
		- moist to wet, dense to very dense
		- fine to medium grained, rounded grains
		- some cobbles below 21.3 m
		- silty, some gravel, non plastic to low plastic below 21.9 m
		- some boulders below 24.4 m
202.65		LIMESTONE (Bedrock)
		- mottled white and light brown
		- no surface weathering
		- no fracture perpendicular to core angle
		- RQD 100% between 24.7 and 25 m
		- RQD 100% between 25 and 26.5 m
		- mottled white and yellow between 25.6 and 26.4 m
		- RQD 80% between 26.5 and 27.7 m
		- RQD 70% between 27.7 and 28 m
199.35	28.0	END OF TEST HOLE AT 28.0 m IN LIMESTONE BEDROCK.

Notes:
1. Solid stem auger refusal at 15.5 m.
2. HQ coring below 15.5 m.
3. Sloughing of till.
4. Backfilled with bentonite grout to surface.

Number	Recovery %	SPT (N)
G59		
G60		
C61		
S62	50	56
C63	10	
S64	30	93
C65	13	
S66		68
	0	
S67	75	113
C68	20	
S69	100	86 SPT blows over 100 mm
	0	
S70	78	36 SPT blows over 50 mm
C71	50	
C72	100	
C73	84	

SAMPLE TYPE: Auger Cuttings | Core Barrel | Split Spoon

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 25/4/06

[redacted] **SUMMARY LOG**

REFERENCE NO.

HOLE NO. **U 05 9035**

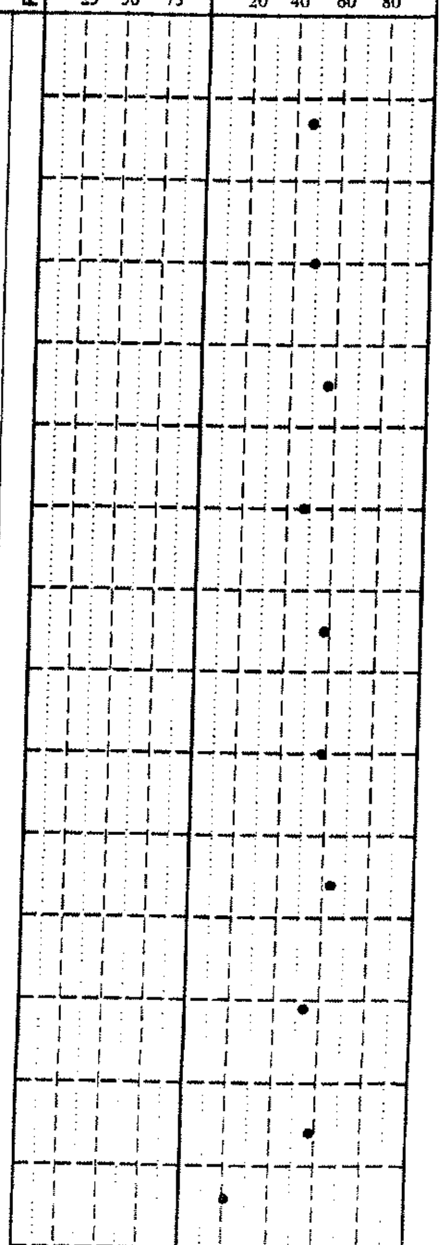
CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	226.59
SITE	[redacted]		
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	21 Mar 05

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
226	1	5		CLAY (Glaciolacustrine) - silty - dark brownish grey - stiff to firm, moist - high plasticity - trace silt inclusions <3 mm dia.			
225	2				Auger Cuttings	G26	
224	3	10		- grey and firm below 2.4 m, becoming soft with depth	Auger Cuttings	G27	
223	4						
222	5	15		- trace gravel between 4.9 and 6.7 m	Auger Cuttings	G28	
221	6	20			Auger Cuttings	G29	
220	7						
219	8	25			Auger Cuttings	G30	
218	9	30			Auger Cuttings	G31	
217	10						
216	11	35			Auger Cuttings	G32	
215	12	40		- trace gravel below 11.3 m	Auger Cuttings	G33	
214	13						
213	14	45			Auger Cuttings	G34	
212.39				SILT (Till) - trace to some sand, trace to some gravel, trace to some cobbles - light brown	Core Barrel	C35	
212	15				Core Barrel	C36	

SPT (N) blows/0.30 m ▲
CONE blows/0.15 m △
25 50 75

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ✱
20 40 60 80

PL MC LL
%
20 40 60 80



SAMPLE TYPE: Auger Cuttings | Core Barrel | Split Spoon

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR
[redacted]

APPROVED [redacted] DATE 25/4/06

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75)	Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80); PL MC LL % (20 40 60 80)
211 – 208.08	16 – 18.9 (55 – 60)		- moist, loose to compact - low to non plastic - sand seam between 14.3 and 14.5 m (brown, moist to wet, fine to medium, rounded) - granite boulder 300 mm dia., grey, between 14.9 and 15.2 m - low plastic, compact to dense below 15.7 m		C37 S38 C39 S40 	 93 100 0	 21 SPT started bouncing after 225 mm 40	
208.08 – 204.79	18.9 – 21.9 (60 – 70)		SAND (Till) - trace silt, trace gravel - light brown - moist, loose to compact, non-plastic - fine to medium grained		S41 S42 	94 0 85 0	25 11	
204.79 – 202.79	21.9 – 23.9 (70 – 75)		SILT (Till) - some sand, some gravel, some cobbles - brown - moist to wet, dense to very dense - non plastic		S43 C44 C45	90	80	
202.79 – 198.99	23.9 – 27.7 (80 – 90)		LIMESTONE (Bedrock) - mottled white and yellow - RQD 95% between 23.8 and 24.7 m - RQD 88 % between 24.7 and 26.2 m - mottled white and light brown below 25 m - RQD 100% between 26.2 and 27.7 m		C46 C47 C48	95 88 100		
198 – 194	27.7 – 33 (90 – 105)		END TESTHOLE AT 27.7 m IN LIMESTONE BEDROCK Notes: 1. Solid stem auger refusal at 14.3 m. 2. HQ coring below 14.3 m. 3. Seepage from till. 4. Sloughing of sand and till layers. 4. Backfilled with bentonite grout to surface.					

SAMPLE TYPE: Auger Cuttings; Core Barrel; Split Spoon

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT [redacted]

PROJECT [redacted]

SITE [redacted]

LOCATION [redacted]

DRILLING METHOD 125 mm ø Solid Stem Auger and HQ Coring, Mobile S61

JOB NO. [redacted]

GROUND ELEV. 226.03

WATER ELEV.

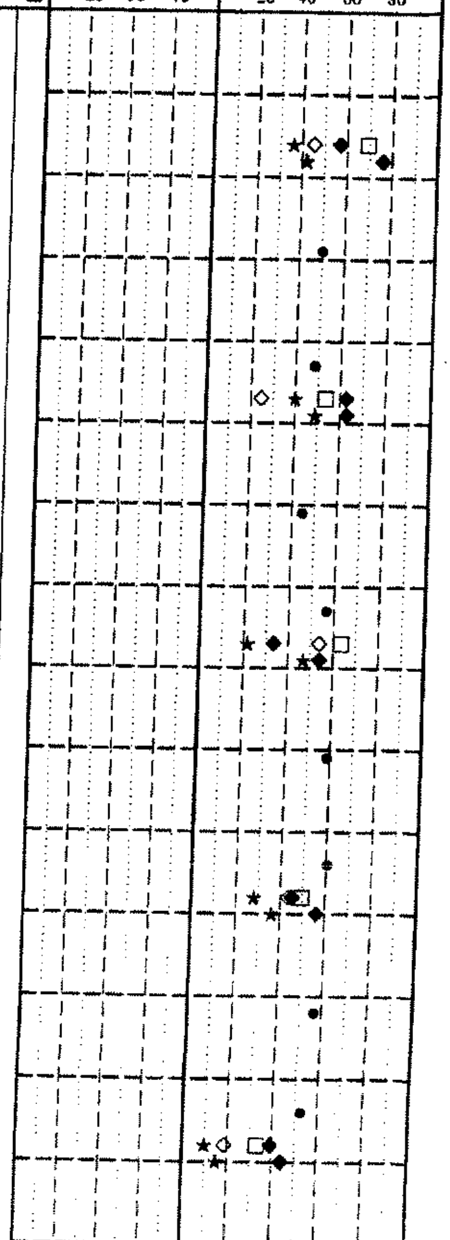
DATE DRILLED 18 Mar 05

ELEVATION m (ft)	DEPTH (m) (ft)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
225	1	CLAY (Glaciolacustrine) - silty, trace sand, trace organics - firm, moist, medium plasticity - frozen to 0.6 m			
224.53	5 ft	CLAY (Glaciolacustrine) - silty - grey - firm to stiff, moist - high plasticity - trace silt inclusions <3mm dia.			
224	2		Shelby Tube	T1	
223	3 / 10 ft		Auger Cuttings	G2	
222	4				
	15 ft		Auger Cuttings	G3	
221	5		Shelby Tube	T4	
220	6 / 20 ft		Auger Cuttings	G5	
219	7				
	25 ft		Auger Cuttings	G6	
218	8		Shelby Tube	T7	
217	9 / 30 ft		Auger Cuttings	G8	
216	10	- firm below 10 m			
	35 ft		Auger Cuttings	G9	
215	11		Shelby Tube	T10	
214	12 / 40 ft	- firm to soft below 12.0 m	Auger Cuttings	G11	
213	13				
	45 ft		Auger Cuttings	G12	
212	14		Shelby Tube	T13	
211.43		SILT (Till) - trace clay, tace to some sand, trace to some gravel, trace to some cobbles.			
211	15		Auger Cuttings	G14	

SPT (N) blows/0.30 m ▲
CONE blows/0.15 m △
25 50 75

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

PL MC LL
%
20 40 60 80



SAMPLE TYPE: Shelby Tube | Auger Cuttings | Split Spoon | Core Barrel

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 25/4/06

[redacted] **SUMMARY LOG** REFERENCE NO. HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75)
				trace to some boulders - yellowish pink - loose at surface dense to very dense with depth, moist to wet - low to non plastic - light brown below 15.5 m		S15	90	85 SPT blows over 211 mm
210	16					C16		
209	17	55				S17	80	79
208	18	60					0	
207	19			- soft layers between 18.4 and 19.8 m		C18	0	
206	20	65						
205	21	70					0	
204	22					S19	100	84 SPT blows over 115 mm
203 / 202.83	23	75				C20		
				LIMESTONE (Bedrock) - mottled white and light brown - no weathering on faces - some breaks 80° to core angle - massive				
202	24	80				C21	100	
						C22		
201	25			- RQD 100% between 24.8 and 25.3 m		C23	100	
				- RQD 96% between 25.3 and 26.8 m				
200	26	85				C24	96	
199	27	90		- RQD 100% between 26.8 and 28.3 m				
198 / 197.73	28					C25	100	
				END OF TEST HOLE AT 28.3 m IN LIMESTONE BEDROCK Notes: 1. Solid stem auger to 15.2 m. 2. HQ coring below 15.2 m. 3. Water level in test hole 4 m below ground surface before coring started. 4. Backfill with bentonite grout to surface.				
197	29	95						
196	30							
195	31	100						
194	32	105						
193	33							

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★
20 40 60 80

PL MC LL
%
20 40 60 80

SAMPLE TYPE: Shelby Tube, Auger Cuttings, Split Spoon, Core Barrel

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR
[redacted]

APPROVED [redacted] DATE 25/4/06

[redacted] **SUMMARY LOG** REFERENCE NO. HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	224.38
SITE	[redacted]		
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	28 Mar 05

ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75 | Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80 | PL MC LL % 20 40 60 80

Elevation scale: 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210.08, 210, 209.88

Depth scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15; (ft): 5, 10, 15, 20, 25, 30, 35, 40, 45

CLAY (Glaciolacustrine)
- silty
- grey
- moist, high plasticity
- stiff, becoming firm with depth
- trace silt inclusions <5mm dia.

- trace fine gravel between 4.9 and 7.3 m

SAND (Till) - grey and brown, wet, medium to coarse

SILT (Till)
- trace clay, trace to some sand, trace gravel, trace cobbles, trace boulders

Samples: G96, G97, G98, G99, G100, G101, G102, G103, G104, G105, G106

SAMPLE TYPE: Auger Cuttings | Split Spoon | Core Barrel

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 25/4/06

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75)	Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80); PL MC LL % (20 40 60 80)
209			- light brown, brown below 14.8 m - moist, compact, dense to very dense below 15.2 m - low plasticity below 15.2 m		G107			
	16				S108	45	106	
208					C109	53		
207	17 / 55				S110	65	100	
						0		
206	18 / 60		SAND seam at 18.3 m, trace silt, some gravel, moist to wet, very dense, non plastic		S111	0.5	78	
205	19		- 240 mm limestone boulder at 19.2 m			0		
204	20 / 65				S112		SPT blows over 50 mm.	
					C113	43		
203	21 / 70				C114	16		
202.28	22							
202			LIMESTONE (Bedrock) - white and yellowish orange mottled - highly fractured - RQD = 12% between 21.6 and 22.9 m - massive, no foliation or lamination below 22.3 m - RQD = 100% between 22.9 and 24.4 m		C115	63		
201	23 / 75				C116	100		
					C117	100		
200	24 / 80		- RQD = 100% between 24.4 and 25.9 m - mottled white and light brown below 24.4 m					
199	25				C118	100		
198.46	26 / 85		END OF TEST HOLE AT 25.9 m IN LIMESTONE BEDROCK					

END OF TEST HOLE AT 25.9 m IN LIMESTONE BEDROCK

Notes:

1. Solid stem auger to 15.2 m.
2. HQ coring below 15.2 m.
3. Water level in test hole 1.5 m below ground surface before coring started.
4. Unsure of lithology between 16.8 and 22.1 m due to poor core and spt sample recovery.
5. Sand blown up into core barrel at 18.3 m.
6. Backfill with bentonite grout to 1.8 m and bentonite chips to surface.

SAMPLE TYPE: Auger Cuttings, Split Spoon, Core Barrel

CONTRACTOR: Paddock Drilling Ltd.   INSPECTOR: [redacted]   APPROVED: [redacted]   DATE: 25/4/06

[redacted] SUMMARY LOG

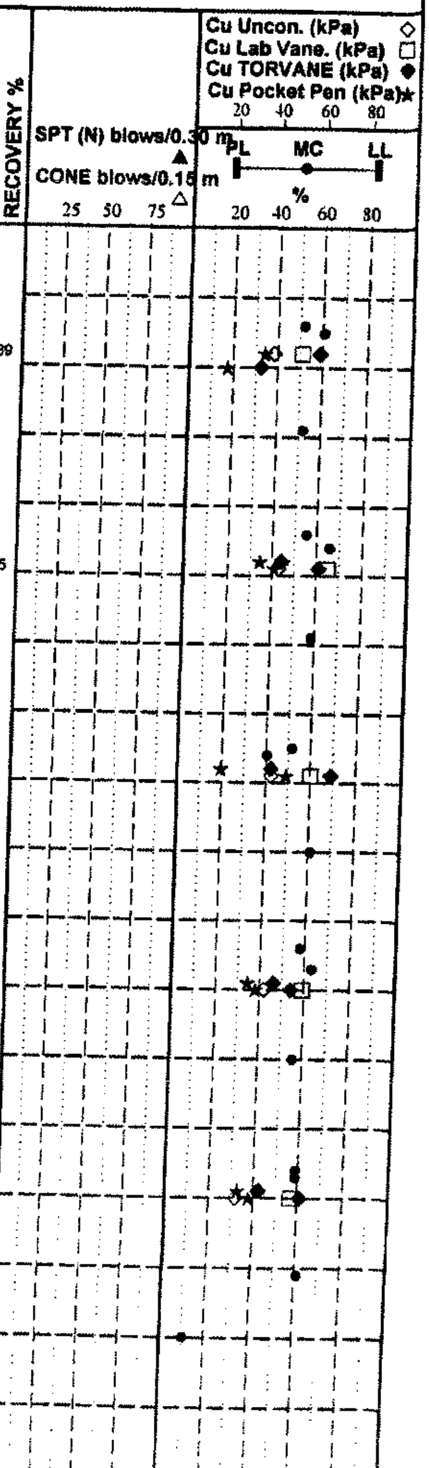
REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	226.22
SITE	[redacted]		
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	30 Mar 05

ELEVATION m (ft)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
226	0	CLAY (Glaciolacustrine) - silty - grey - moist, firm to stiff - high plastic - frozen to 0.9 m			
224	2		Auger Cuttings / Shelby Tube	G119 / T120	89
223	3	- trace silt inclusions < 5 mm dia. below 3 m	Auger Cuttings	G121	
221	5		Auger Cuttings / Shelby Tube	G122 / T123	75
220	6		Auger Cuttings	G124	
218	8		Auger Cuttings / Shelby Tube	G125 / T126	83
217	9		Auger Cuttings	G127	
215	11		Auger Cuttings / Shelby Tube	G128 / T129	73
214	12	- trace gravel below 12.2 m	Auger Cuttings	G130	
212	14		Auger Cuttings / Shelby Tube	G131 / T132	72
210.72	15.5	SILT (Till) - sandy	Auger Cuttings	G133	
210.02	16.2	END OF TEST HOLE AT 16.2 m IN SILT TILL Notes: 1. Seepage from till layer. 2. Sand blow up 16.2 m. 3. Backfilled with bentonite pellets to surface.	Auger Cuttings	G134	

Depth scale: (m) 1–18; (ft) 5–55. Elevation scale: 226–209.



SAMPLE TYPE: Auger Cuttings, Shelby Tube

CONTRACTOR Paddock Drilling Ltd.	INSPECTOR [redacted]	APPROVED [redacted]	DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	226.74
SITE	[redacted]		
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	31 Mar 05

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75)	Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80); PL MC LL % (20 40 60 80)
226 – 211.04	0 – 15.7		CLAY (Glaciolacustrine) - silty - grey - moist, stiff, high plastic - frozen to 0.3 m - trace organics to 0.3 m	Auger Cuttings	G135			
				Auger Cuttings	G136			
			- trace silt inclusions < 5 mm dia. below 3.7 m	Auger Cuttings	G137			
			- firm below 6 m	Auger Cuttings	G138			
				Auger Cuttings	G139			
				Auger Cuttings	G140			
				Auger Cuttings	G141			
				Auger Cuttings	G142			
				Auger Cuttings	G143			
				Auger Cuttings	G144			
			- trace silt inclusions < 30 mm dia. below 15.4 m	Split Spoon	S145	10	▲38	
211 – 209	15.7 – 18		SILT (Till) - trace to some sand, trace gravel, trace cobbles, trace boulders - light brown, compact at surface, dense to very dense below 16.5 m - moist to wet, low to non plastic - granite boulder between 15.8 and 16.1 m	Core Barrel	C146	28		
				Split Spoon	S147	88	▲96 SPT blows over 226 mm	
				Core Barrel	C148	66		

SAMPLE TYPE: Auger Cuttings; Split Spoon; Core Barrel

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted]

DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △	PL / MC / LL %
208	19	60		S149	40	93	
					0		
206.94	20	65	SAND (Till) - light brown - moist to wet, very dense	S150	88	128	50 blows over 225 mm
206	21	70		C151	32		
205	22			C152	13		
203.84	23	75	LIMESTONE (Bedrock) - mottled white and light brown - massive - RQD 98% between 22.9 and 24.4 m - RQD 100% between 24.4 and 25.9 m	C153	100		
202	25	80		C154	100		
200.84	26	85	END TEST HOLE AT 25.9 m IN LIMESTONE BEDROCK				

Notes:
1. Solid Stem auger to 16.8 m.
2. HQ coring below 16.8 m.
3. Sloughing of till layer.
4. Seepage from till.
5. Backfilled with bentonite grout to near ground surface and capped with bentonite chips.

Cu Uncon. (kPa) ◇
Cu Lab Vane. (kPa) □
Cu TORVANE (kPa) ◆
Cu Pocket Pen (kPa) ★

SAMPLE TYPE: Auger Cuttings; Split Spoon; Core Barrel

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted]

DATE: 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

CLIENT [redacted]

PROJECT [redacted]

SITE [redacted]

LOCATION [redacted]

DRILLING METHOD 125 mm ø Solid Stem Auger and HQ Coring, Mobile S61

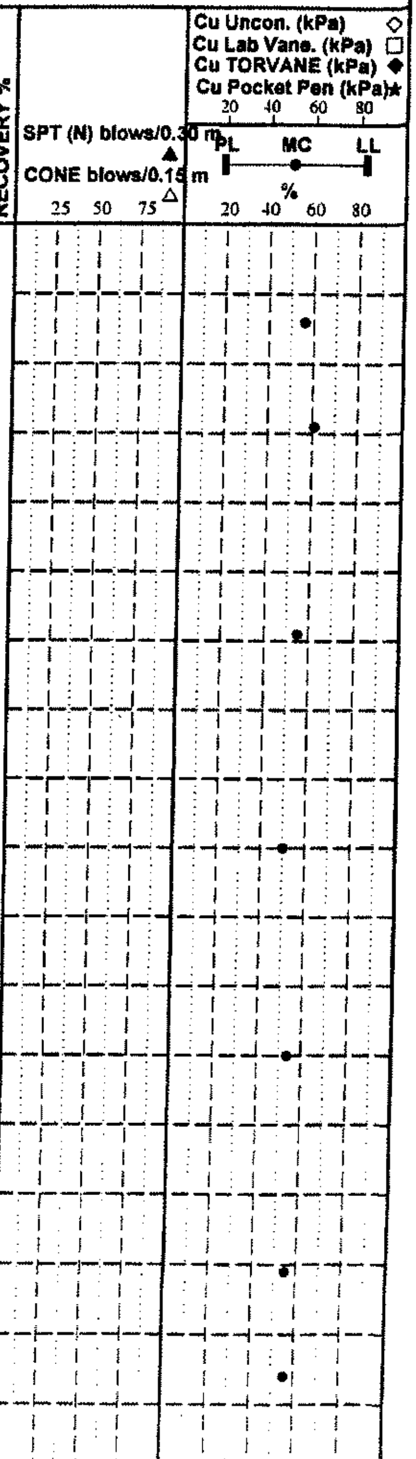
JOB NO. [redacted]

GROUND ELEV. 228.50

WATER ELEV.

DATE DRILLED 27 Oct 05

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
228	0			CLAY (Glaciolacustrine) - silty - brown - moist, stiff - high plastic			
227	1–2	5			Auger Cuttings	G40	
226	3	10		- grey and firm below 3.0 m	Auger Cuttings	G41	
225	4						
224	5	15					
223							
222	6	20			Auger Cuttings	G42	
221	7–8	25					
220	9	30			Auger Cuttings	G43	
219	10						
218	11	35					
217							
216	12	40			Auger Cuttings	G44	
215	13–14	45					
214							
213	15	50		- trace sand, trace gravel, soft below 15.5 m	Auger Cuttings	G45	
212	16–17	55		- some light brown till inclusions below 16.3 m	Auger Cuttings	G46	
211	18						



SAMPLE TYPE: Auger Cuttings | Core Barrel | Split Spoon

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ (25 50 75) | Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ (20 40 60 80) | PL MC LL % (20 40 60 80)

Elevation (m)	Depth (m)	Description and Classification
208.80		
208.40	20	SILT (Till) - some sand to sandy, some gravel, some cobbles - light brown - moist, dense to very dense, low to non plastic
203.50	25	SAND (Till) - trace gravel - brown - moist to wet, dense to very dense - fine grained sand - sand is well graded below 23.6 m, trace to some gravel, trace to some cobbles below 23.6 m - gravelly below 24.4 m
199.50	29	LIMESTONE (Bedrock) - mottled yellow and white - R4 strong - close spacing - Class 2 aperture - RQD = 37 % between 24.4 and 25.9 m - RQD = 94 % between 25.9 and 27.4 m - RQD = 94% between 27.4 and 29 m

Number	Recovery %
G47	
C48	28
S49	
C50	0
S50	
C51	0
C53	13
S54	
C55	79
C56	100
C57	99

105 SPT blows over 280 mm

50 SPT blows over 115 mm

84 SPT blows over 255 mm

65 SPT blows over 100 mm

END TEST HOLE AT 29 m IN LIMESTONE BEDROCK

Notes:
1. Solid stem auger to 18.3 m
2. HQ coring below 18.3 m
3. Seepage from clay and till.
4. Squeezing of clay.
5. Blow-up of sand
6. Loss of circulation in bedrock.
7. Backfilled with bentonite grout to 4.9 m and bentonite chips to surface.

SAMPLE TYPE: Auger Cuttings | Core Barrel | Split Spoon

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

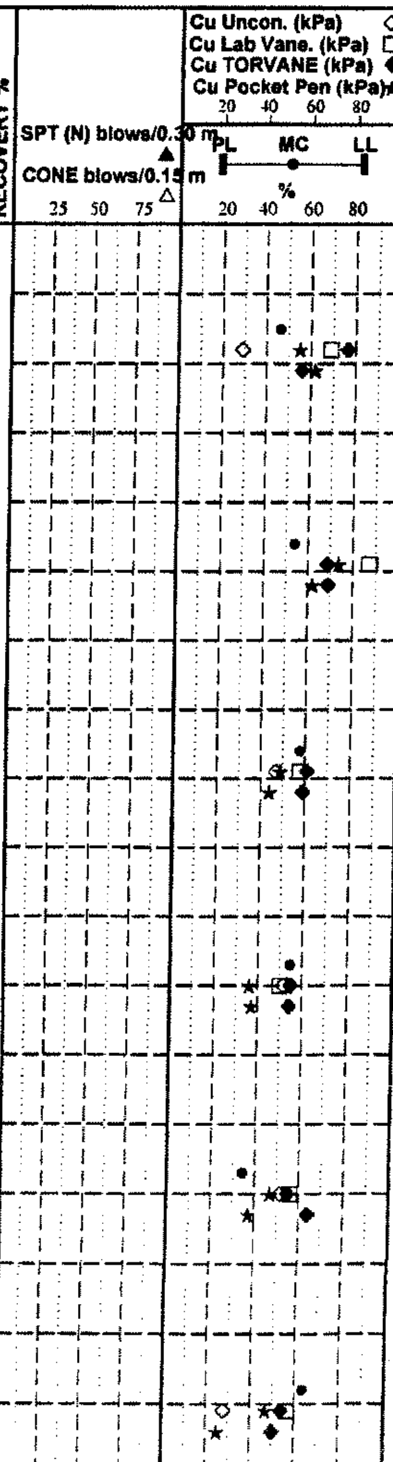
APPROVED: [redacted]

DATE: 25/4/06

[redacted] SUMMARY LOG REFERENCE NO. HOLE NO. [redacted]

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	231.07
SITE	[redacted]		
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	31 Oct 05

ELEVATION m (ft)	DEPTH (m)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
231–229	0–2		CLAY (Glaciolacustrine) - silty - brown - moist, stiff - high plastic - trace silt inclusions	Shelby Tube	T58	
228	3		- grey below 3 m			
226	5			Shelby Tube	T59	
223	8		- firm to stiff below 8.0 m	Shelby Tube	T60	
220	11			Shelby Tube	T61	
217	14			Shelby Tube	T62	
214	17			Shelby Tube	T63	



SAMPLE TYPE: Shelby Tube, Auger Cuttings, Split Spoon, Core Barrel

CONTRACTOR Paddock Drilling Ltd. INSPECTOR [redacted] APPROVED [redacted] DATE 25/4/06

[redacted] **SUMMARY LOG**　　REFERENCE NO.　　HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE / NUMBER	RECOVERY %	SPT (N) blows/0.30 m / CONE blows/0.15 m	MC %
213 – 210.97	18.3 – 20.3	- firm below 18.3 m - 0.3 m sand, silty, some gravel, light brown, wet below 18.4 m - 0.15 m sand, silty, some gravel, light brown, wet at 19.4 m	G64			•
210.97 – 209.77	20.3 – 21.5	SILT (Till) - sandy, trace gravel - light brown - moist, dense, low plasticity	G65			•
209.77 – 203.67	21.5 – 27.6	SAND (Till) - light grey - wet, dense to very dense - fine sand, some coarse sand - well graded below 22.9 m	S66		▲35	•
			C67	0		
			S68		▲ SPT blows over 230 mm	•
			C69	0		
			S70		▲103 SPT blows over 230 mm	
			C71	24		
			C72	47		
			S73			•
			C74	47		
203.67 – 200.57	27.6 – 30.5	LIMESTONE (Bedrock) - mottled yellow and white - R4-strong - close spacing - Class 2 aperture - RQD = 92% between 27.4 and 29.0 m - fracture at 28.3 m - RQD = 100% between 29 and 30.5 m	S75	100		
			C76			
			C77	100		
			C78	100		
200.57	30.5	END TEST HOLE AT 30.5 m IN LIMESTONE BEDROCK				

Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ — 20 40 60 80
PL — MC — LL (%) 20 40 60 80
SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75

END TEST HOLE AT 30.5 m IN LIMESTONE BEDROCK

Notes:

1. Solid stem auger to 21.3 m.
2. HQ coring below 21. 3 m.
3. Squeezing of clay.
4. Seepage from sand below 21.3 m.
5. Blow up of sand between 21.3 and 27.4 m.
6. Backfill with bentonite grout to 3.4 m and bentonite chips to surface.

SAMPLE TYPE: Shelby Tube | Auger Cuttings | Split Spoon | Core Barrel

CONTRACTOR: Paddock Drilling Ltd.　　INSPECTOR: [redacted]　　APPROVED: [redacted]　　DATE: 25/4/06

[redacted] **SUMMARY LOG** REFERENCE NO. HOLE NO. [redacted] of 2

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	232.56
SITE	[redacted]	TOP OF PVC ELEV.	233.48
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	2 Nov 05

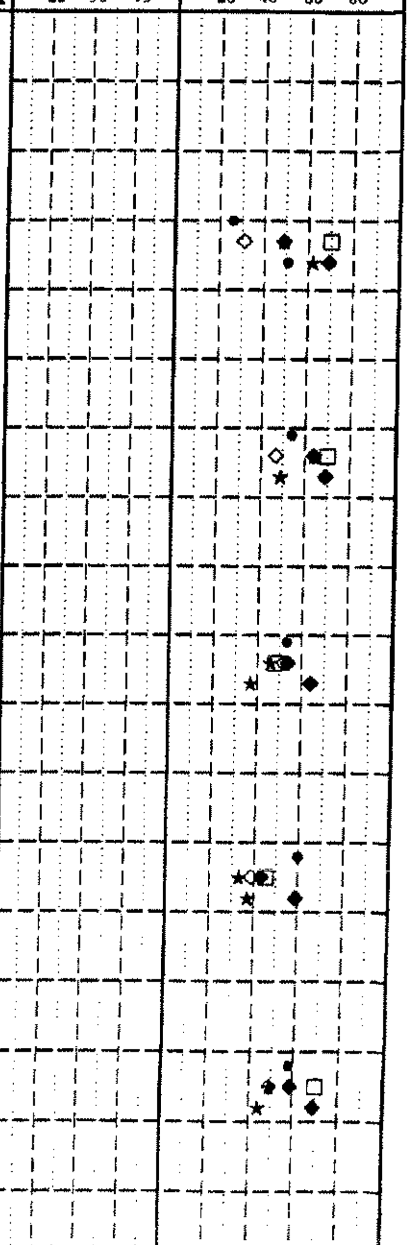
ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	DEPTH (m)	SAMPLE TYPE	NUMBER
232 / 231.66			CLAY (Fill) - silty, trace organics - black/brown mottled, moist, very stiff, high plasticity			
231 / 230 / 229.66	1 / 2	5	CLAY (Glaciolacustrine) - silty - brown - moist, very stiff - high plasticity - trace organics to 1.5 m		Auger Cuttings	G79
229.41	3	10	SILT - some clay, light brown, loose, wet, low plastic, seeping			
229 / 228 / 227	4 / 5	15	CLAY (Glaciolacustrine) - silty - brown - moist, stiff - high plasticity - grey with depth below 4.5 m	3.3	Shelby Tube	T80
226 / 225 / 224	6 / 7 / 8	20 / 25	- grey, trace silt inclusions below 6.1 m		Shelby Tube	T81
223 / 222 / 221	9 / 10 / 11	30 / 35			Shelby Tube	T82
220 / 219 / 218 / 217 / 216 / 215	12 / 13 / 14 / 15 / 16 / 17 / 18	40 / 45 / 50 / 55	- becoming firm below 12 m		Shelby Tube	T83, T84

PIEZ. LOG — RECOVERY %

SPT (N) blows/0.30 m ▲ — CONE blows/0.15 m △ — 25 50 75

Cu Uncon. (kPa) ◇ — Cu Lab Vane. (kPa) □ — Cu TORVANE (kPa) ◆ — Cu Pocket Pen (kPa) ★ — 20 40 60 80

PL — MC — LL — % — 20 40 60 80



SAMPLE TYPE: Auger Cuttings — Shelby Tube — Core Barrel — Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	[redacted]	[redacted]	25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted] of 2

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %
214	19	60–65			Shelby Tube	T85	
212.46	20	65	SILT (Till) - sandy, trace boulders - light brown - moist, dense, low plasticity		Core Barrel	C86	20
211.26	21	70	SAND (Till) - silty, trace gravel, trace cobbles - brown - wet, dense to very dense		Split Spoon / Core Barrel	S87 C88	 17
209.66	23	75	SILT (Till) - sandy, trace gravel - brown - moist, dense to very dense, low to non plastic		Split Spoon / Core Barrel	S89 C90	
208.16	24	80	SAND (Till) - trace gravel - brown - wet, very dense - fine grained		Split Spoon / Core Barrel	S91 C92 S93 C94 S95 C96	 0 0 10
203.16	29.3	95	LIMESTONE (Bedrock) - mottled yellow and white - fractured and broken Below 29.9 m: - R4-strong - close spacing - Class 2 aperture - RQD = 73% between 30.5 and 32.0 m - clay seam ~50 mm thick at 31.3 m - RQD = 89% between 32.0 and 33.5 m	30.9 33.2 33.5	Core Barrel	C97 C98 C99	70 79 100
199.06	33.5	110	END TEST HOLE AT 33.5 m IN LIMESTONE BEDROCK				

Elevation scale: 214, 213, 212.46, 212, 211.26, 211, 210, 209.66, 209, 208.16, 208, 207, 206, 205, 204, 203.16, 203, 202, 201, 200, 199.06, 199, 198, 197, 196, 195, 194

Depth (m): 19 – 39; Depth (ft): 60 – 125

SPT (N) blows/0.30 m ▲ — CONE blows/0.15 m △ — 25 50 75

Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ — 20 40 60 80

PL MC LL — % — 20 40 60 80

▲55 (S87)

▲60 SPT blows over 75 mm (S89)

▲98 SPT blows over 225 mm (S91)

▲55 SPT blows over 100 mm (S93)

▲55 SPT blows over 100 mm (S95)

Notes:
1. Solid stem auger to 20.1 m
2. HQ core below 20.1 m.
3. No sloughing.
4. Seepage from till.
5. Installed bedrock piezometer at 33.5 m. Backfill with sand to 30.9 m, bentonite grout to 3.4 m, and bentonite chips to surface. (U 05 9042 A).

SAMPLE TYPE: Auger Cuttings | Shelby Tube | Core Barrel | Split Spoon

CONTRACTOR: Paddock Drilling Ltd.

INSPECTOR: [redacted]

APPROVED: [redacted] DATE: 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted] of 2

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	233.68
SITE	[redacted]	TOP OF PVC ELEV.	234.58
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	2 Nov 05

ELEVATION m (ft)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75	Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80; PL MC LL % 20 40 60 80
233			CLAY (Fill) - silty, trace organics - black/brown mottled, moist, very stiff, high plasticity							
232 231.88			CLAY (Glaciolacustrine) - silty - brown - moist, very stiff - high plasticity - trace organics to 1.5 m							
231										
230 229.88			SILT - some clay, light brown, loose, wet, low plastic, seeping							
229.43			CLAY (Glaciolacustrine) - silty - brown - moist, stiff - high plasticity - grey with depth below 4.5 m							
229										
228										
227			- grey, trace silt inclusions below 6.1 m							
226										
225										
224										
223										
222										
221										
220										
219										
218										
217										
216										

Depth scale (m): 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18; (ft): 5 10 15 20 25 30 35 40 45 50 55

SAMPLE TYPE

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 25/4/06

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75	Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80; PL MC LL % 20 40 60 80
		60									
215	19										
214	20	65				20.4					
213	21										
212.48		70		SILT (Till) - sandy, trace boulders - light brown, moist, dense, low plastic		21.6					
212.04				END TEST HOLE AT 21.64 m IN SILT TILL							
212	22										
211	23	75									
210	24	80									
209	25										
208	26	85									
207	27	90									
206	28										
205	29	95									
204	30	100									
203	31										
202	32	105									
201	33	110									
200	34										
199	35	115									
198	36	120									
197	37										
196	38	125									
195	39										

Notes:
1. Solid stem auger refusal at 21.64 m on boulder. SPT attempted at bottom of hole, bouncing on a boulder.
2. No sloughing.
3. Seepage from till.
4. Installed till piezometer at 21.64 m. Backfilled with sand to 20.4 m and bentonite to surface.
5. Soil stratigraphy is based on U05 9042 A (located 9 m away).

SAMPLE TYPE

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]2

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	233.70
SITE	[redacted]	TOP OF PVC ELEV.	234.57
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	2 Nov 05

Column headings: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75 | Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80; PL MC LL % 20 40 60 80

ELEVATION m	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	PIEZ. DEPTH (m)
231.70	0.0 – 2.0	CLAY (Fill) - silty, trace organics - black/brown mottled, moist, very stiff, high plasticity	
229.70	2.0 – 4.0	CLAY (Glaciolacustrine) - silty - brown - moist, very stiff - high plasticity - trace organics to 1.5 m	
229.45	4.0 – 4.25	SILT - some clay, light brown, loose, wet, low plastic, seeping	
218.50	4.25 – 15.2	CLAY (Glaciolacustrine) - silty - brown - moist, stiff - high plasticity - grey with depth below 5.6 m - grey, trace silt inclusions below 7.2 m - firm below 13.1 m	10.7 15.2

Elevation scale: 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216

Depth scale (m): 1 – 18; (ft): 5 – 55

END TEST HOLE AT 15.2 m IN CLAY

Notes:

1. Solid stem auger to 15.2 m.
2. No seepage.
3. No sloughing.
4. Installed pneumatic piezometers at U05-9042 D at 15.2m and U05-9042 C at 10.7 m in clay. Backfilled with a cement and bentonite grout mixture (1 bag cement, 2 bags bentonite

SAMPLE TYPE

CONTRACTOR Paddock Drilling Ltd. | INSPECTOR [redacted] | APPROVED [redacted] | DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted]

ELEVATION m (ft)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m ▲ CONE blows/0.15 m △ 25 50 75	Cu Uncon. (kPa) ◇ Cu Lab Vane. (kPa) □ Cu TORVANE (kPa) ◆ Cu Pocket Pen (kPa) ★ 20 40 60 80; PL MC LL % 20 40 60 80
		60		grout per45 gallon drum) 5. Stratigraphy is based on 9042A (located 10 m away).							
215	19										
214	20	65									
213	21	70									
212	22										
211	23	75									
210	24	80									
209	25										
208	26	85									
207	27	90									
206	28										
205	29	95									
204	30	100									
203	31										
202	32	105									
201	33	110									
200	34										
199	35	115									
198	36	120									
197	37										
196	38	125									
195	39										

SAMPLE TYPE

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

APPROVED [redacted] DATE 25/4/06

[redacted] SUMMARY LOG

REFERENCE NO.

HOLE NO. [redacted] 1 of 1

CLIENT	[redacted]	JOB NO.	[redacted]
PROJECT	[redacted]	GROUND ELEV.	233.81
SITE	[redacted]	TOP OF PVC ELEV.	234.62
LOCATION	[redacted]	WATER ELEV.	
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Coring, Mobile S61	DATE DRILLED	2 Nov 05

Column headings: ELEVATION m (ft) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.30 m ▲ / CONE blows/0.15 m △ (25 50 75) | Cu Uncon. (kPa) ◇ / Cu Lab Vane. (kPa) □ / Cu TORVANE (kPa) ◆ / Cu Pocket Pen (kPa) ★ (20 40 60 80); PL MC LL % (20 40 60 80)

ELEVATION m	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	DEPTH (m)
233.81	0	CLAY (Fill) - silty, trace organics - black/brown mottled, moist, very stiff, high plasticity	
231.81	2.0	CLAY (Glaciolacustrine) - silty - brown - moist, very stiff - high plasticity	
229.61	4.2	SILT - some clay, light brown, loose, wet, low plastic, seeping	
229.36	4.45	CLAY (Glaciolacustrine) - silty - brown - moist, stiff - high plasticity - grey with depth below 5.8 m - grey, trace silt inclusions below 7.4 m - firm below 13.3 m	14.8
218.61	15.2	END TEST HOLE AT15.2 m IN CLAY	15.2

Notes:
1. No seepage.
2. No sloughing.
3. Soil stratigraphy is based on U05 9042 A (located 9 m away).
4. Installed pneumatic piezometer at 15.2 m in clay. Backfilled with sand to 14.6 m, and bentonite to surface.

SAMPLE TYPE

CONTRACTOR Paddock Drilling Ltd.

INSPECTOR [redacted]

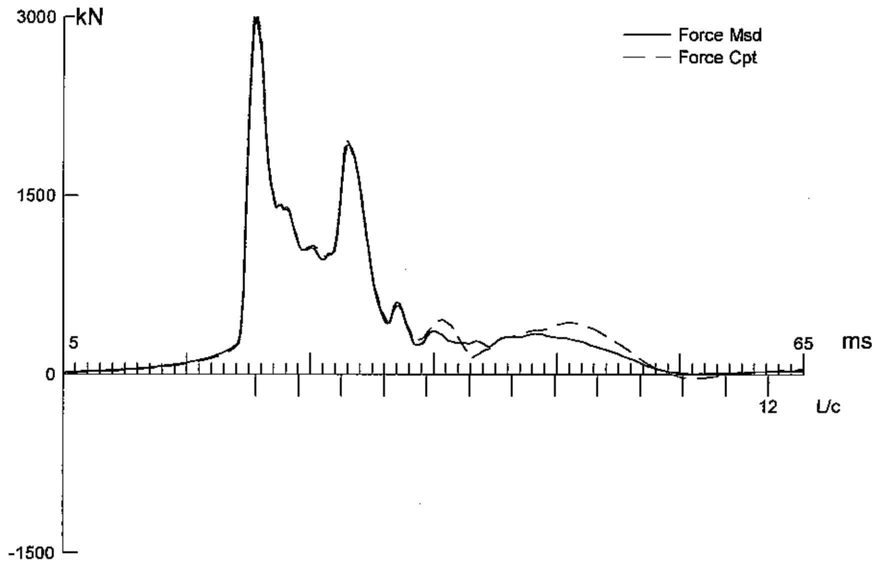
APPROVED [redacted] DATE 25/4/06

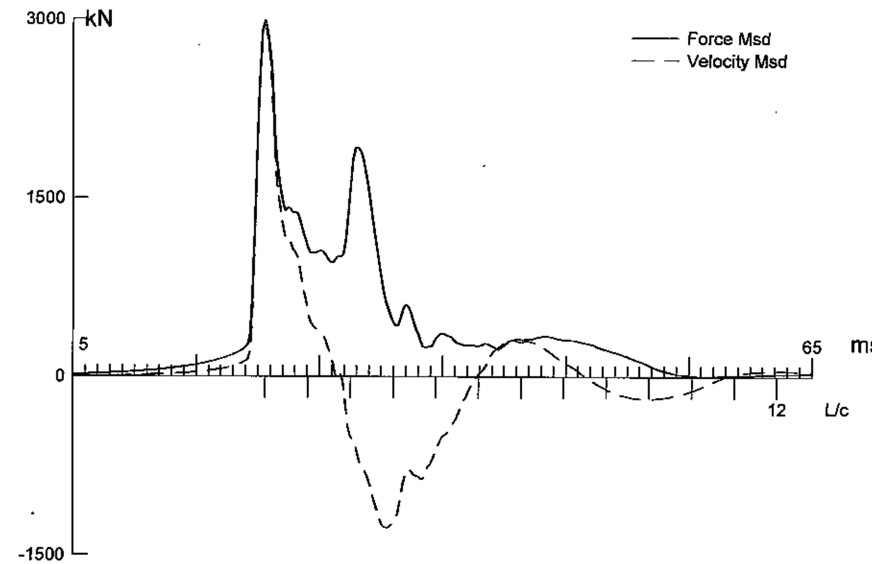
Appendix B:

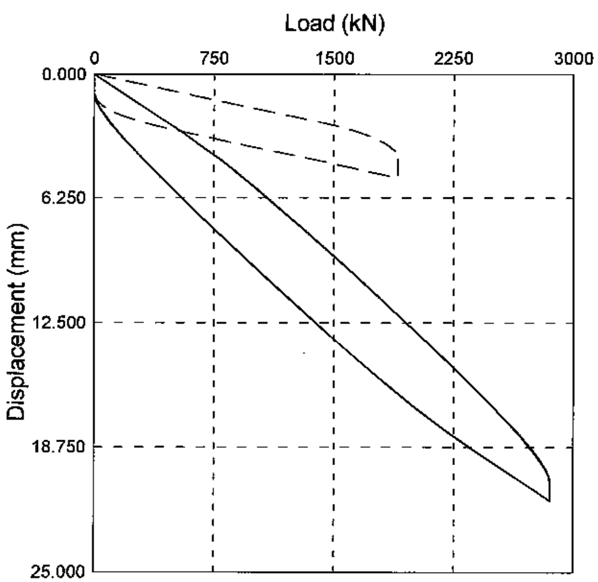
CAPWAP Outputs

Site 1

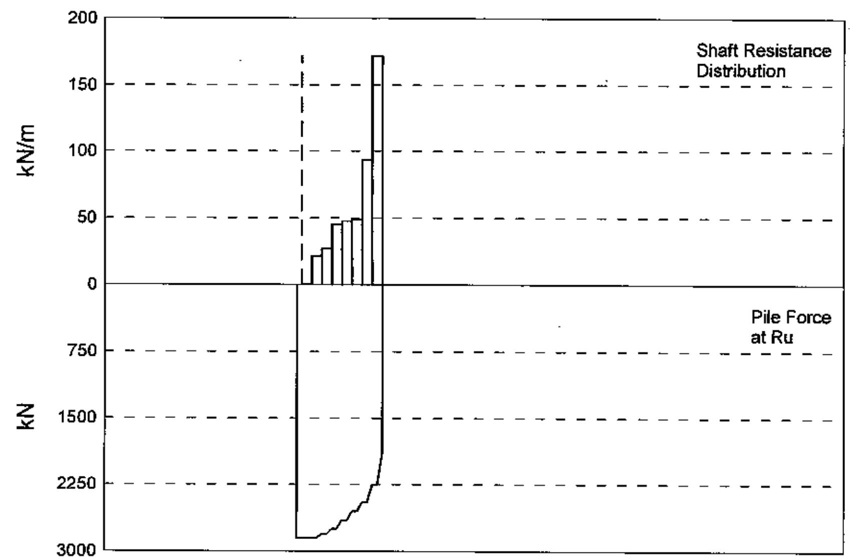
[redacted]







Ru = 2852.5 kN
Rs = 952.5 kN
Rb = 1900.0 kN
Dy = 20.4 mm
Dx = 21.4 mm



[redacted]

[redacted]

EOID; Blow: 345

[redacted]

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2852.5; along Shaft 952.5; at Toe 1900.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2852.5				
1	3.1	1.1	1.0	2851.5	1.0	0.88	0.71	0.279
2	5.2	3.2	44.9	2806.6	45.9	21.50	17.40	0.279
3	7.3	5.3	56.2	2750.4	102.1	26.91	21.77	0.279
4	9.4	7.4	94.3	2656.1	196.4	45.16	36.54	0.279
5	11.5	9.5	99.6	2556.5	296.0	47.70	38.59	0.279
6	13.6	11.6	102.1	2454.4	398.1	48.89	39.56	0.279
7	15.7	13.7	194.8	2259.6	592.9	93.28	75.47	0.279
8	17.8	15.8	359.6	1900.0	952.5	172.20	139.32	0.279
Avg. Shaft			119.1			60.48	48.93	0.279
Toe			1900.0				19899.46	0.214

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.871	3.265
Case Damping Factor		0.467	0.714
Unloading Quake	(% of loading quake)	87	85
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	3	
Resistance Gap (included in Toe Quake) (mm)			0.499

```
CAPWAP match quality   =    2.20        (Wave Up Match) ; RSA = 0
Observed: final set    =   1.000 mm;   blow count     =    1000 b/m
Computed: final set    =   0.740 mm;   blow count     =    1351 b/m

max. Top Comp. Stress  =   216.7 MPa    (T=  21.0 ms, max= 1.051 x Top)
max. Comp. Stress      =   227.7 MPa    (Z=  17.8 m, T=  24.5 ms)
max. Tens. Stress      =  -19.56 MPa    (Z=  13.6 m, T=  36.3 ms)
max. Energy (EMX)      =   27.84 kJ;    max. Measured Top Displ. (DMX)=14.92 mm
```

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3055.7	-40.1	216.7	-2.85	27.84	5.1	14.573
2	2.1	3062.9	-48.7	217.2	-3.45	27.64	5.1	14.198
3	3.1	3082.2	-56.3	218.6	-3.99	27.41	5.1	13.782
4	4.2	3109.6	-63.1	220.5	-4.48	27.08	5.0	13.326
5	5.2	3136.6	-68.9	222.5	-4.89	26.72	5.0	12.822
6	6.3	3066.3	-71.4	217.5	-5.07	25.38	4.9	12.319
7	7.3	3105.0	-78.0	220.2	-5.53	25.02	4.8	11.805
8	8.4	3026.8	-88.8	214.7	-6.30	23.50	4.8	11.274
9	9.4	3073.2	-168.9	218.0	-11.98	22.98	4.7	10.663
10	10.4	2911.5	-222.9	206.5	-15.81	20.67	4.6	10.011
11	11.5	2961.0	-255.2	210.0	-18.10	19.92	4.5	9.259
12	12.5	2792.4	-262.9	198.0	-18.65	17.61	4.4	8.517
13	13.6	2863.5	-275.8	203.1	-19.56	16.76	4.3	7.745
14	14.6	2723.0	-266.6	193.1	-18.91	14.67	4.1	6.989
15	15.7	2756.2	-265.6	195.5	-18.84	13.74	4.1	6.166
16	16.7	2865.5	-232.6	203.2	-16.50	11.01	3.8	5.383
17	17.8	3210.0	-259.9	227.7	-18.43	8.99	3.0	4.576
Absolute	17.8			227.7			(T =	24.5 ms)
	13.6				-19.56		(T =	36.3 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3970.6	3756.8	3543.0	3329.2	3115.4	2901.7	2687.9	2474.1	2260.3	2046.6
RX	4063.1	3869.6	3708.9	3553.9	3398.8	3243.8	3088.8	2942.6	2799.8	2718.7
RU	4052.6	3847.1	3641.5	3435.9	3230.4	3024.8	2819.2	2613.7	2408.1	2202.5

RAU = 1661.1 (kN); RA2 = 2556.3 (kN)

Current CAPWAP Ru = 2852.5 (kN); Corresponding J(RP)= 0.52; J(RX) = 0.76

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.39	20.79	3069.8	3038.6	3059.6	14.917	0.927	1.000	28.4	3564.5

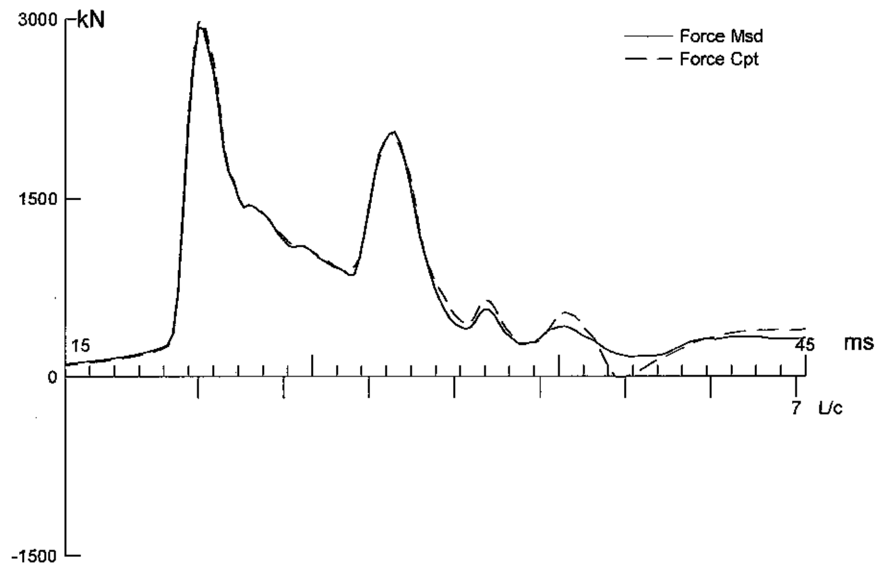
PILE PROFILE AND PILE MODEL

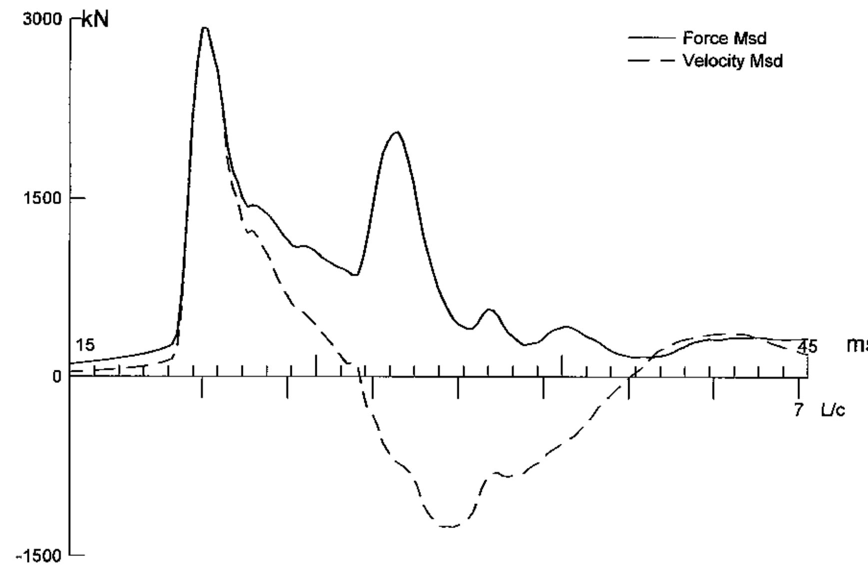
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.75	141.00	206842.7	77.287	1.236

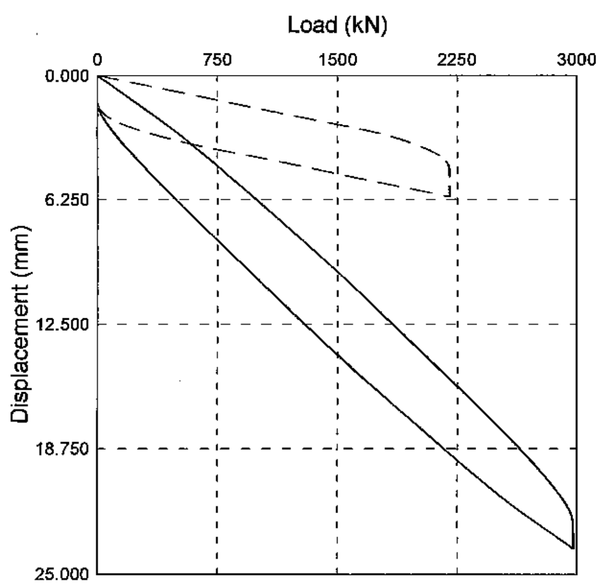
Toe Area 0.095 m^2

Top Segment Length 1.04 m, Top Impedance 569.29 kN/m/s

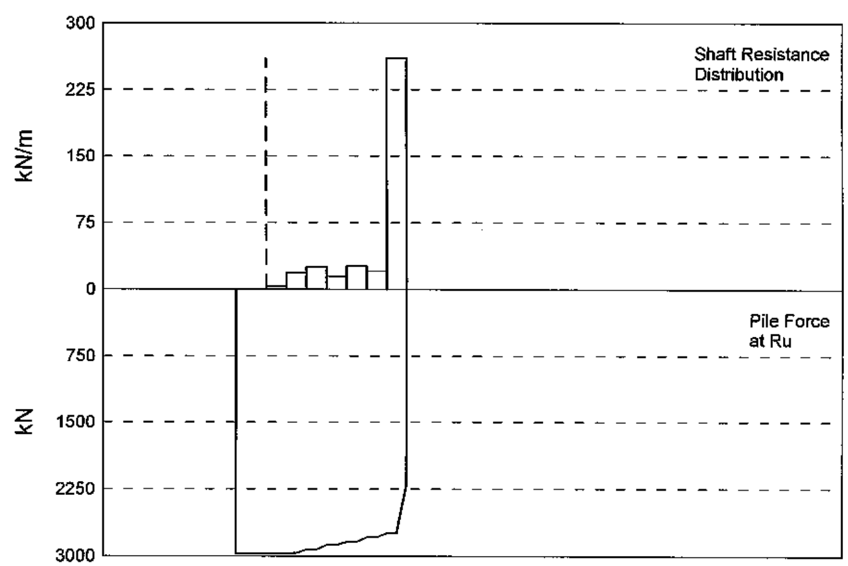
Pile Damping 0.5 %, Time Incr 0.204 ms, Wave Speed 5123.0 m/s, 2L/c 6.9 ms







Ru =	2975.9	kN
Rs =	773.6	kN
Rb =	2202.3	kN
Dy =	22.3	mm
Dx =	23.7	mm



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CAPWAP(R) 2006-3
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2975.9; along Shaft 773.6; at Toe 2202.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2975.9					
1	5.2	1.5	8.1	2967.8	8.1	5.51	4.46	0.722	1.153
2	7.3	3.6	38.4	2929.4	46.5	18.39	14.88	0.722	1.154
3	9.4	5.6	53.0	2876.4	99.5	25.38	20.53	0.722	1.154
4	11.5	7.7	31.2	2845.2	130.7	14.94	12.09	0.722	1.154
5	13.6	9.8	54.9	2790.3	185.6	26.29	21.27	0.722	1.154
6	15.7	11.9	43.3	2747.0	228.9	20.74	16.78	0.722	1.154
7	17.8	14.0	544.7	2202.3	773.6	260.84	211.04	0.722	1.154
Avg. Shaft			110.5			55.26	44.71	0.722	1.154
Toe			2202.3				23065.56	0.106	3.625

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.981	0.410
Unloading Quake	(% of loading quake)	30	31
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	16	
Resistance Gap (included in Toe Quake) (mm)			0.373

CAPWAP match quality = 2.74 (Wave Up Match) ; RSA = 0
Observed: final set = 1.400 mm; blow count = 714 b/m
Computed: final set = 1.754 mm; blow count = 570 b/m

max. Top Comp. Stress = 211.6 MPa (T= 21.0 ms, max= 1.160 x Top)
max. Comp. Stress = 245.5 MPa (Z= 17.8 m, T= 24.7 ms)
max. Tens. Stress = -23.02 MPa (Z= 17.8 m, T= 34.9 ms)
max. Energy (EMX) = 27.84 kJ; max. Measured Top Displ. (DMX)=15.38 mm

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EOID; Blow: 231
[redacted]

Test: 26-Jun-2012 13:33:
CAPWAP(R) 2006-3
OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2982.9	-59.3	211.6	-4.21	27.84	5.0	14.964
2	2.1	2989.2	-63.5	212.0	-4.50	27.64	5.0	14.598
3	3.1	3001.7	-81.3	212.9	-5.77	27.41	5.0	14.186
4	4.2	3016.8	-108.5	214.0	-7.70	27.12	4.9	13.725
5	5.2	3052.7	-126.9	216.5	-9.00	26.75	4.9	13.228
6	6.3	3056.7	-145.5	216.8	-10.32	26.00	4.8	12.675
7	7.3	3106.0	-193.6	220.3	-13.73	25.52	4.7	12.086
8	8.4	2989.5	-257.4	212.0	-18.25	23.65	4.6	11.499
9	9.4	3026.3	-290.5	214.6	-20.60	23.07	4.6	10.862
10	10.4	2836.1	-302.3	201.1	-21.44	20.70	4.5	10.179
11	11.5	2882.4	-304.5	204.4	-21.60	19.91	4.4	9.425
12	12.5	2809.0	-292.8	199.2	-20.76	18.13	4.3	8.628
13	13.6	2852.5	-289.6	202.3	-20.54	17.12	4.2	7.782
14	14.6	2681.9	-268.1	190.2	-19.02	14.85	4.2	6.931
15	15.7	2879.4	-322.1	204.2	-22.84	13.69	4.0	6.022
16	16.7	3135.4	-324.2	222.4	-22.99	11.60	3.6	5.042
17	17.8	3461.2	-324.5	245.5	-23.02	6.38	2.9	4.045
Absolute	17.8			245.5			(T =	24.7 ms)
	17.8				-23.02		(T =	34.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3924.0	3709.5	3495.0	3280.4	3065.9	2851.4	2636.9	2422.4	2207.8	1993.3
RX	3989.0	3788.4	3651.5	3514.6	3377.7	3243.0	3114.4	3007.1	2926.9	2869.2
RU	3911.6	3695.9	3480.1	3264.4	3048.6	2832.8	2617.1	2401.3	2185.6	1969.8

RAU = 1374.3 (kN); RA2 = 2565.9 (kN)

Current CAPWAP Ru = 2975.9 (kN); Corresponding J(RP)= 0.44; J(RX) = 0.74

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.34	20.58	3039.5	3029.7	3029.7	15.377	1.595	1.400	28.4	3381.0

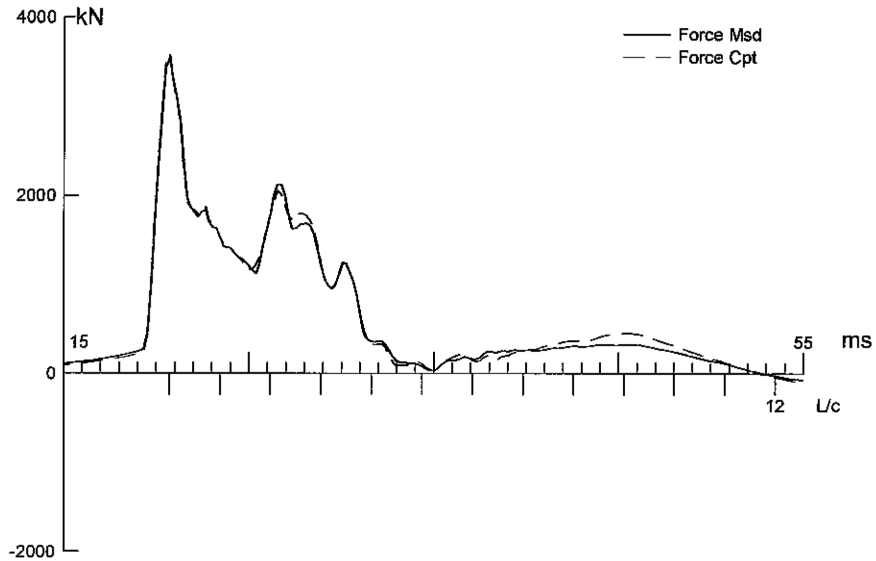
PILE PROFILE AND PILE MODEL

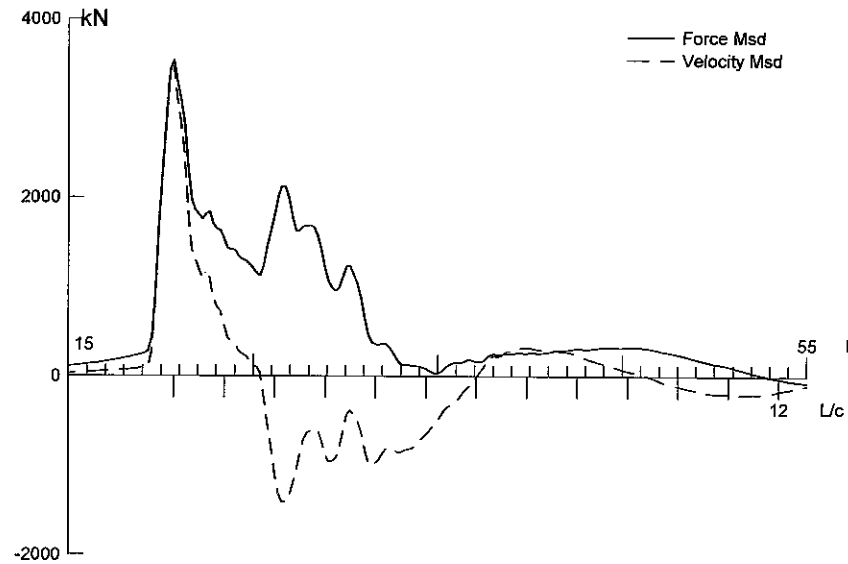
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.75	141.00	206842.7	77.287	1.236

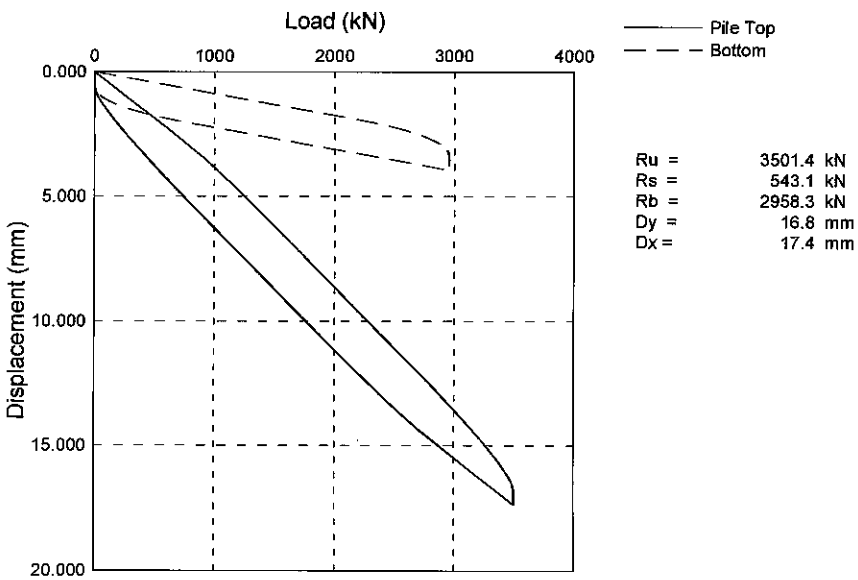
Toe Area 0.095 m^2

Top Segment Length 1.04 m, Top Impedance 569.29 kN/m/s

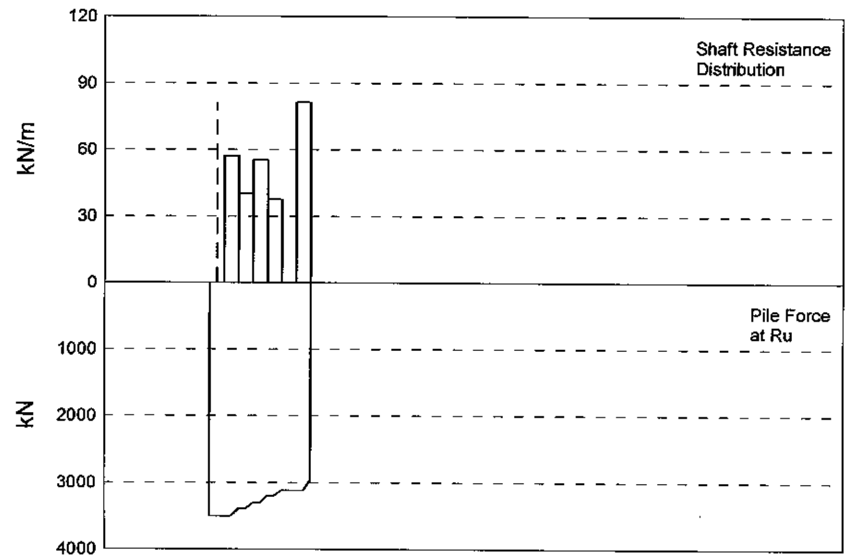
Pile Damping 1.0 %, Time Incr 0.204 ms, Wave Speed 5123.0 m/s, 2L/c 6.9 ms







Ru = 3501.4 kN
Rs = 543.1 kN
Rb = 2958.3 kN
Dy = 16.8 mm
Dx = 17.4 mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3501.4; along Shaft 543.1; at Toe 2958.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				3501.4				
1	4.0	2.6	114.1	3387.3	114.1	43.89	35.00	0.632
2	6.0	4.6	80.3	3307.0	194.4	40.13	32.00	0.632
3	8.0	6.6	110.4	3196.6	304.8	55.20	44.02	0.632
4	10.0	8.6	75.2	3121.4	380.0	37.62	30.00	0.632
5	12.0	10.6	0.0	3121.4	380.0	0.00	0.00	0.000
6	14.0	12.6	163.1	2958.3	543.1	81.54	65.02	0.632
Avg. Shaft			90.5			43.10	34.37	0.632
Toe			2958.3				30100.00	0.290

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.980	2.578
Case Damping Factor		0.509	1.272
Unloading Quake	(% of loading quake)	100	107
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	16	
Resistance Gap (included in Toe Quake) (mm)			0.193

```
CAPWAP match quality    =    1.82        (Wave Up Match) ; RSA = 0
Observed: final set     =   0.600 mm;    blow count     =    1667 b/m
Computed: final set     =   0.100 mm;    blow count     =    9999 b/m

max. Top Comp. Stress   =   216.0 MPa    (T=  21.1 ms, max= 1.154 x Top)
max. Comp. Stress       =   249.3 MPa    (Z=  14.0 m, T=  23.6 ms)
max. Tens. Stress       =  -16.57 MPa    (Z=  14.0 m, T=  36.5 ms)
max. Energy (EMX)       =   27.17 kJ;    max. Measured Top Displ. (DMX)=11.59 mm
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EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3607.5	-118.7	216.0	-7.11	27.17	5.1	11.468
2	2.0	3687.5	-124.9	220.8	-7.48	26.88	5.0	11.055
3	3.0	3786.6	-128.9	226.7	-7.72	26.53	4.8	10.613
4	4.0	3874.2	-132.9	232.0	-7.96	26.14	4.7	10.130
5	5.0	3500.6	-116.4	209.6	-6.97	22.75	4.6	9.637
6	6.0	3592.1	-158.6	215.1	-9.50	22.26	4.4	9.119
7	7.0	3374.1	-183.7	202.0	-11.00	19.83	4.3	8.524
8	8.0	3457.2	-227.5	207.0	-13.62	19.11	4.2	7.893
9	9.0	3115.7	-234.1	186.6	-14.02	16.12	4.1	7.200
10	10.0	3152.2	-256.1	188.8	-15.33	15.29	4.0	6.503
11	11.0	2919.5	-244.2	174.8	-14.62	13.18	4.0	5.789
12	12.0	3238.5	-270.2	193.9	-16.18	12.10	3.7	4.982
13	13.0	3753.3	-275.9	224.7	-16.52	10.54	3.2	4.057
14	14.0	4163.7	-276.8	249.3	-16.57	9.25	2.4	3.088
Absolute	14.0			249.3			(T =	23.6 ms)
	14.0				-16.57		(T =	36.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4970.1	4740.9	4511.7	4282.5	4053.3	3824.2	3595.0	3365.8	3136.6	2907.5
RX	5037.0	4829.8	4626.9	4455.7	4305.2	4165.4	4037.7	3918.2	3798.7	3679.2
RU	5120.8	4906.7	4692.6	4478.5	4264.4	4050.3	3836.2	3622.1	3408.0	3193.9

RAU = 2021.2 (kN); RA2 = 3637.0 (kN)

Current CAPWAP Ru = 3501.4 (kN); Corresponding J(RP)= 0.64; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.39	20.89	3632.2	3629.7	3629.7	11.586	0.522	0.600	27.6	4525.0

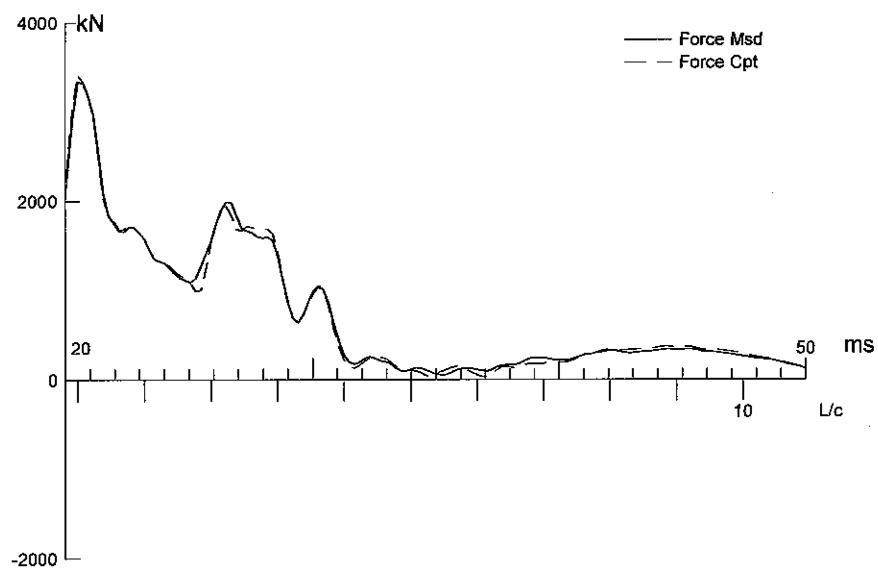
PILE PROFILE AND PILE MODEL

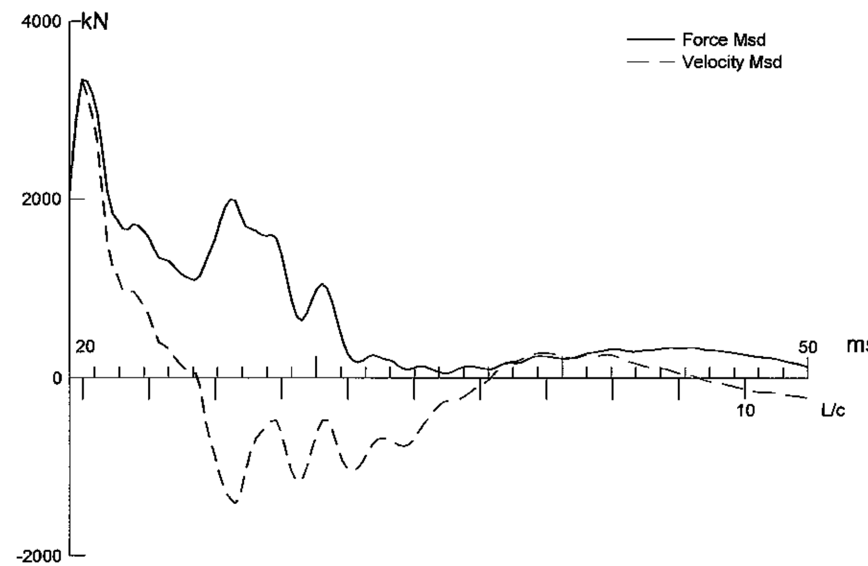
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	167.00	206842.7	77.287	1.254
14.00	167.00	206842.7	77.287	1.254

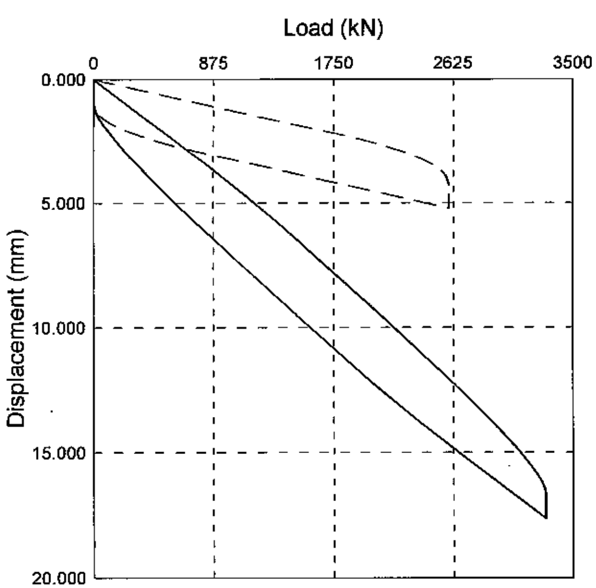
Toe Area 0.098 m^2

Top Segment Length 1.00 m, Top Impedance 674.26 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 5.5 ms

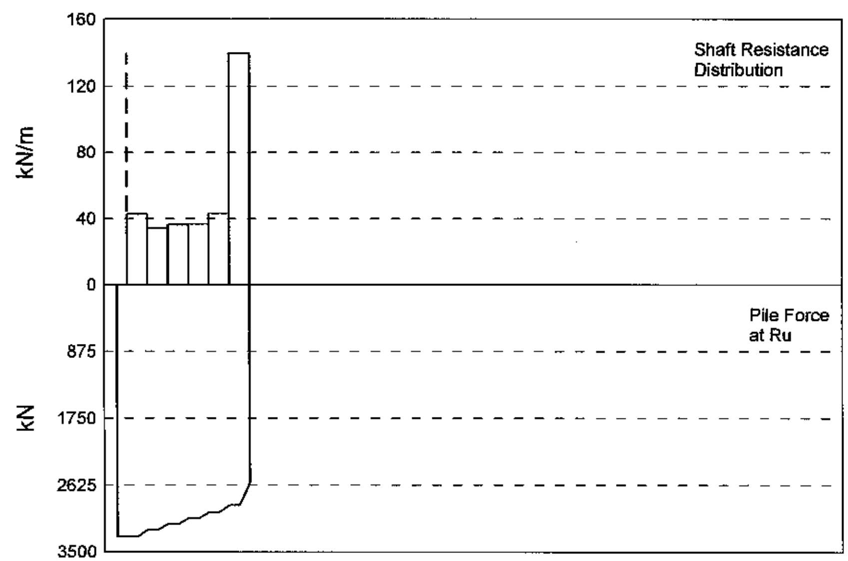






Pile Top
Bottom

Ru = 3300.1 kN
Rs = 707.4 kN
Rb = 2592.7 kN
Dy = 16.6 mm
Dx = 17.6 mm



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CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3300.1; along Shaft 707.4; at Toe 2592.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				3300.1				
1	3.2	1.8	90.7	3209.4	90.7	50.82	40.53	0.766
2	5.3	3.9	72.8	3136.6	163.5	34.29	27.34	0.766
3	7.4	6.0	77.4	3059.2	240.9	36.46	29.07	0.766
4	9.6	8.2	78.2	2981.0	319.1	36.83	29.37	0.766
5	11.7	10.3	91.8	2889.2	410.9	43.24	34.48	0.766
6	13.8	12.4	296.5	2592.7	707.4	139.66	111.37	0.766
Avg. Shaft			117.9			57.05	45.49	0.766
Toe			2592.7				26380.21	0.559

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	3.178	3.236
Case Damping Factor		0.804	2.149
Damping Type			Smith
Unloading Quake	(% of loading quake)	93	106
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	18	
Resistance Gap (included in Toe Quake)	(mm)		0.235
Soil Plug Weight	(kN)		0.30

CAPWAP match quality = 1.40 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.243 mm; blow count = 4107 b/m

max. Top Comp. Stress = 207.3 MPa (T= 20.9 ms, max= 1.128 x Top)
max. Comp. Stress = 233.8 MPa (Z= 13.8 m, T= 23.8 ms)
max. Tens. Stress = -16.76 MPa (Z= 13.8 m, T= 36.7 ms)
max. Energy (EMX) = 25.22 kJ; max. Measured Top Displ. (DMX)=11.15 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.1	3461.1	-90.1	207.3	-5.40	25.22	4.8	11.047
2	2.1	3538.8	-112.4	211.9	-6.73	24.99	4.6	10.675
3	3.2	3634.8	-150.7	217.7	-9.03	24.73	4.5	10.255
4	4.2	3307.3	-105.9	198.0	-6.34	21.83	4.4	9.814
5	5.3	3389.1	-113.5	202.9	-6.80	21.42	4.3	9.327
6	6.4	3155.8	-123.0	189.0	-7.37	19.12	4.2	8.776
7	7.4	3236.7	-200.8	193.8	-12.03	18.44	4.0	8.135
8	8.5	3001.4	-218.5	179.7	-13.08	15.99	3.9	7.456
9	9.6	3083.0	-234.6	184.6	-14.05	15.25	3.8	6.777
10	10.6	2866.5	-214.7	171.6	-12.85	13.02	3.7	6.064
11	11.7	3040.9	-243.5	182.1	-14.58	11.93	3.7	5.250
12	12.7	3532.7	-253.2	211.5	-15.16	9.33	3.4	4.295
13	13.8	3904.4	-280.0	233.8	-16.76	7.01	2.4	3.331
Absolute	13.8			233.8			(T =	23.8 ms)
	13.8				-16.76		(T =	36.7 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4568.5	4342.0	4115.4	3888.8	3662.3	3435.7	3209.1	2982.5	2756.0	2529.4
RX	4696.2	4538.6	4392.8	4246.9	4101.8	3967.9	3850.9	3735.3	3647.2	3559.1
RU	4720.4	4509.0	4297.6	4086.2	3874.8	3663.4	3452.1	3240.7	3029.3	2817.9

RAU = 1939.2 (kN); RA2 = 3647.6 (kN)

Current CAPWAP Ru = 3300.1 (kN); Corresponding J(RP)= 0.56;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.12	20.72	3452.0	3382.3	3458.2	11.153	0.698	1.000	25.7	4232.7

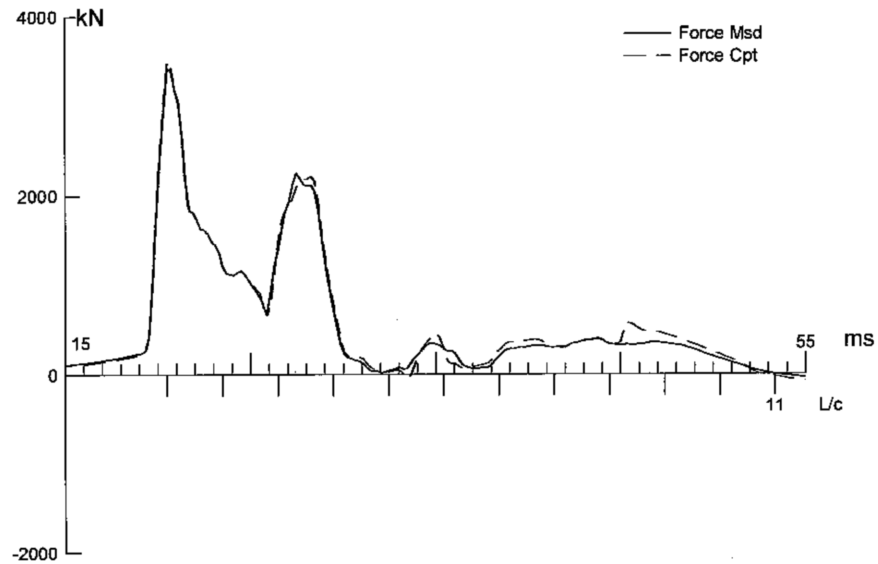
PILE PROFILE AND PILE MODEL

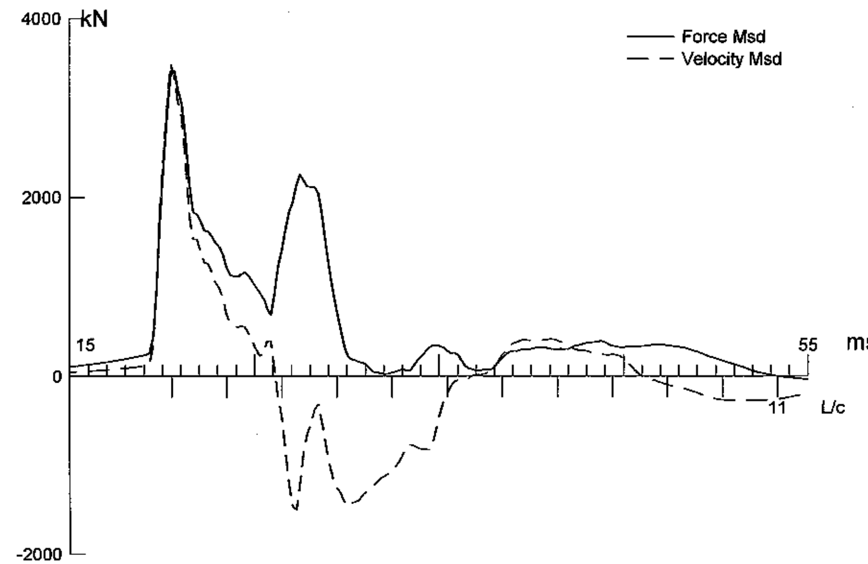
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	167.00	206842.7	77.287	1.254
13.80	167.00	206842.7	77.287	1.254

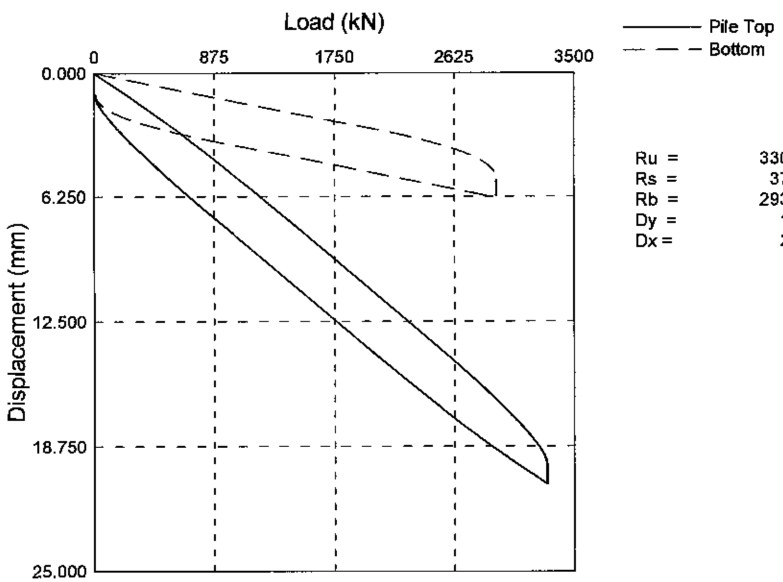
Toe Area 0.098 m^2

Top Segment Length 1.06 m, Top Impedance 674.26 kN/m/s

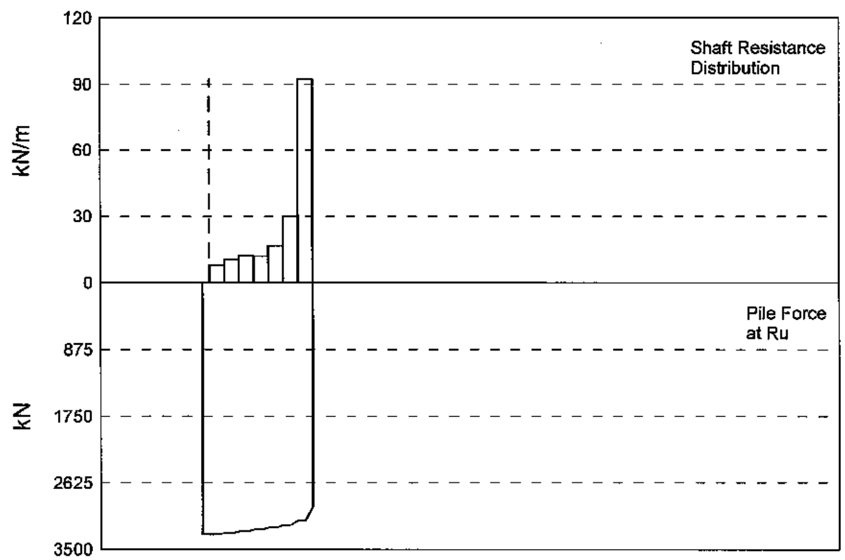
Pile Damping 1.0 %, Time Incr 0.207 ms, Wave Speed 5123.0 m/s, 2L/c 5.4 ms







Ru = 3301.9 kN
Rs = 370.3 kN
Rb = 2931.6 kN
Dy = 19.6 mm
Dx = 20.6 mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3301.9; along Shaft 370.3; at Toe 2931.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				3301.9					
1	3.1	1.1	16.1	3285.8	16.1	15.19	12.17	1.313	1.264
2	5.1	3.1	21.4	3264.4	37.5	10.50	8.41	1.313	1.264
3	7.1	5.1	24.7	3239.7	62.2	12.11	9.70	1.313	1.264
4	9.2	7.2	24.3	3215.4	86.5	11.91	9.54	1.313	1.264
5	11.2	9.2	34.1	3181.3	120.6	16.71	13.39	1.313	1.264
6	13.3	11.3	61.1	3120.2	181.7	29.95	24.00	1.313	1.264
7	15.3	13.3	188.6	2931.6	370.3	92.45	74.08	1.313	1.176
Avg. Shaft			52.9			27.84	22.31	1.313	1.219
Toe			2931.6				30115.88	0.332	4.078

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.721	1.443
Damping Type			Smith
Unloading Quake	(% of loading quake)	78	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	35	
Resistance Gap (included in Toe Quake) (mm)			0.714

CAPWAP match quality = 2.15 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.200 mm; blow count = 5008 b/m

max. Top Comp. Stress = 211.0 MPa (T= 20.9 ms, max= 1.188 x Top)
max. Comp. Stress = 250.5 MPa (Z= 15.3 m, T= 24.3 ms)
max. Tens. Stress = -26.23 MPa (Z= 11.2 m, T= 33.8 ms)
max. Energy (EMX) = 28.35 kJ; max. Measured Top Displ. (DMX)=13.47 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3523.4	-90.3	211.0	-5.41	28.35	5.0	13.310
2	2.0	3546.8	-175.4	212.4	-10.50	28.15	5.0	12.974
3	3.1	3577.6	-227.8	214.2	-13.64	27.92	4.9	12.619
4	4.1	3488.8	-249.1	208.9	-14.92	26.71	4.8	12.195
5	5.1	3525.6	-280.4	211.1	-16.79	26.31	4.8	11.704
6	6.1	3405.2	-294.1	203.9	-17.61	24.76	4.7	11.224
7	7.1	3443.6	-335.5	206.2	-20.09	24.36	4.6	10.733
8	8.2	3303.0	-381.5	197.8	-22.84	22.57	4.6	10.184
9	9.2	3345.7	-426.0	200.3	-25.51	21.85	4.5	9.512
10	10.2	3230.5	-406.8	193.4	-24.36	19.88	4.4	8.797
11	11.2	3314.5	-438.1	198.5	-26.23	18.84	4.3	8.023
12	12.2	3136.4	-371.8	187.8	-22.26	16.41	4.2	7.193
13	13.3	3274.0	-392.3	196.0	-23.49	15.09	4.6	6.313
14	14.3	3776.8	-289.0	226.2	-17.30	11.65	4.1	5.351
15	15.3	4184.2	-278.1	250.5	-16.65	8.63	2.9	4.345
Absolute	15.3			250.5			(T =	24.3 ms)
	11.2				-26.23		(T =	33.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4446.9	4188.1	3929.3	3670.6	3411.8	3153.0	2894.2	2635.5	2376.7	2117.9
RX	4729.3	4599.3	4469.4	4339.4	4220.5	4114.1	4007.7	3911.6	3837.8	3764.0
RU	4560.0	4312.5	4065.0	3817.6	3570.1	3322.6	3075.2	2827.7	2580.2	2332.8

RAU = 1917.3 (kN); RA2 = 3655.9 (kN)

Current CAPWAP Ru = 3301.9 (kN); Corresponding J(RP)= 0.44;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.28	20.71	3562.9	3471.8	3532.8	13.466	0.851	1.000	28.7	3961.9

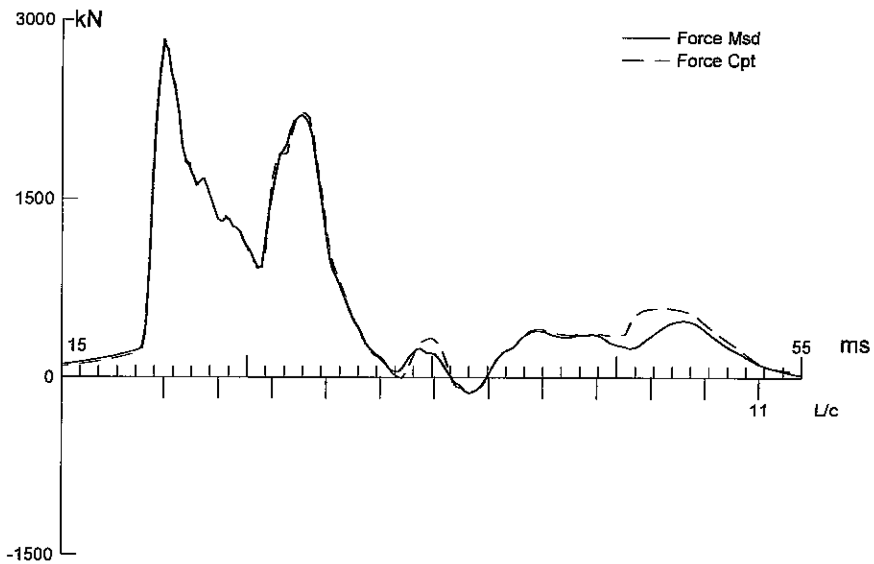
PILE PROFILE AND PILE MODEL

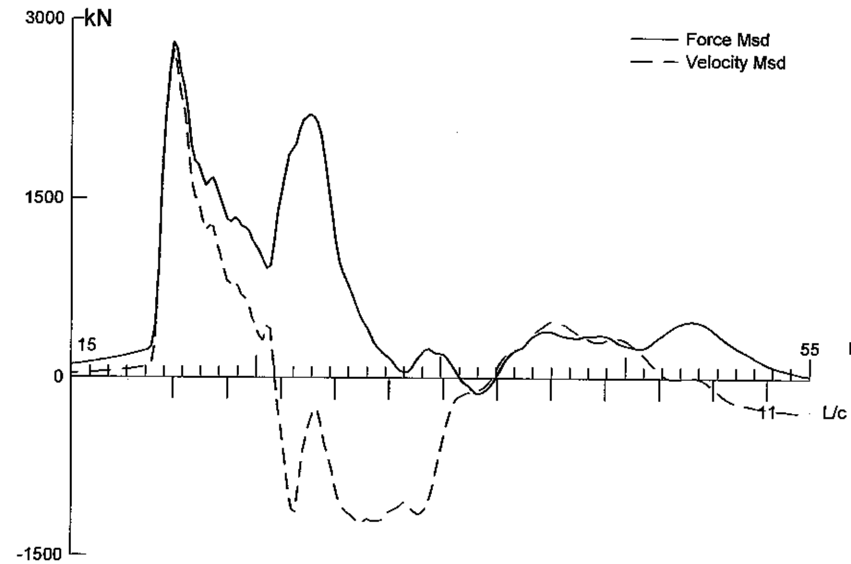
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	167.00	206842.7	77.287	1.248
15.30	167.00	206842.7	77.287	1.248

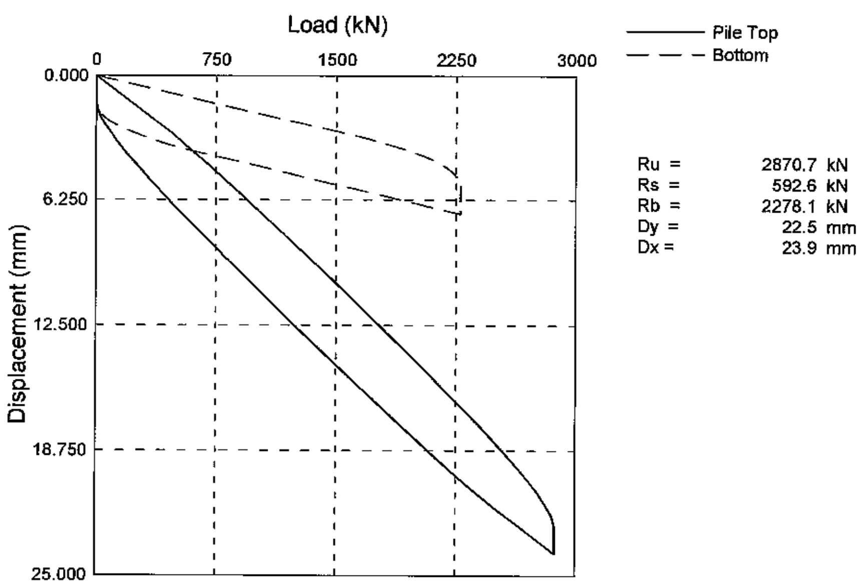
Toe Area 0.097 m^2

Top Segment Length 1.02 m, Top Impedance 674.26 kN/m/s

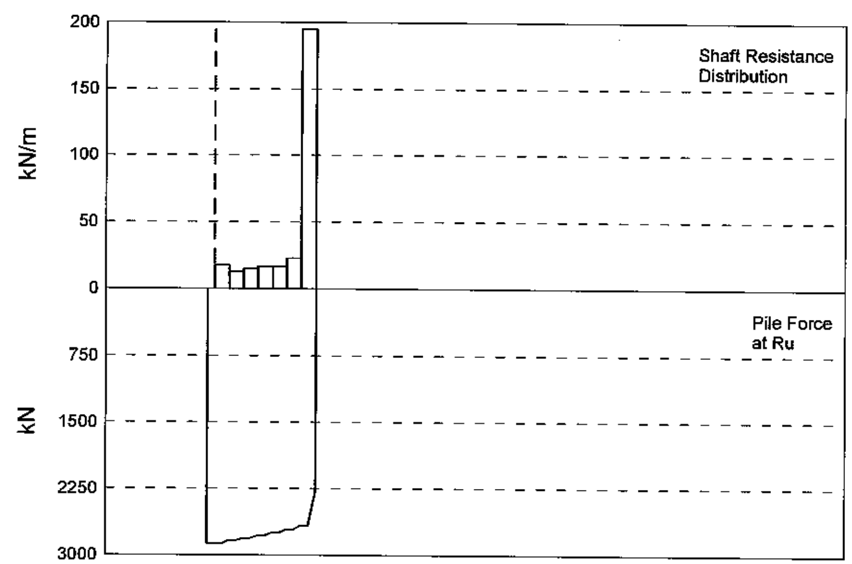
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 5123.0 m/s, 2L/c 6.0 ms







Ru =	2870.7	kN
Rs =	592.6	kN
Rb =	2278.1	kN
Dy =	22.5	mm
Dx =	23.9	mm



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RSTRK; Blow: 5
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Test: 01-Aug-2012 10:45:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2870.7; along Shaft 592.6; at Toe 2278.1 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2870.7				
1	3.0	1.3	35.0	2835.7	35.0	26.92	22.03	0.731
2	5.0	3.3	25.0	2810.7	60.0	12.50	10.23	0.731
3	7.0	5.3	30.0	2780.7	90.0	15.00	12.27	0.731
4	9.0	7.3	33.3	2747.4	123.3	16.65	13.63	0.731
5	11.0	9.3	33.3	2714.1	156.6	16.65	13.63	0.731
6	13.0	11.3	46.0	2668.1	202.6	23.00	18.82	0.731
7	15.0	13.3	390.0	2278.1	592.6	195.00	159.57	0.731
Avg. Shaft			84.7			44.56	36.46	0.731
Toe			2278.1				24410.66	0.348

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.044	4.268
Case Damping Factor		0.902	1.650
Damping Type			Smith
Unloading Quake	(% of loading quake)	96	42
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	24	
Resistance Gap (included in Toe Quake) (mm)			0.697

CAPWAP match quality = 2.49 (Wave Up Match) ; RSA = 0
Observed: final set = 1.400 mm; blow count = 714 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 240.2 MPa (T= 20.9 ms, max= 1.225 x Top)
max. Comp. Stress = 294.3 MPa (Z= 15.0 m, T= 23.8 ms)
max. Tens. Stress = -35.77 MPa (Z= 11.0 m, T= 35.9 ms)
max. Energy (EMX) = 31.18 kJ; max. Measured Top Displ. (DMX)=17.16 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2858.9	-136.8	240.2	-11.50	31.18	5.6	16.980
2	2.0	2900.6	-152.4	243.8	-12.81	30.82	5.5	16.431
3	3.0	2930.3	-170.6	246.2	-14.33	30.37	5.4	15.855
4	4.0	2786.7	-275.3	234.2	-23.14	28.06	5.4	15.191
5	5.0	2814.4	-352.9	236.5	-29.65	27.37	5.3	14.435
6	6.0	2727.3	-396.3	229.2	-33.30	25.44	5.2	13.650
7	7.0	2758.7	-421.9	231.8	-35.45	24.61	5.1	12.827
8	8.0	2653.2	-405.2	223.0	-34.05	22.47	5.1	11.998
9	9.0	2686.2	-419.1	225.7	-35.22	21.40	5.0	11.041
10	10.0	2662.4	-414.1	223.7	-34.80	18.87	4.9	10.024
11	11.0	2806.3	-425.6	235.8	-35.77	17.50	4.8	8.962
12	12.0	2763.5	-400.5	232.2	-33.65	15.10	4.7	7.878
13	13.0	2924.4	-394.9	245.7	-33.18	13.46	4.9	6.728
14	14.0	3241.9	-353.4	272.4	-29.69	10.77	4.4	5.544
15	15.0	3502.4	-348.7	294.3	-29.31	6.60	3.2	4.406
Absolute	15.0			294.3			(T =	23.8 ms)
	11.0				-35.77		(T =	35.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3858.2	3679.7	3501.3	3322.9	3144.4	2966.0	2787.6	2609.1	2430.7	2252.3
RX	3956.4	3813.7	3727.3	3645.3	3563.2	3481.2	3399.2	3327.7	3258.6	3189.6
RU	3927.4	3755.9	3584.4	3412.9	3241.4	3069.9	2898.4	2726.9	2555.4	2383.9

RAU = 1751.2 (kN); RA2 = 3051.3 (kN)

Current CAPWAP Ru = 2870.7 (kN); Corresponding J(RP)= 0.55;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.83	20.69	2799.1	2843.4	2847.4	17.165	1.297	1.400	31.6	3406.4

PILE PROFILE AND PILE MODEL

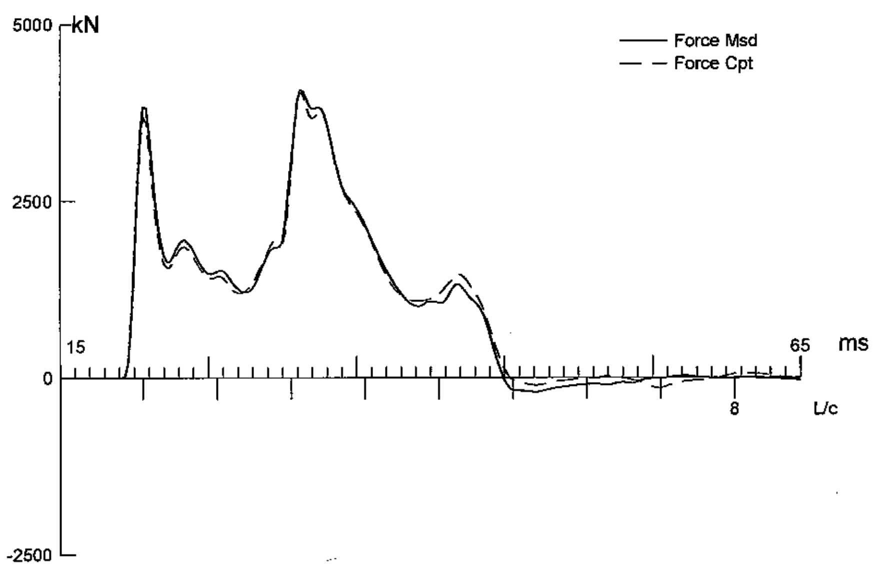
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	119.00	206842.7	77.287	1.222
15.00	119.00	206842.7	77.287	1.222

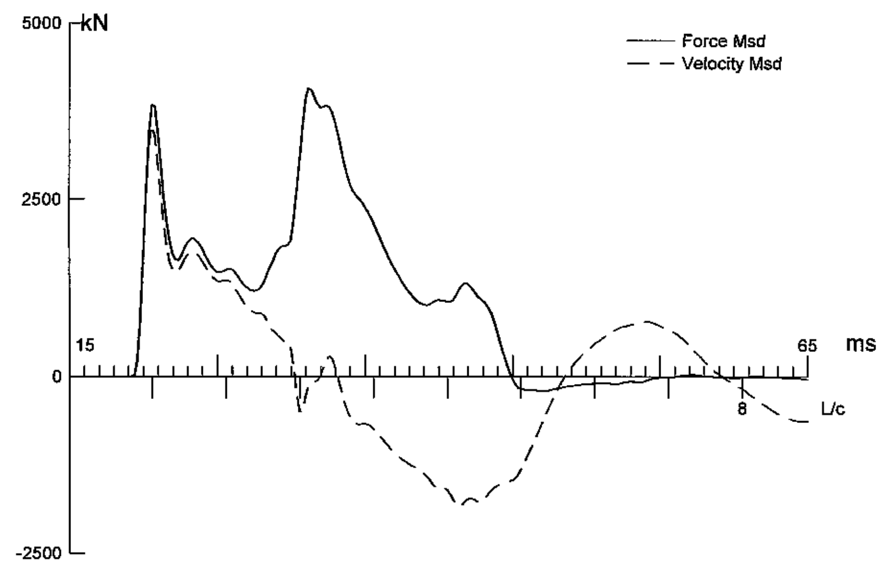
Toe Area 0.093 m^2

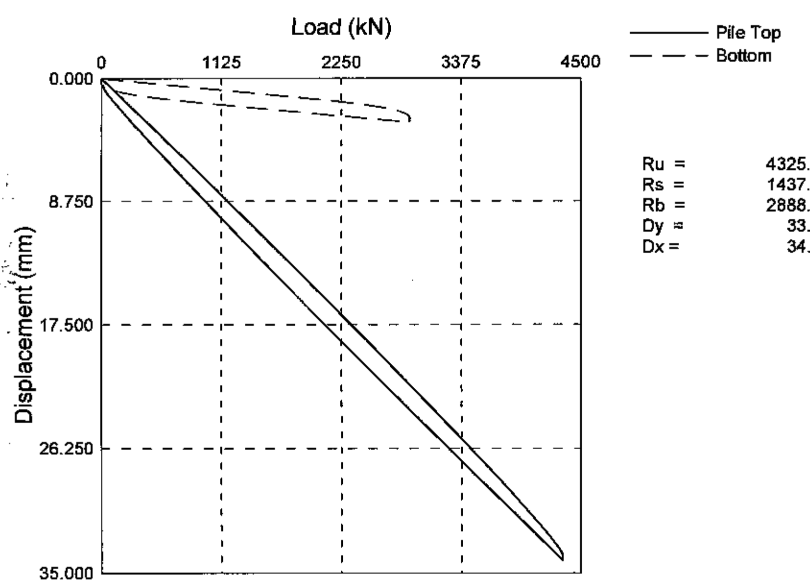
Top Segment Length 1.00 m, Top Impedance 480.46 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 5.9 ms

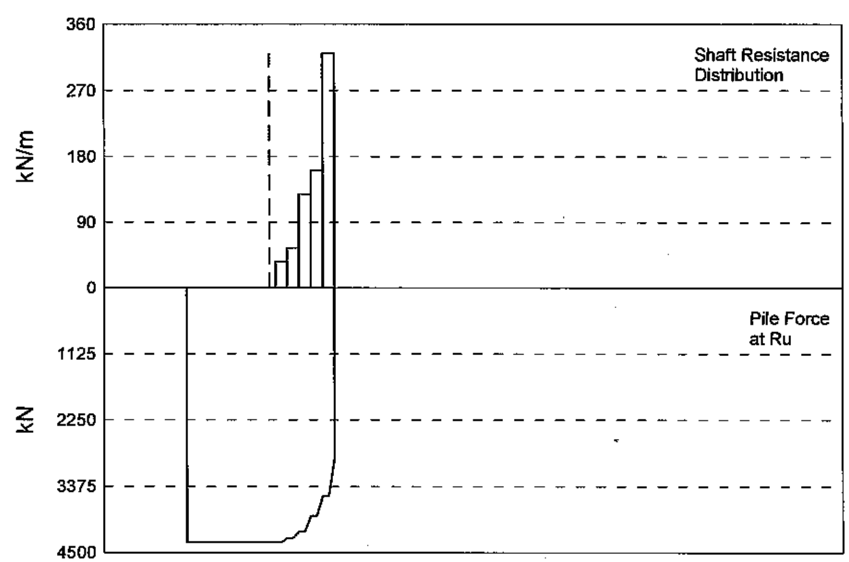
Site 2







Ru = 4325.0 kN
Rs = 1437.1 kN
Rb = 2888.0 kN
Dy = 33.7 mm
Dx = 34.1 mm



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RSTRK; Blow: 14 CAPWAP(R) 2006-3
[redacted] OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 4325.0; along Shaft 1437.1; at Toe 2888.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				4325.0					
1	17.4	2.3	74.3	4250.8	74.3	32.17	22.22	0.523	3.163
2	19.5	4.4	113.4	4137.4	187.6	55.36	38.23	0.523	3.163
3	21.5	6.4	263.6	3873.8	451.3	128.73	88.90	0.523	3.163
4	23.6	8.5	329.5	3544.2	780.8	160.90	111.12	0.523	3.163
5	25.6	10.5	656.3	2888.0	1437.1	320.44	221.30	0.523	2.099
Avg. Shaft			287.4			136.86	94.52	0.523	2.677
Toe			2888.0				22058.52	0.709	2.106

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.108	3.017
Damping Type			Smith
Unloading Quake	(% of loading quake)	100	91
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			0.123
Soil Plug Weight	(kN)		0.49

CAPWAP match quality = 3.34 (Wave Up Match) ; RSA = 0
Observed: final set = 0.400 mm; blow count = 2500 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 230.6 MPa (T= 31.6 ms, max= 1.121 x Top)
max. Comp. Stress = 258.4 MPa (Z= 25.6 m, T= 26.0 ms)
max. Tens. Stress = -61.50 MPa (Z= 17.4 m, T= 49.0 ms)
max. Energy (EMX) = 45.45 kJ; max. Measured Top Displ. (DMX)=22.18 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3873.8	-194.6	230.6	-11.59	45.45	5.4	22.301
2	2.0	3754.8	-288.4	223.5	-17.17	44.63	5.4	21.606
4	4.1	3703.8	-463.5	220.5	-27.59	42.98	5.4	20.209
6	6.1	3677.9	-626.7	218.9	-37.31	41.50	5.4	18.894
8	8.2	3670.9	-754.2	218.5	-44.89	40.12	5.4	17.612
9	9.2	3667.4	-808.2	218.3	-48.11	39.32	5.4	16.936
10	10.2	3663.8	-857.6	218.1	-51.05	38.34	5.4	16.197
11	11.3	3660.2	-898.6	217.9	-53.49	37.25	5.4	15.384
12	12.3	3656.5	-928.0	217.6	-55.24	36.03	5.4	14.534
13	13.3	3652.8	-955.6	217.4	-56.88	34.81	5.4	13.691
14	14.3	3649.6	-992.7	217.2	-59.09	33.62	5.4	12.844
15	15.4	3665.4	-1011.1	218.2	-60.19	32.30	5.3	11.945
16	16.4	3730.7	-1013.9	222.1	-60.35	30.74	5.3	11.000
17	17.4	3814.2	-1033.1	227.0	-61.50	28.89	5.1	9.975
18	18.4	3842.9	-973.8	228.7	-57.97	25.21	5.0	8.919
19	19.5	3879.5	-1007.4	230.9	-59.96	23.36	4.8	7.870
20	20.5	3742.4	-891.2	222.8	-53.05	19.84	4.6	6.940
21	21.5	3820.9	-931.7	227.4	-55.46	18.63	4.3	6.086
22	22.5	3356.4	-692.9	199.8	-41.25	13.93	4.0	5.263
23	23.6	3580.9	-743.5	213.1	-44.25	12.32	3.9	4.274
24	24.6	3840.2	-598.5	228.6	-35.63	7.59	3.6	3.224
25	25.6	4341.9	-617.8	258.4	-36.78	4.68	2.3	2.110
Absolute	25.6			258.4			(T =	26.0 ms)
	17.4				-61.50		(T =	49.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	5695.9	5514.5	5333.1	5151.7	4970.3	4788.8	4607.4	4426.0	4244.6	4063.2
RX	5695.9	5514.5	5333.1	5164.4	5005.9	4847.5	4706.3	4570.9	4435.5	4318.4
RU	5738.2	5561.0	5383.8	5206.7	5029.5	4852.3	4675.1	4497.9	4320.8	4143.6

RAU = 3991.5 (kN); RA2 = 4375.7 (kN)

Current CAPWAP Ru = 4325.0 (kN); Corresponding J(RP)= 0.76; J(RX) = 0.89

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.28	20.79	3582.5	3927.5	4095.8	22.183	0.397	0.400	46.5	4118.6

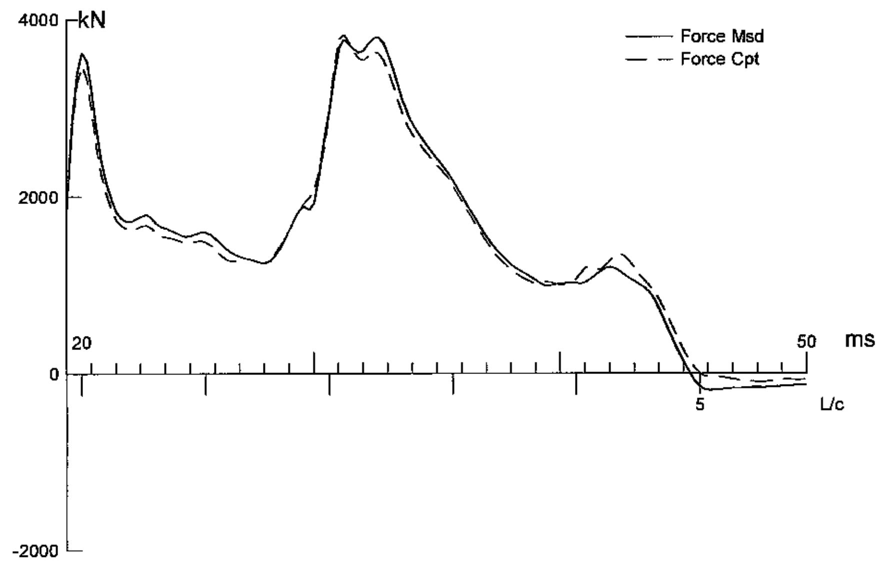
PILE PROFILE AND PILE MODEL

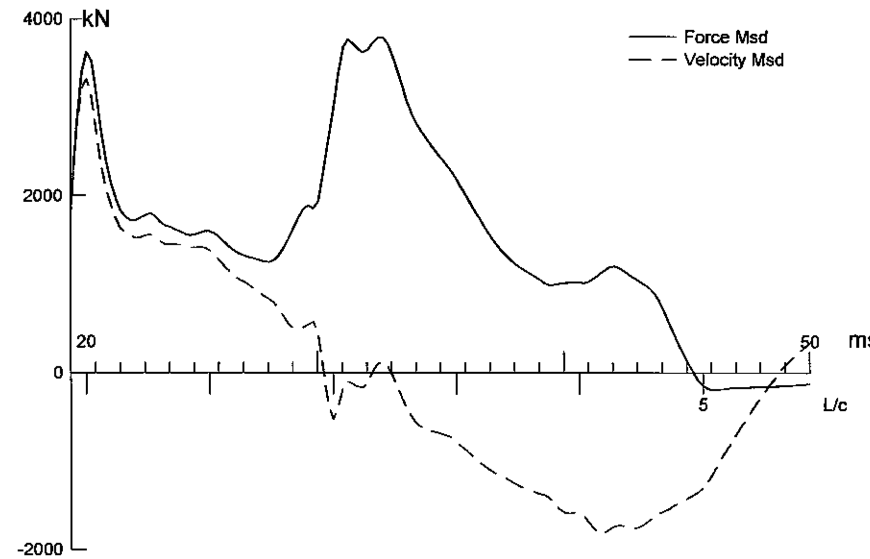
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
25.60	168.00	206842.7	77.287	1.448

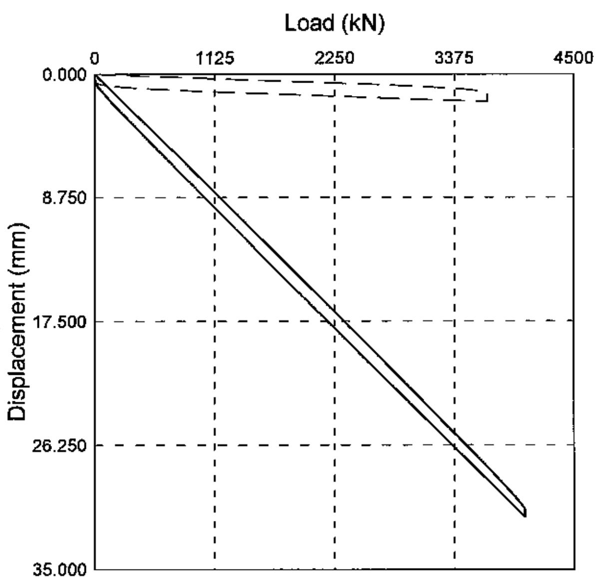
Toe Area 0.131 m^2

Top Segment Length 1.02 m, Top Impedance 678.30 kN/m/s

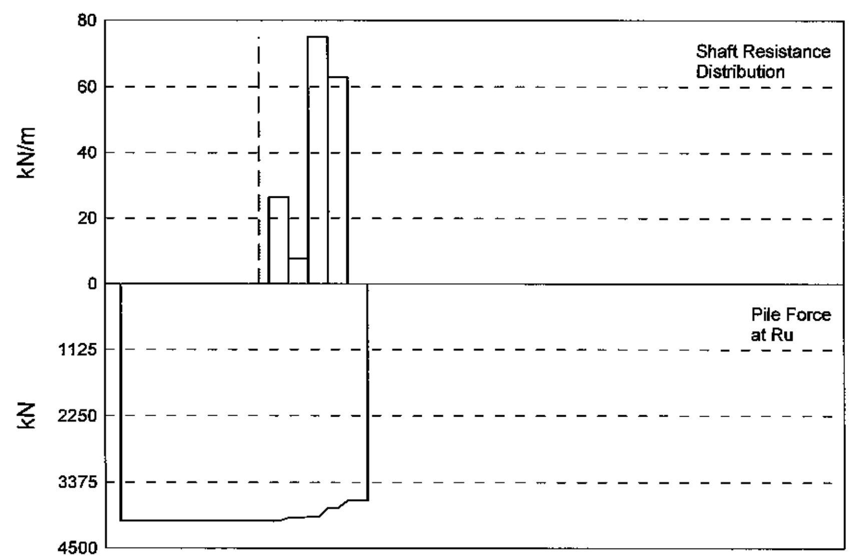
Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 5123.0 m/s, 2L/c 10.0 ms







Ru = 4036.5 kN
Rs = 353.0 kN
Rb = 3683.5 kN
Dy = 31.1 mm
Dx = 31.3 mm



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Steel HP360 x 132; Blow: 14
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Test: 09-Feb-2012 10:07:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 4036.5; along Shaft 353.0; at Toe 3683.5 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				4036.5					
1	17.4	2.3	54.1	3982.4	54.1	23.59	16.29	1.153	3.670
2	19.5	4.3	16.1	3966.3	70.2	7.86	5.43	1.153	3.671
3	21.5	6.4	153.7	3812.6	223.9	74.92	51.74	1.153	3.671
4	23.6	8.4	129.1	3683.5	353.0	62.92	43.45	1.153	3.156
5	25.6	10.5	0.0	3683.5	353.0	0.00	0.00	0.000	0.010
Avg. Shaft			70.6			33.62	23.22	1.153	3.483
Toe			3683.5				28134.56	0.555	1.013

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.600	3.014
Damping Type			Smith
Unloading Quake	(% of loading quake)	94	106
Reloading Level	(% of Ru)	100	100
Soil Plug Weight	(kN)		2.41

CAPWAP match quality = 3.72 (Wave Up Match) ; RSA = 0
Observed: final set = 0.200 mm; blow count = 5000 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 221.6 MPa (T= 31.6 ms, max= 1.195 x Top)
max. Comp. Stress = 264.9 MPa (Z= 25.6 m, T= 26.0 ms)
max. Tens. Stress = -58.15 MPa (Z= 17.4 m, T= 49.5 ms)
max. Energy (EMX) = 43.12 kJ; max. Measured Top Displ. (DMX)=21.70 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3723.3	-175.9	221.6	-10.47	43.12	5.1	21.969
2	2.1	3604.8	-263.1	214.6	-15.66	42.22	5.1	21.249
4	4.1	3520.7	-462.2	209.6	-27.51	40.83	5.1	19.959
6	6.2	3463.9	-632.5	206.2	-37.65	39.52	5.1	18.722
8	8.2	3459.5	-752.5	205.9	-44.79	37.98	5.1	17.384
9	9.2	3457.5	-814.1	205.8	-48.46	37.13	5.1	16.665
10	10.3	3455.3	-864.7	205.7	-51.47	36.16	5.1	15.900
11	11.3	3453.0	-895.8	205.5	-53.32	35.02	5.1	15.096
12	12.3	3450.8	-907.2	205.4	-54.00	33.75	5.1	14.229
13	13.3	3489.7	-928.3	207.7	-55.26	32.40	5.1	13.334
14	14.4	3536.9	-950.3	210.5	-56.57	31.07	5.1	12.447
15	15.4	3517.9	-950.0	209.4	-56.55	29.73	5.0	11.552
16	16.4	3563.3	-941.6	212.1	-56.05	28.33	4.9	10.637
17	17.4	3616.6	-976.9	215.3	-58.15	26.87	4.8	9.701
18	18.5	3578.9	-909.7	213.0	-54.15	23.09	4.8	8.748
19	19.5	3637.8	-923.9	216.5	-55.00	21.54	4.6	7.797
20	20.5	3619.7	-923.5	215.5	-54.97	19.56	4.3	6.874
21	21.5	3686.1	-972.5	219.4	-57.89	18.08	4.1	5.925
22	22.6	3576.7	-790.9	212.9	-47.08	12.96	3.8	4.988
23	23.6	3727.7	-820.6	221.9	-48.85	11.14	3.5	3.945
24	24.6	3979.9	-719.0	236.9	-42.80	7.20	2.9	2.828
25	25.6	4449.8	-727.2	264.9	-43.29	5.76	1.7	1.643
Absolute	25.6			264.9			(T =	26.0 ms)
	17.4				-58.15		(T =	49.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	5316.9	5140.7	4964.5	4788.3	4612.1	4435.9	4259.8	4083.6	3907.4	3731.2
RX	5328.0	5164.4	5000.8	4837.2	4673.6	4510.0	4363.7	4220.2	4087.3	3964.4
RU	5366.2	5195.0	5023.8	4852.5	4681.3	4510.0	4338.8	4167.5	3996.3	3825.0

RAU = 3723.0 (kN); RA2 = 4128.4 (kN)

Current CAPWAP Ru = 4036.5 (kN); Corresponding J(RP)= 0.73; J(RX) = 0.84

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.99	20.83	3384.8	3693.9	3806.2	21.698	0.196	0.200	44.3	4050.6

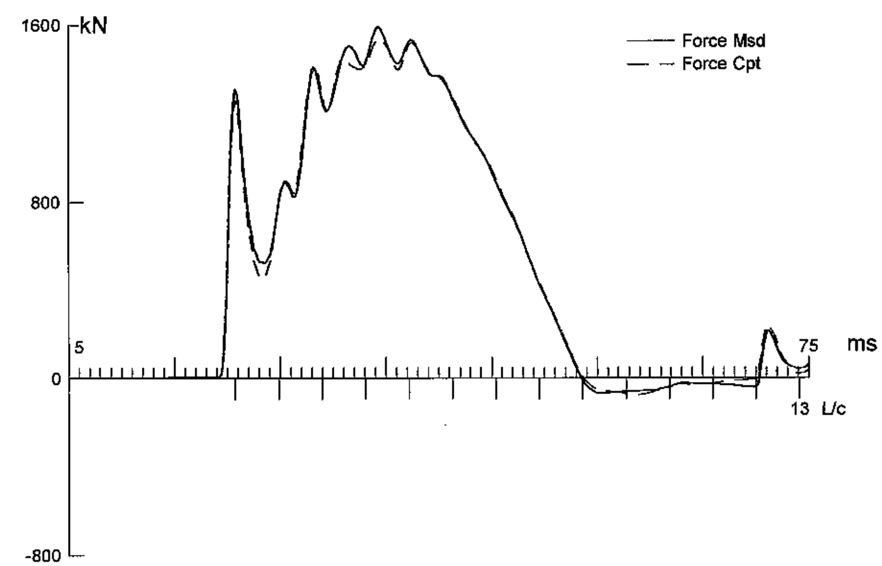
PILE PROFILE AND PILE MODEL

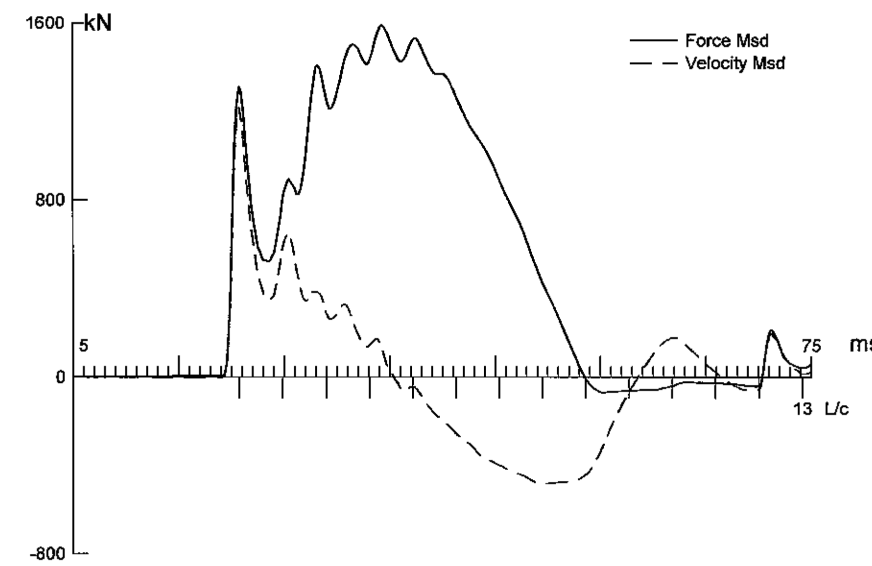
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
25.65	168.00	206842.7	77.287	1.448

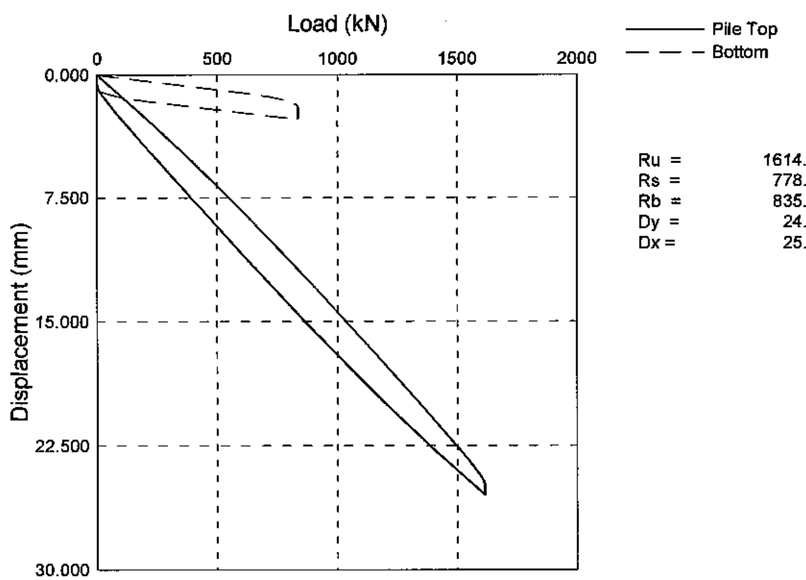
Toe Area 0.131 m^2

Top Segment Length 1.03 m, Top Impedance 678.30 kN/m/s

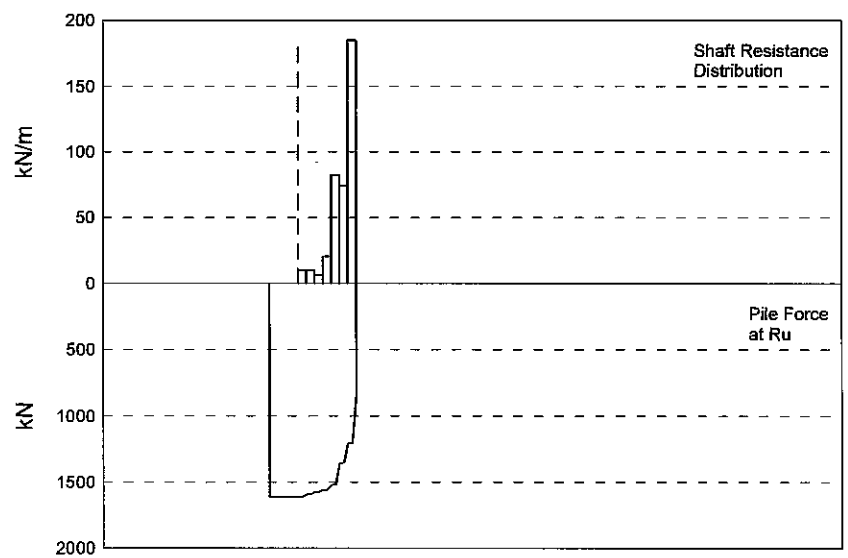
Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 5123.0 m/s, 2L/c 10.0 ms







Ru = 1614.2 kN
Rs = 778.4 kN
Rb = 835.8 kN
Dy = 24.9 mm
Dx = 25.5 mm



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Test: 17-Feb-2012 13:45:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1614.2; along Shaft 778.4; at Toe 835.8 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				1614.2				
1	9.0	2.0	20.4	1593.8	20.4	10.33	12.57	1.313
2	11.0	4.0	19.8	1574.1	40.1	9.86	12.00	1.313
3	13.0	6.0	13.2	1560.9	53.3	6.58	8.00	1.313
4	15.0	8.0	41.2	1519.7	94.5	20.55	25.00	1.313
5	17.0	10.0	164.8	1354.9	259.3	82.20	100.00	1.313
6	19.0	12.0	148.3	1206.6	407.6	73.99	90.01	1.313
7	21.0	14.0	370.8	835.8	778.4	184.94	224.99	1.313
Avg. Shaft			111.2			55.60	67.64	1.313
Toe			835.8				19792.55	0.168

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.627	1.496
Case Damping Factor		3.701	0.508
Unloading Quake	(% of loading quake)	100	39
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	77	
Resistance Gap (included in Toe Quake)	(mm)		0.364
Soil Plug Weight	(kN)		1.49

CAPWAP match quality = 2.30 (Wave Up Match) ; RSA = 0
Observed: final set = 0.600 mm; blow count = 1667 b/m
Computed: final set = 0.916 mm; blow count = 1092 b/m

max. Top Comp. Stress = 230.1 MPa (T= 34.6 ms, max= 1.050 x Top)
max. Comp. Stress = 241.6 MPa (Z= 9.0 m, T= 36.0 ms)
max. Tens. Stress = -37.00 MPa (Z= 15.0 m, T= 57.9 ms)
max. Energy (EMX) = 23.63 kJ; max. Measured Top Displ. (DMX)=23.12 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	1573.6	-91.7	230.1	-13.41	23.63	4.6	23.196
2	2.0	1599.6	-108.8	233.9	-15.91	22.89	4.6	22.171
3	3.0	1608.6	-128.4	235.2	-18.77	22.12	4.6	21.127
4	4.0	1599.0	-149.9	233.8	-21.91	21.33	4.6	20.061
5	5.0	1582.7	-172.1	231.4	-25.16	20.51	4.6	18.977
6	6.0	1598.8	-193.3	233.7	-28.26	19.65	4.6	17.875
7	7.0	1622.5	-213.1	237.2	-31.15	18.78	4.5	16.756
8	8.0	1641.0	-231.6	239.9	-33.86	17.89	4.4	15.630
9	9.0	1652.4	-249.4	241.6	-36.46	17.00	4.2	14.508
10	10.0	1630.4	-236.0	238.4	-34.50	15.11	4.1	13.404
11	11.0	1628.4	-250.1	238.1	-36.56	14.25	4.0	12.298
12	12.0	1601.5	-236.9	234.1	-34.63	12.56	4.0	11.204
13	13.0	1604.3	-248.7	234.5	-36.36	11.72	3.8	10.108
14	14.0	1589.2	-243.5	232.3	-35.60	10.44	3.6	9.017
15	15.0	1592.7	-253.1	232.9	-37.00	9.61	3.2	7.925
16	16.0	1546.7	-215.2	226.1	-31.47	7.87	2.7	6.861
17	17.0	1550.5	-222.2	226.7	-32.49	7.09	2.3	5.803
18	18.0	1410.0	-96.3	206.1	-14.08	4.48	2.0	4.815
19	19.0	1419.1	-101.9	207.5	-14.90	3.82	1.6	3.841
20	20.0	1301.5	0.0	190.3	0.00	2.42	1.0	2.970
21	21.0	1300.0	-0.1	190.1	-0.01	1.15	1.1	2.127
Absolute	9.0			241.6			(T =	36.0 ms)
	15.0				-37.00		(T =	57.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1760.9	1682.5	1604.2	1525.8	1447.5	1369.1	1290.8	1212.4	1134.0	1055.7
RX	1760.9	1682.5	1640.5	1627.8	1615.3	1603.5	1593.8	1584.1	1577.5	1572.7
RU	1760.9	1682.5	1604.2	1525.8	1447.5	1369.1	1290.8	1212.4	1134.0	1055.7

RAU = 1065.2 (kN); RA2 = 1126.3 (kN)

Current CAPWAP Ru = 1614.2 (kN); Corresponding J(RP)= 0.19; J(RX) = 0.41

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.44	20.94	1227.5	1317.0	1592.1	23.123	0.597	0.600	23.8	2008.0

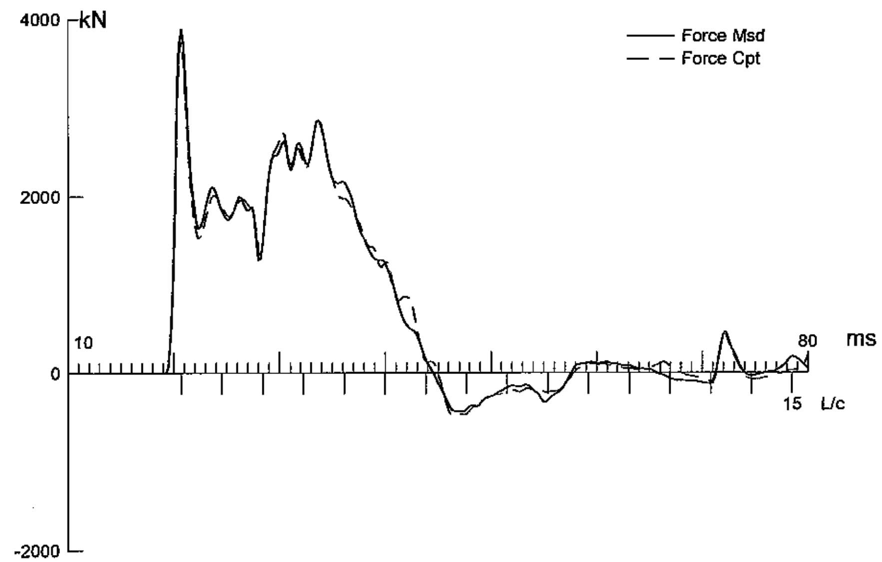
PILE PROFILE AND PILE MODEL

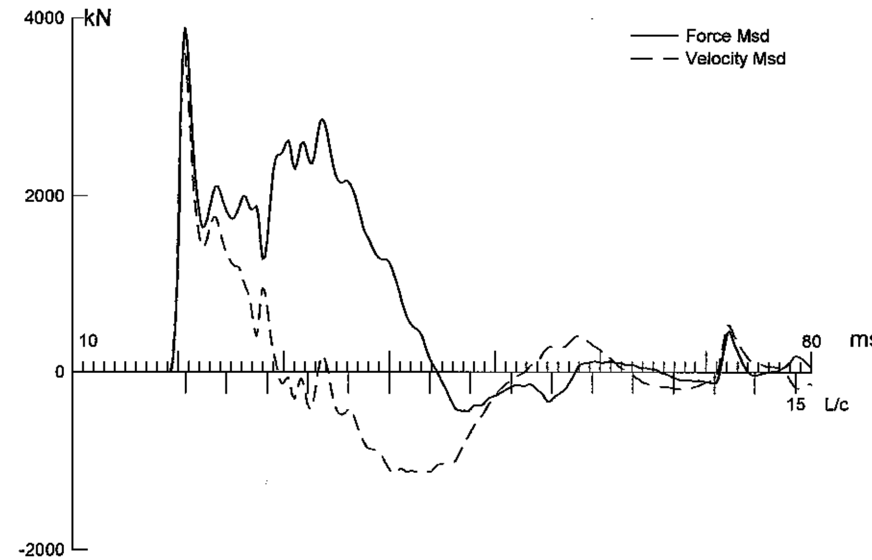
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	68.40	206842.7	77.287	0.822
21.05	68.40	206842.7	77.287	0.822

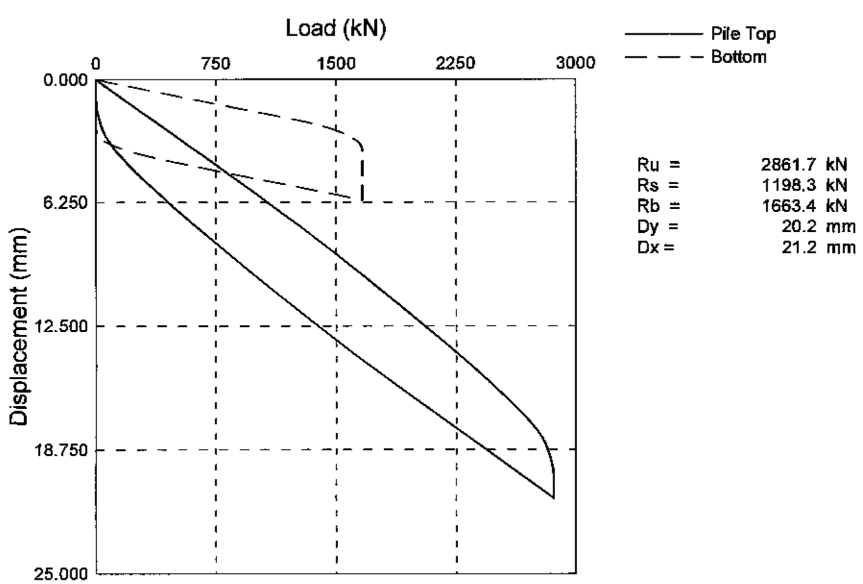
Toe Area 0.042 m^2

Top Segment Length 1.00 m, Top Impedance 276.17 kN/m/s

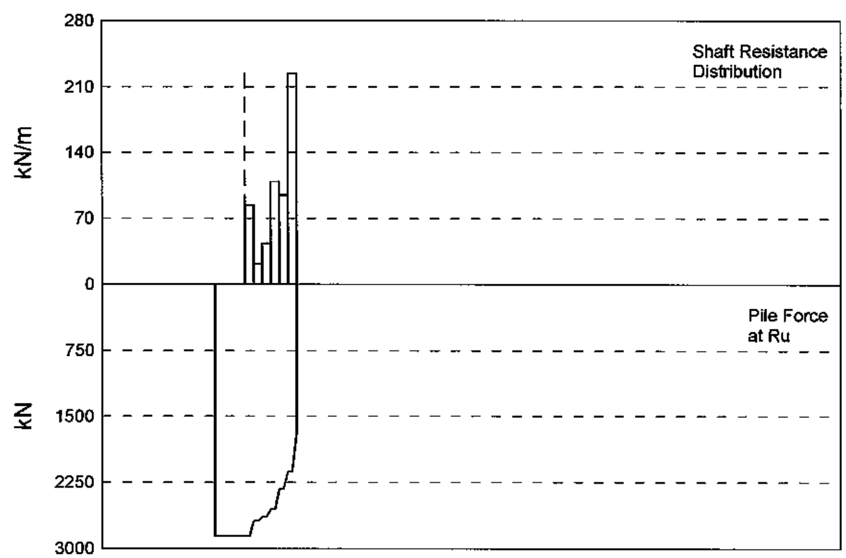
Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 5123.0 m/s, 2L/c 8.2 ms







Ru = 2861.7 kN
Rs = 1198.3 kN
Rb = 1663.4 kN
Dy = 20.2 mm
Dx = 21.2 mm



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Test: 17-Feb-2012 14:14:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2861.7; along Shaft 1198.3; at Toe 1663.4 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2861.7				
1	9.4	1.9	174.7	2687.0	174.7	94.16	65.03	0.455
2	11.4	3.9	45.2	2641.8	219.9	21.72	15.00	0.455
3	13.5	6.0	90.3	2551.5	310.2	43.44	30.00	0.455
4	15.6	8.1	225.8	2325.7	536.0	108.63	75.02	0.455
5	17.7	10.2	195.7	2130.0	731.7	94.13	65.01	0.455
6	19.8	12.3	466.6	1663.4	1198.3	224.45	155.01	0.455
Avg. Shaft			199.7			97.82	67.56	0.455
Toe			1663.4				12704.87	0.048

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	3.946	2.722
Case Damping Factor		0.804	0.118
Damping Type			Smith
Unloading Quake	(% of loading quake)	83	33
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	50	
Resistance Gap (included in Toe Quake)	(mm)		0.500
Soil Plug Weight	(kN)		0.71

CAPWAP match quality = 2.82 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 1.928 mm; blow count = 519 b/m

max. Top Comp. Stress = 223.3 MPa (T= 21.1 ms, max= 1.067 x Top)
max. Comp. Stress = 238.3 MPa (Z= 9.4 m, T= 22.7 ms)
max. Tens. Stress = -41.28 MPa (Z= 9.4 m, T= 48.5 ms)
max. Energy (EMX) = 45.31 kJ; max. Measured Top Displ. (DMX)=20.24 mm

Analysis: 20-Feb-2012

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RSTRK; Blow: 247
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Test: 17-Feb-2012 14:14:
CAPWAP(R) 2006-3
OP: GR

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3751.2	-513.2	223.3	-30.55	45.31	5.5	19.899
2	2.1	3749.8	-556.5	223.2	-33.13	44.45	5.5	19.184
3	3.1	3748.3	-604.0	223.1	-35.95	43.69	5.5	18.509
4	4.2	3746.7	-629.7	223.0	-37.48	42.91	5.5	17.828
5	5.2	3745.1	-634.6	222.9	-37.77	42.10	5.5	17.129
6	6.2	3748.0	-643.1	223.1	-38.28	41.17	5.5	16.392
7	7.3	3795.8	-657.2	225.9	-39.12	40.12	5.4	15.601
8	8.3	3916.6	-665.6	233.1	-39.62	38.95	5.3	14.764
9	9.4	4002.7	-693.5	238.3	-41.28	37.69	5.1	13.891
10	10.4	3532.4	-566.5	210.3	-33.72	31.57	5.1	13.082
11	11.4	3574.2	-600.6	212.8	-35.75	30.58	5.0	12.312
12	12.5	3508.8	-579.8	208.9	-34.51	28.53	4.9	11.584
13	13.5	3598.3	-591.4	214.2	-35.20	27.62	4.8	10.848
14	14.6	3492.6	-536.9	207.9	-31.96	24.77	4.6	10.116
15	15.6	3616.5	-569.5	215.3	-33.90	23.83	4.4	9.365
16	16.6	3170.1	-406.3	188.7	-24.19	19.10	4.2	8.671
17	17.7	3126.7	-423.2	186.1	-25.19	18.42	4.3	8.025
18	18.7	2387.0	-281.4	142.1	-16.75	14.88	4.9	7.475
19	19.8	2695.2	-308.8	160.4	-18.38	9.58	4.8	6.867
Absolute	9.4			238.3			(T =	22.7 ms)
	9.4				-41.28		(T =	48.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4132.4	3779.2	3426.1	3072.9	2719.8	2366.6	2013.5	1660.3	1307.2	954.0
RX	4209.3	3867.3	3559.5	3277.6	3048.4	2929.3	2903.8	2884.7	2865.7	2846.6
RU	4132.4	3779.2	3426.1	3072.9	2719.8	2366.6	2013.5	1660.3	1307.2	954.0

RAU = 2599.5 (kN); RA2 = 2851.9 (kN)

Current CAPWAP Ru = 2861.7 (kN); Corresponding J(RP)= 0.36; J(RX) = 0.82

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.49	20.90	3725.8	3938.1	3961.2	20.238	0.998	1.000	46.6	4392.3

PILE PROFILE AND PILE MODEL

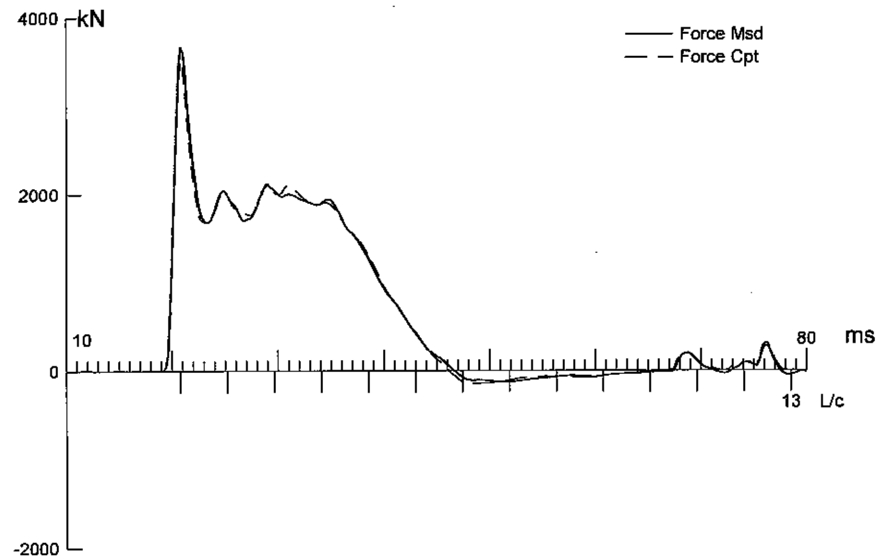
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
19.75	168.00	206842.7	77.287	1.448

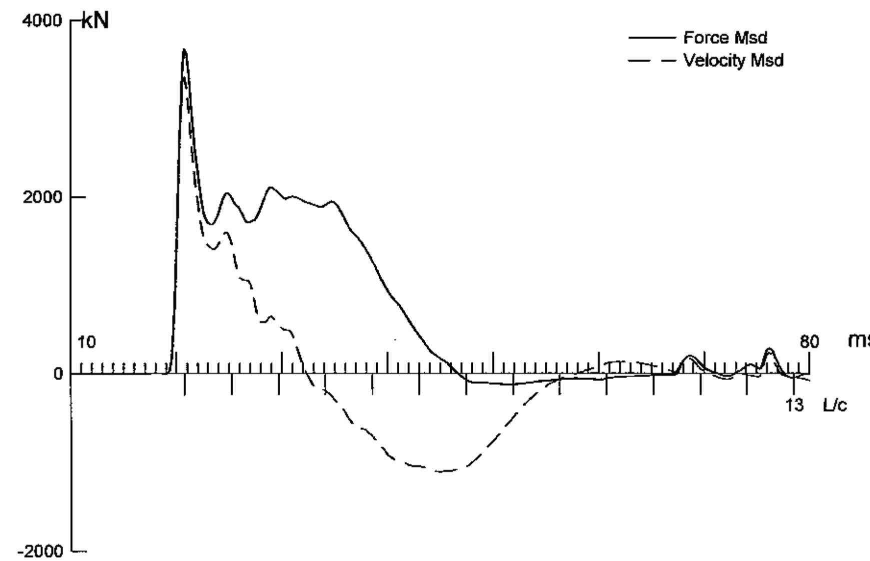
Toe Area 0.131 m^2

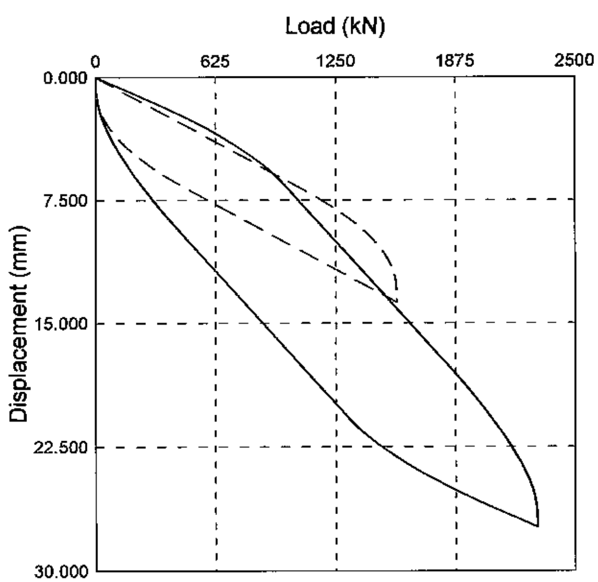
Top Segment Length 1.04 m, Top Impedance 678.30 kN/m/s

Pile Damping 1.0 %, Time Incr 0.203 ms, Wave Speed 5123.0 m/s, 2L/c 7.7 ms

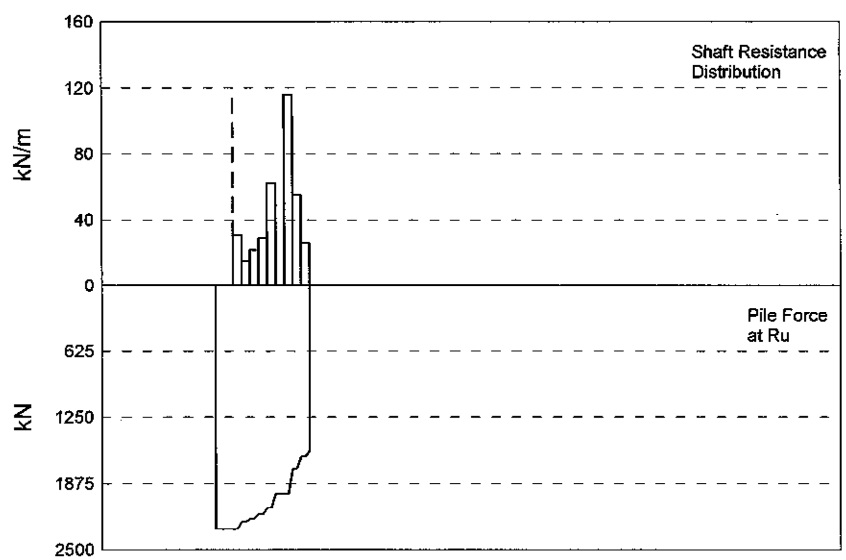
 Analysis: 20-Feb-2012







Ru =	2302.4	kN
Rs =	735.1	kN
Rb =	1567.3	kN
Dy =	26.6	mm
Dx =	27.4	mm



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RSTRK; Blow: 312
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Test: 09-Mar-2012 13:08:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2302.4; along Shaft 735.1; at Toe 1567.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2302.4				
1	6.2	1.3	63.6	2238.8	63.6	50.68	35.00	0.523
2	8.3	3.3	30.6	2208.2	94.2	14.80	10.22	0.523
3	10.3	5.4	44.9	2163.3	139.1	21.72	15.00	0.523
4	12.4	7.5	59.9	2103.4	199.0	28.96	20.00	0.523
5	14.5	9.5	128.8	1974.6	327.8	62.26	43.00	0.523
6	16.5	11.6	0.0	1974.6	327.8	0.00	0.00	0.000
7	18.6	13.7	239.6	1735.0	567.4	115.87	80.02	0.523
8	20.7	15.7	113.8	1621.2	681.2	55.02	38.00	0.523
9	22.7	17.8	53.9	1567.3	735.1	26.06	18.00	0.523
Avg. Shaft			81.7			41.30	28.52	0.523
Toe			1567.3				11971.11	0.405

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.318	9.946
Case Damping Factor		0.567	0.936
Unloading Quake	(% of loading quake)	100	99
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	32	
Resistance Gap (included in Toe Quake)	(mm)		0.065
Soil Plug Weight	(kN)		0.31

```
CAPWAP match quality   =   1.51        (Wave Up Match) ; RSA = 0
Observed: final set    =   0.800 mm;   blow count     =   1250 b/m
Computed: final set    =   0.495 mm;   blow count     =   2019 b/m

max. Top Comp. Stress  =   209.4 MPa   (T=  21.2 ms, max= 1.034 x Top)
max. Comp. Stress      =   216.6 MPa   (Z=   6.2 m, T=  22.2 ms)
max. Tens. Stress      =  -16.98 MPa   (Z=  12.4 m, T=  51.3 ms)
max. Energy (EMX)      =   50.87 kJ;    max. Measured Top Displ. (DMX)=22.75 mm
```


Analysis: 11-Mar-2012

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3518.3	-168.7	209.4	-10.04	50.87	5.2	22.347
2	2.1	3516.7	-189.8	209.3	-11.30	50.25	5.2	21.739
3	3.1	3519.5	-209.8	209.5	-12.49	49.62	5.2	21.125
4	4.1	3545.9	-231.5	211.1	-13.78	48.98	5.1	20.508
5	5.2	3600.7	-253.2	214.3	-15.07	48.33	5.0	19.888
6	6.2	3638.6	-275.2	216.6	-16.38	47.66	5.0	19.260
7	7.2	3437.9	-237.6	204.6	-14.15	44.23	4.9	18.641
8	8.3	3467.9	-260.5	206.4	-15.50	43.58	4.9	18.017
9	9.3	3398.2	-256.6	202.3	-15.27	41.68	4.8	17.397
10	10.3	3439.9	-279.2	204.8	-16.62	41.02	4.8	16.771
11	11.4	3337.0	-263.7	198.6	-15.69	38.72	4.7	16.152
12	12.4	3409.0	-285.3	202.9	-16.98	38.07	4.6	15.529
13	13.4	3299.9	-263.4	196.4	-15.68	35.45	4.4	14.920
14	14.5	3354.3	-283.3	199.7	-16.86	34.81	4.3	14.308
15	15.5	2951.0	-216.6	175.7	-12.90	30.45	4.3	13.733
16	16.5	3062.6	-235.3	182.3	-14.01	29.89	4.2	13.158
17	17.6	3235.3	-253.7	192.6	-15.10	29.33	4.0	12.582
18	18.6	3318.0	-272.3	197.5	-16.21	28.77	3.8	12.003
19	19.6	2744.2	-151.1	163.3	-8.99	23.00	3.6	11.488
20	20.7	2810.4	-168.1	167.3	-10.00	22.55	3.5	10.970
21	21.7	2510.3	-121.5	149.4	-7.23	19.86	3.4	10.485
22	22.7	2556.8	-136.9	152.2	-8.15	19.01	3.4	10.000
Absolute	6.2			216.6			(T =	22.2 ms)
	12.4				-16.98		(T =	51.3 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4296.3	4016.0	3735.7	3455.4	3175.1	2894.8	2614.5	2334.2	2053.9	1773.6
RX	4296.3	4016.0	3735.7	3455.4	3175.1	2894.8	2614.5	2338.3	2303.4	2268.5
RU	4296.3	4016.0	3735.7	3455.4	3175.1	2894.8	2614.5	2334.2	2053.9	1773.6

RAU = 1248.4 (kN); RA2 = 2684.7 (kN)

Current CAPWAP Ru = 2302.4 (kN); Corresponding J(RP)= 0.71; J(RX) = 0.80

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.01	20.99	3401.4	3697.8	3704.0	22.752	0.799	0.800	52.1	4426.5

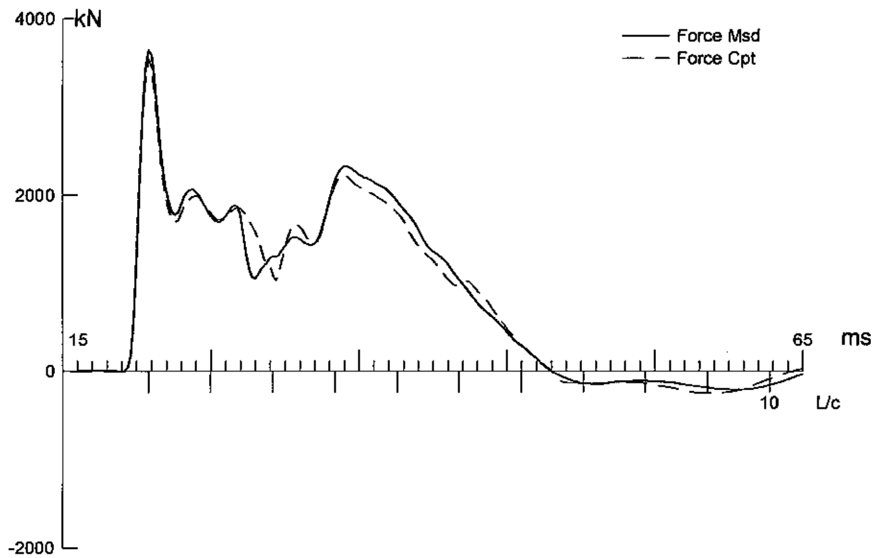
PILE PROFILE AND PILE MODEL

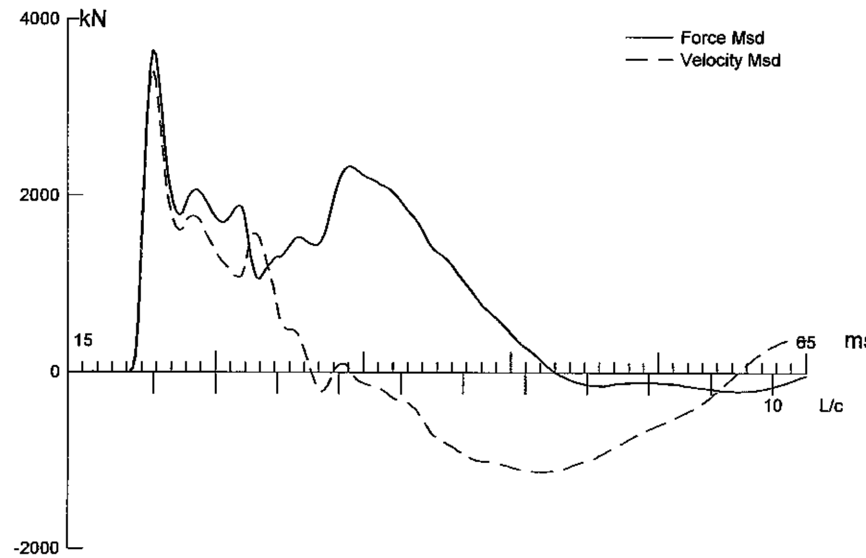
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
22.75	168.00	206842.7	77.287	1.448

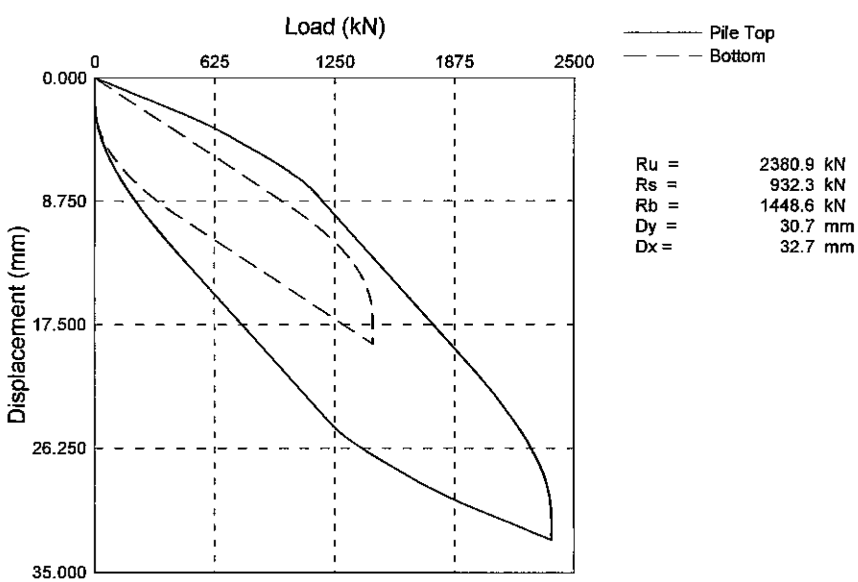
Toe Area 0.131 m^2

Top Segment Length 1.03 m, Top Impedance 678.30 kN/m/s

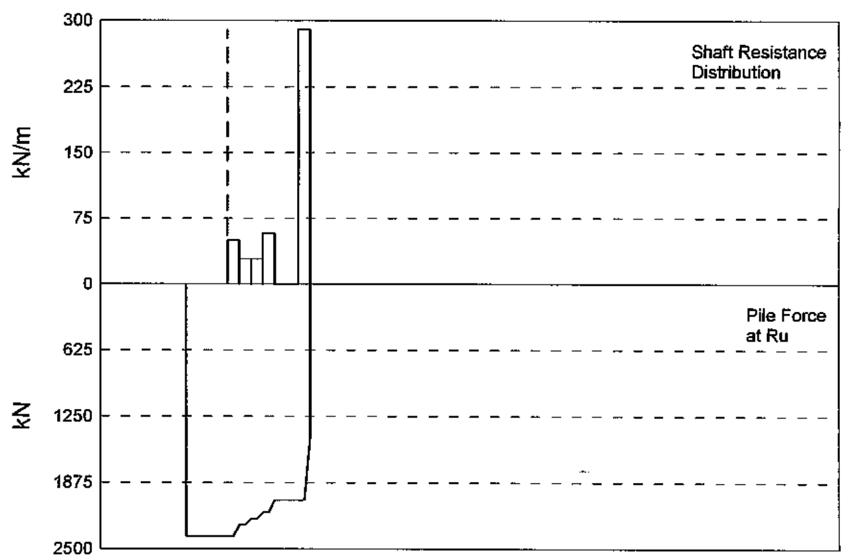
Pile Damping 1.0 %, Time Incr 0.202 ms, Wave Speed 5123.0 m/s, 2L/c 8.9 ms







Ru =	2380.9	kN
Rs =	932.3	kN
Rb =	1448.6	kN
Dy =	30.7	mm
Dx =	32.7	mm



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RSTRK; Blow: 33
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Test: 09-Mar-2012 14:50:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2380.9; along Shaft 932.3; at Toe 1448.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2380.9				
1	9.2	2.0	102.1	2278.8	102.1	50.68	35.00	0.404
2	11.3	4.1	59.3	2219.5	161.4	28.96	20.00	0.404
3	13.3	6.1	59.3	2160.2	220.7	28.96	20.00	0.404
4	15.4	8.2	118.6	2041.6	339.3	57.92	40.00	0.404
5	17.4	10.2	0.0	2041.6	339.3	0.00	0.00	0.000
6	19.5	12.3	0.0	2041.6	339.3	0.00	0.00	0.000
7	21.5	14.3	593.0	1448.6	932.3	289.61	200.01	0.404
Avg. Shaft			133.2			65.20	45.02	0.404
Toe			1448.6				11064.52	0.140

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.344	12.980
Case Damping Factor		0.555	0.299
Unloading Quake	(% of loading quake)	46	99
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	38	
Resistance Gap (included in Toe Quake) (mm)			2.914

CAPWAP match quality = 4.35 (Wave Up Match) ; RSA = 0
Observed: final set = 2.000 mm; blow count = 500 b/m
Computed: final set = 1.474 mm; blow count = 679 b/m

max. Top Comp. Stress = 210.7 MPa (T= 21.2 ms, max= 1.045 x Top)
max. Comp. Stress = 220.3 MPa (Z= 9.2 m, T= 22.8 ms)
max. Tens. Stress = -25.50 MPa (Z= 9.2 m, T= 59.4 ms)
max. Energy (EMX) = 51.05 kJ; max. Measured Top Displ. (DMX)=24.83 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3540.0	-264.9	210.7	-15.77	51.05	5.2	24.004
2	2.0	3538.2	-288.6	210.6	-17.18	50.61	5.2	23.524
3	3.1	3536.4	-310.4	210.5	-18.48	50.18	5.2	23.023
4	4.1	3534.5	-332.9	210.4	-19.82	49.72	5.2	22.505
5	5.1	3532.7	-354.8	210.3	-21.12	49.22	5.2	21.963
6	6.1	3536.3	-375.7	210.5	-22.36	48.68	5.2	21.400
7	7.2	3572.5	-394.1	212.6	-23.46	48.10	5.1	20.822
8	8.2	3646.2	-412.3	217.0	-24.54	47.54	5.0	20.247
9	9.2	3700.6	-428.4	220.3	-25.50	47.01	4.9	19.690
10	10.2	3440.4	-390.4	204.8	-23.24	42.48	4.9	19.141
11	11.3	3480.2	-400.8	207.2	-23.86	41.93	4.8	18.576
12	12.3	3350.4	-382.7	199.4	-22.78	39.22	4.7	18.011
13	13.3	3408.7	-394.2	202.9	-23.46	38.77	4.6	17.433
14	14.3	3312.8	-375.2	197.2	-22.33	36.25	4.5	16.897
15	15.4	3354.4	-383.9	199.7	-22.85	35.77	4.5	16.369
16	16.4	3024.6	-335.2	180.0	-19.95	31.53	4.5	15.854
17	17.4	3022.5	-345.2	179.9	-20.55	31.05	4.5	15.327
18	18.4	2993.0	-358.6	178.2	-21.35	30.55	4.5	14.781
19	19.5	2829.8	-374.1	168.4	-22.27	29.99	4.8	14.212
20	20.5	2752.9	-390.9	163.9	-23.27	29.40	4.8	13.621
21	21.5	2754.4	-407.5	164.0	-24.26	14.30	4.8	13.020
Absolute	9.2			220.3			(T =	22.8 ms)
	9.2				-25.50		(T =	59.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3843.2	3514.3	3185.3	2856.4	2527.4	2198.5	1869.5	1540.5	1211.6	882.6
RX	3885.6	3563.9	3242.1	2920.4	2601.6	2533.0	2502.7	2472.4	2442.1	2411.8
RU	3843.2	3514.3	3185.3	2856.4	2527.4	2198.5	1869.5	1540.5	1211.6	882.6

RAU = 2325.5 (kN); RA2 = 2582.0 (kN)

Current CAPWAP Ru = 2380.9 (kN); Corresponding J(RP)= 0.44; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.10	20.98	3457.0	3675.7	3677.9	24.830	2.001	2.000	51.8	3860.3

PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
21.50	168.00	206842.7	77.287	1.448

Toe Area 0.131 m^2

Top Segment Length 1.02 m, Top Impedance 678.30 kN/m/s

Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 5123.0 m/s, 2L/c 8.4 ms

Site 3

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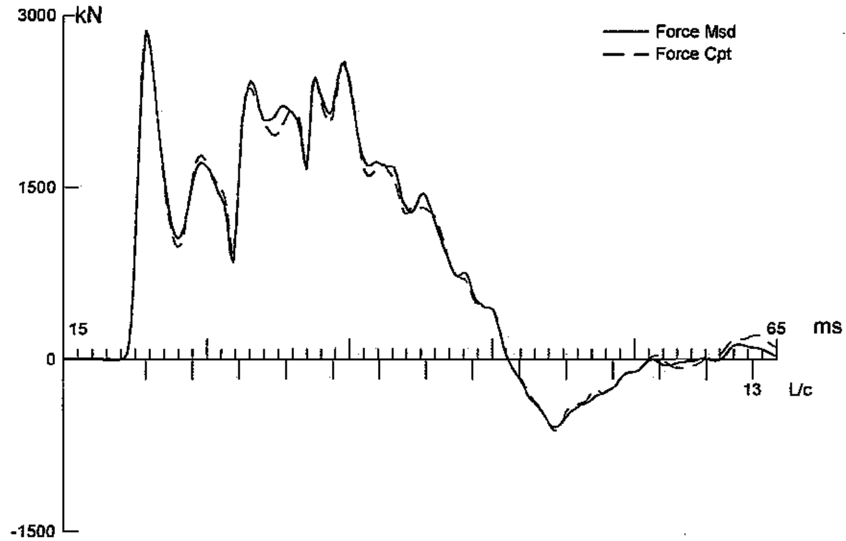
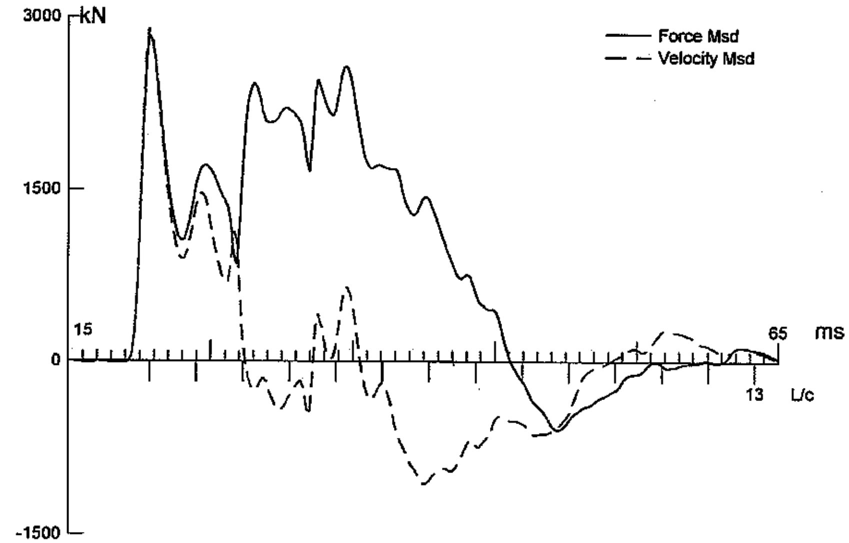
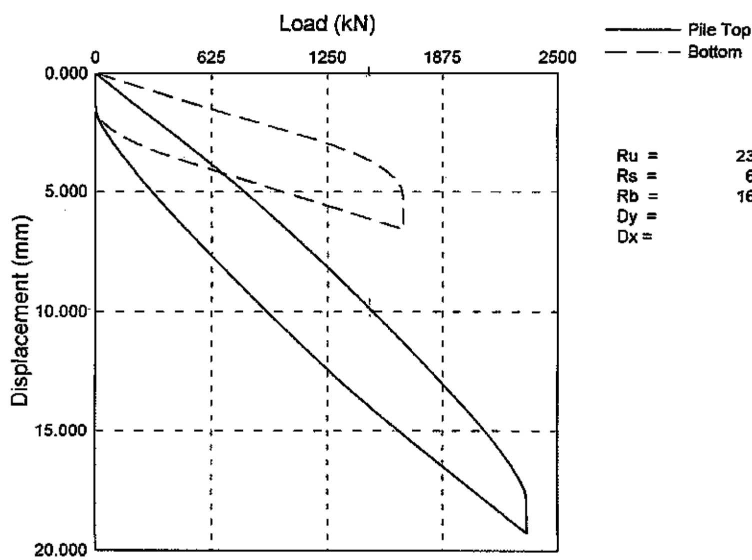
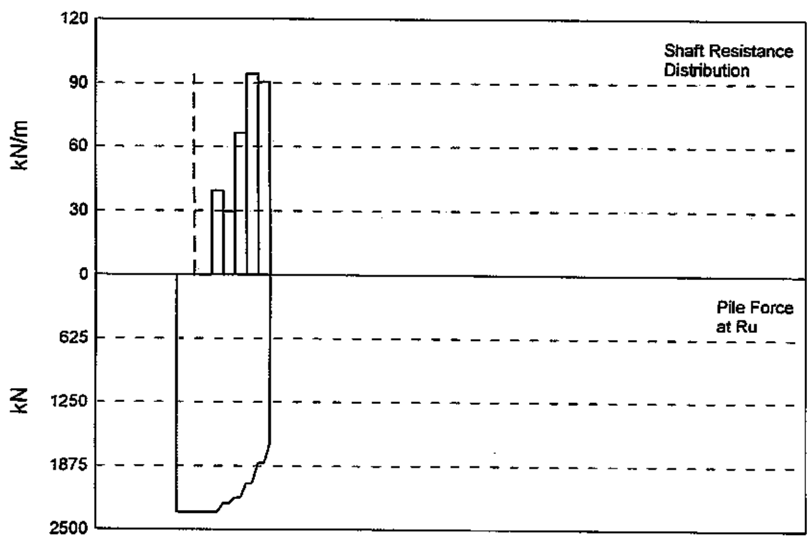






Ru = 2332.2 kN
Rs = 671.1 kN
Rb = 1661.1 kN
Dy = 17.9 mm
Dx = 19.3 mm



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RSTRK; Blow: 23
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Test: 02-May-2012 14:07:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2332.2; along Shaft 671.1; at Toe 1661.1 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2332.2				
1	6.3	2.3	0.0	2332.2	0.0	0.00	0.00	0.000
2	8.4	4.4	82.3	2249.9	82.3	39.18	31.68	0.485
3	10.5	6.5	61.7	2188.2	144.0	29.38	23.76	0.485
4	12.6	8.6	138.8	2049.4	282.8	66.12	53.46	0.485
5	14.7	10.7	198.0	1851.4	480.8	94.29	76.24	0.485
6	16.8	12.8	190.3	1661.1	671.1	90.60	73.26	0.485
Avg. Shaft			111.8			52.43	42.39	0.485
Toe			1661.1				17377.23	0.484

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.422	3.952
Case Damping Factor		0.573	1.416
Damping Type			Smith
Unloading Quake	(% of loading quake)	48	32
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	65	
Resistance Gap (included in Toe Quake) (mm)			0.455

CAPWAP match quality = 3.09 (Wave Up Match) ; RSA = 0
Observed: final set = 1.400 mm; blow count = 714 b/m
Computed: final set = 2.050 mm; blow count = 488 b/m

max. Top Comp. Stress = 204.1 MPa (T= 21.1 ms, max= 1.043 x Top)
max. Comp. Stress = 212.9 MPa (Z= 8.4 m, T= 22.5 ms)
max. Tens. Stress = -51.38 MPa (Z= 8.4 m, T= 51.4 ms)
max. Energy (EMX) = 32.33 kJ; max. Measured Top Displ. (DMX)=18.01 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.1	2870.3	-634.9	204.1	-45.14	32.33	5.1	17.226
2	2.1	2867.7	-613.3	203.9	-43.60	31.69	5.1	16.526
3	3.2	2865.1	-639.3	203.7	-45.45	31.07	5.1	15.836
4	4.2	2862.4	-684.8	203.5	-48.69	30.43	5.0	15.152
5	5.3	2861.8	-716.7	203.5	-50.96	29.72	5.0	14.434
6	6.3	2879.5	-717.0	204.7	-50.98	28.96	5.0	13.701
7	7.4	2932.9	-689.9	208.5	-49.05	28.15	4.9	12.941
8	8.4	2994.6	-722.7	212.9	-51.38	27.39	4.8	12.196
9	9.5	2781.3	-653.9	197.8	-46.50	24.39	4.7	11.463
10	10.5	2853.2	-651.9	202.9	-46.35	23.48	4.6	10.647
11	11.6	2762.8	-584.3	196.4	-41.55	21.11	4.4	9.827
12	12.6	2882.1	-565.5	204.9	-40.21	20.19	4.2	9.013
13	13.7	2558.3	-443.5	181.9	-31.53	16.60	4.3	8.248
14	14.7	2487.4	-435.9	176.9	-31.00	15.72	4.9	7.457
15	15.8	2613.5	-222.6	185.8	-15.83	11.70	5.1	6.723
16	16.8	2959.0	-204.5	210.4	-14.54	10.14	4.1	6.020
Absolute	8.4			212.9			(T =	22.5 ms)
	8.4				-51.38		(T =	51.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3690.3	3474.3	3258.2	3042.2	2826.1	2610.1	2394.0	2177.9	1961.9	1745.8
RX	3972.8	3817.9	3663.0	3508.1	3367.3	3230.1	3092.8	2977.7	2882.5	2800.5
RU	3690.3	3474.3	3258.2	3042.2	2826.1	2610.1	2394.0	2177.9	1961.9	1745.8

RAU = 2272.0 (kN); RA2 = 2363.6 (kN)

Current CAPWAP Ru = 2332.2 (kN); RMX requires J > 0.9;

Check with PDA-W; RA2 may be a better Case Method

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.21	20.91	2957.4	2893.5	2893.5	18.005	1.393	1.400	32.7	3368.4

PILE PROFILE AND PILE MODEL

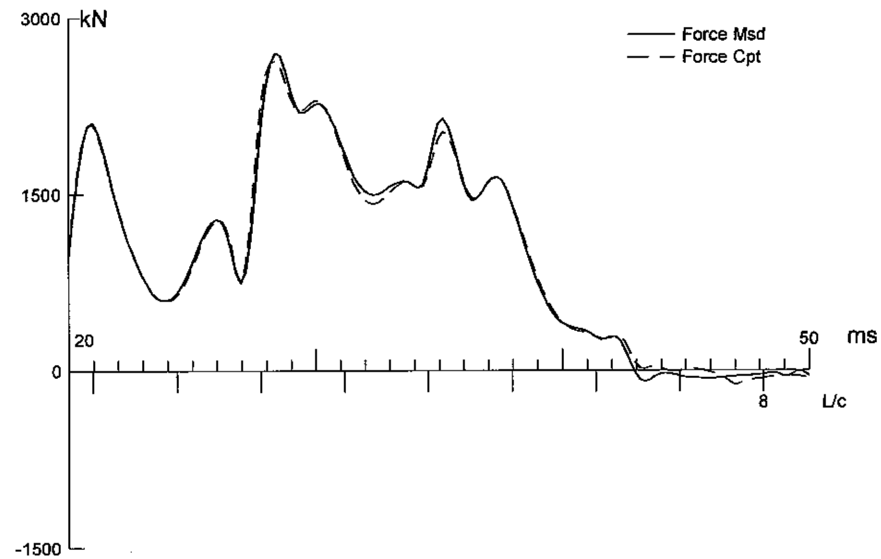
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	140.64	206842.7	77.287	1.237
16.80	140.64	206842.7	77.287	1.237

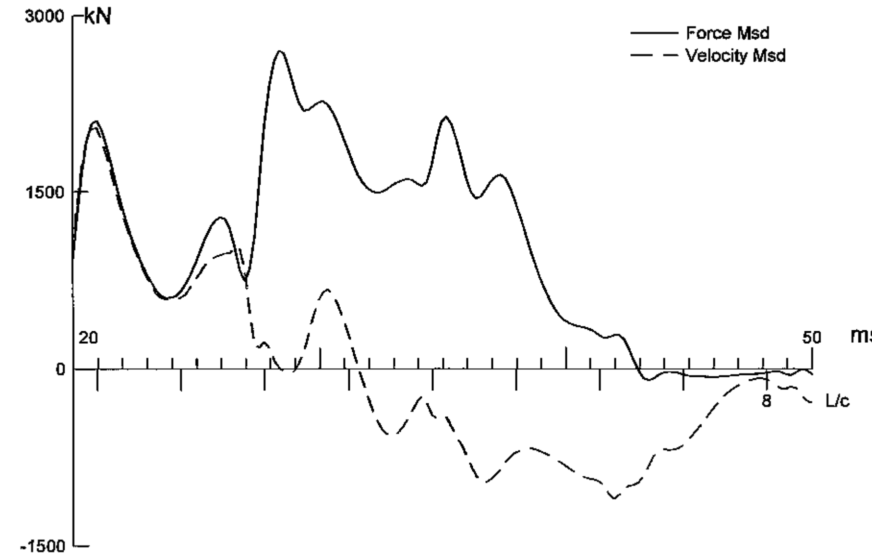
Toe Area 0.096 m^2

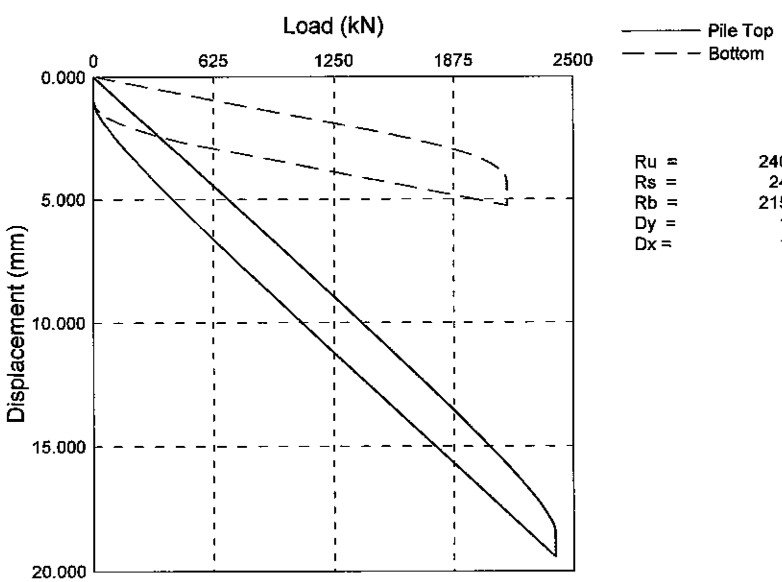
Top Segment Length 1.05 m, Top Impedance 567.85 kN/m/s

Pile Damping 1.0 %, Time Incr 0.205 ms, Wave Speed 5123.0 m/s, 2L/c 6.6 ms

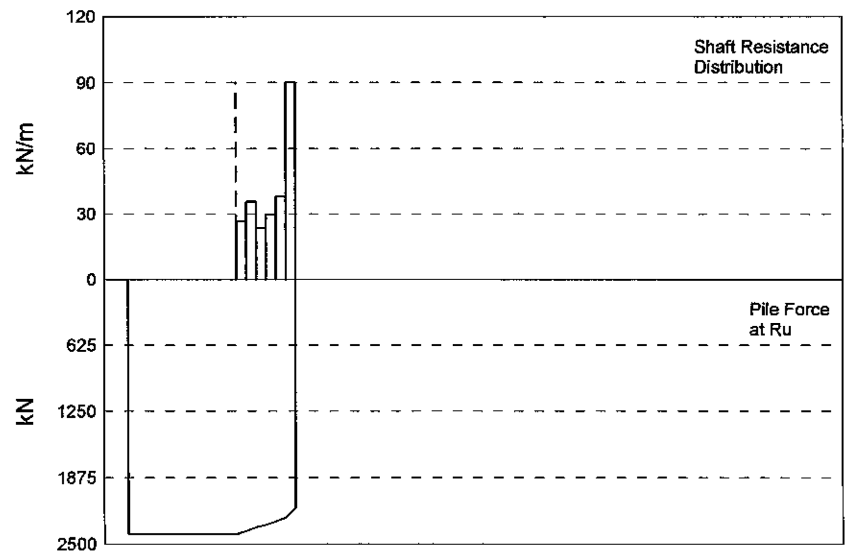
Site 4







Ru = 2404.8 kN
Rs = 249.7 kN
Rb = 2155.1 kN
Dy = 18.4 mm
Dx = 19.4 mm



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RSTRK; Blow: 11
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Test: 12-Oct-2012 08:25:
CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2404.8; along Shaft 249.7; at Toe 2155.1 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2404.8				
1	12.3	0.4	27.3	2377.5	27.3	71.28	57.67	0.807
2	13.3	1.4	36.5	2341.0	63.7	35.65	28.84	0.807
3	14.3	2.4	24.3	2316.7	88.0	23.74	19.21	0.807
4	15.4	3.5	30.4	2286.3	118.4	29.70	24.03	0.807
5	16.4	4.5	38.9	2247.4	157.3	38.01	30.75	0.807
6	17.4	5.5	92.4	2155.1	249.7	90.23	73.00	0.807
Avg. Shaft			41.6			45.40	36.73	0.807
Toe			2155.1				22570.85	0.869

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.952	3.270
Case Damping Factor		0.354	3.290
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	40
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	79	
Resistance Gap (included in Toe Quake)	(mm)		0.269
Soil Plug Weight	(kN)		0.23

CAPWAP match quality = 3.18 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 183.9 MPa (T= 28.6 ms, max= 1.149 x Top)
max. Comp. Stress = 211.3 MPa (Z= 17.4 m, T= 24.8 ms)
max. Tens. Stress = -37.68 MPa (Z= 10.2 m, T= 45.4 ms)
max. Energy (EMX) = 22.37 kJ; max. Measured Top Displ. (DMX)=16.28 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2592.9	-148.6	183.9	-10.54	22.37	3.6	15.583
2	2.0	2491.5	-196.6	176.7	-13.94	21.86	3.6	14.981
3	3.1	2370.1	-252.8	168.1	-17.93	21.25	3.6	14.337
4	4.1	2318.8	-308.5	164.5	-21.88	20.57	3.6	13.643
5	5.1	2412.2	-357.6	171.1	-25.36	19.81	3.6	12.915
6	6.1	2449.1	-382.6	173.7	-27.14	19.06	3.6	12.200
7	7.2	2497.5	-388.9	177.1	-27.58	18.46	3.6	11.546
8	8.2	2494.5	-407.0	176.9	-28.86	17.81	3.6	10.877
9	9.2	2433.8	-497.5	172.6	-35.29	17.03	3.6	10.136
10	10.2	2432.1	-531.2	172.5	-37.68	16.13	3.5	9.339
11	11.3	2551.0	-517.6	180.9	-36.71	15.10	3.5	8.495
12	12.3	2648.9	-513.6	187.9	-36.43	14.03	3.4	7.627
13	13.3	2663.6	-450.8	188.9	-31.97	12.40	3.3	6.785
14	14.3	2587.2	-361.3	183.5	-25.63	10.63	3.7	5.947
15	15.4	2606.7	-324.2	184.9	-22.99	9.11	3.8	5.068
16	16.4	2841.2	-263.7	201.5	-18.70	7.52	3.3	4.178
17	17.4	2978.9	-160.9	211.3	-11.41	6.27	2.2	3.325
Absolute	17.4			211.3			(T =	24.8 ms)
	10.2				-37.68		(T =	45.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	2934.7	2809.9	2685.0	2560.2	2435.4	2310.5	2185.7	2060.8	1936.0	1811.2
RX	3139.9	3085.3	3030.7	2976.2	2924.4	2882.7	2851.6	2820.5	2789.4	2758.3
RU	2934.7	2809.9	2685.0	2560.2	2435.4	2310.5	2185.7	2060.8	1936.0	1811.2

RAU = 2176.9 (kN); RA2 = 2282.8 (kN)

Current CAPWAP Ru = 2404.8 (kN); Corresponding J(RP)= 0.42;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.62	21.18	2060.1	2123.1	2723.2	16.279	0.895	1.000	22.9	2651.5

PILE PROFILE AND PILE MODEL

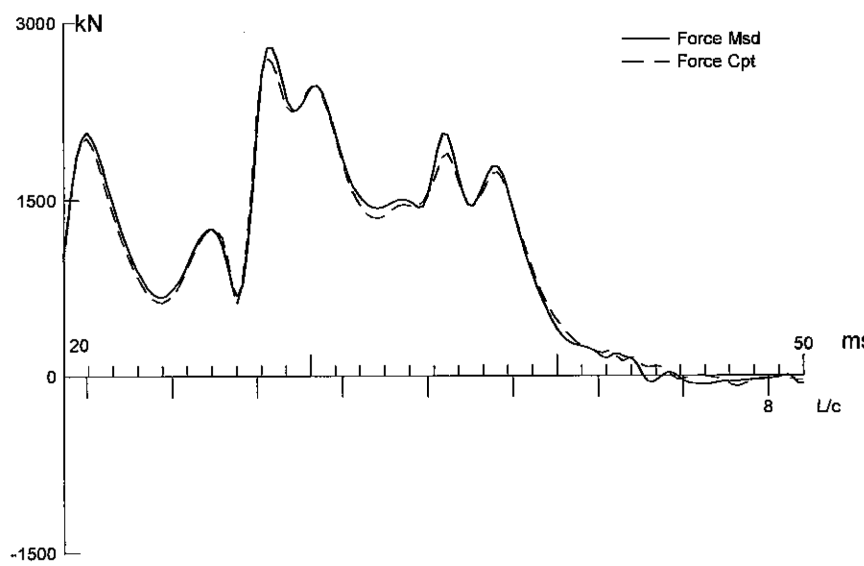
Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.40	141.00	206842.7	77.287	1.236

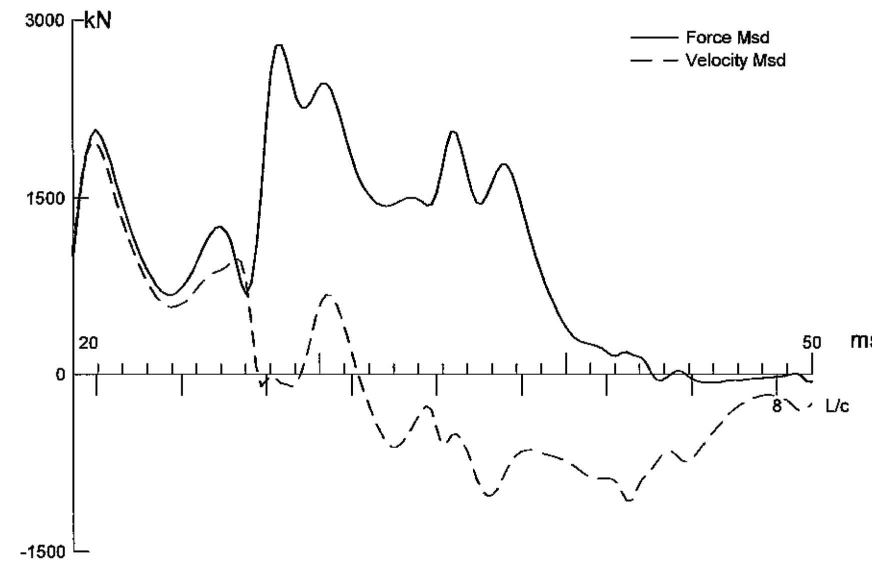
Toe Area 0.095 m²

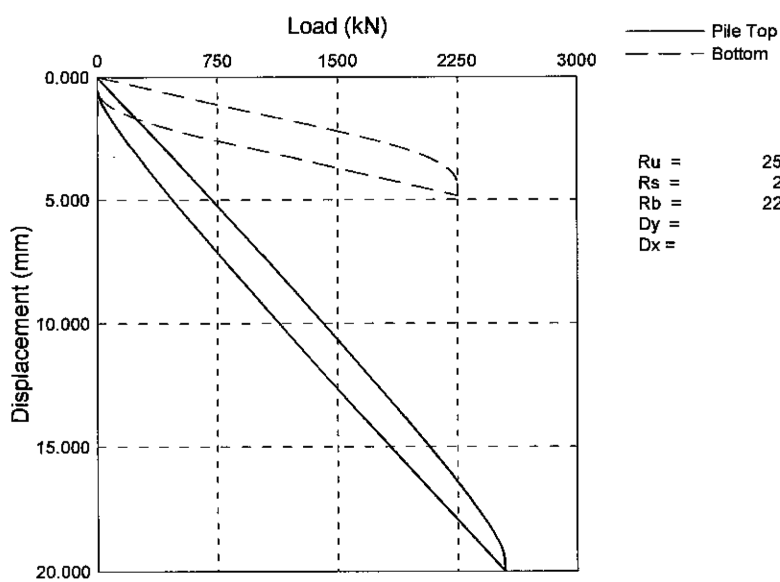
Segmnt Number	Dist. B.G. m	Impedance kN/m/s	Imped. Change %	Tension Slack mm	Tension Eff.	Compression Slack mm	Compression Eff.	Perim. m	Soil Plug kN
1	1.02	569.29	0.00	0.000	0.000	-0.000	0.000	1.236	0.00
2	2.05	569.29	0.00	0.000	0.000	-0.000	0.000	1.236	0.03

Segmnt Number	Dist. B.G. m	Impedance kN/m/s	Imped. Change %	Slack mm	Tension Eff.	Compression Slack mm	Compression Eff.	Perim. m	Soil Plug kN
17	17.40	569.29	0.00	0.000	0.000	-0.000	0.000	1.236	0.03

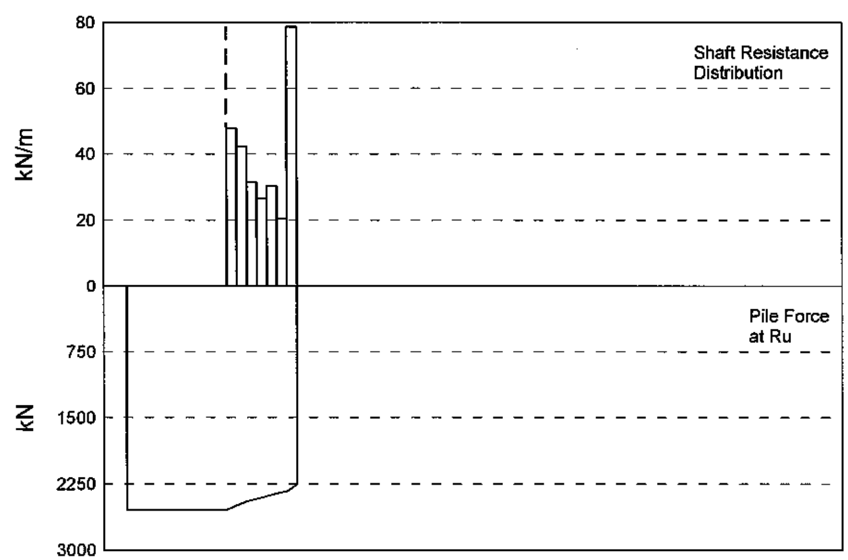
Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 5123.0 m/s, 2L/c 6.8 ms







Ru =	2542.9	kN
Rs =	289.1	kN
Rb =	2253.8	kN
Dy =	19.5	mm
Dx =	20.0	mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2542.9; along Shaft 289.1; at Toe 2253.8 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2542.9				
1	11.5	0.7	49.8	2493.1	49.8	76.21	61.66	0.513
2	12.5	1.7	44.1	2449.0	93.9	42.36	34.27	0.513
3	13.5	2.7	32.8	2416.2	126.6	31.47	25.46	0.513
4	14.6	3.8	27.7	2388.5	154.3	26.61	21.53	0.513
5	15.6	4.8	31.5	2357.0	185.8	30.24	24.47	0.513
6	16.7	5.9	21.4	2335.6	207.2	20.55	16.63	0.513
7	17.7	6.9	81.8	2253.8	289.1	78.61	63.60	0.513
Avg. Shaft			41.3			41.89	33.90	0.513
Toe			2253.8				23604.91	0.801

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.855	3.342
Case Damping Factor		0.260	3.171
Damping Type			Smith
Unloading Quake	(% of loading quake)	83	58
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	60	
Resistance Gap (included in Toe Quake)	(mm)		0.426
Soil Plug Weight	(kN)		0.69

CAPWAP match quality = 3.19 (Wave Up Match) ; RSA = 0
Observed: final set = 0.500 mm; blow count = 2000 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 186.6 MPa (T= 28.7 ms, max= 1.122 x Top)
max. Comp. Stress = 209.4 MPa (Z= 17.7 m, T= 24.8 ms)
max. Tens. Stress = -27.15 MPa (Z= 11.5 m, T= 45.1 ms)
max. Energy (EMX) = 21.76 kJ; max. Measured Top Displ. (DMX)=15.76 mm

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RSTRK; Blow: 27
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Test: 12-Oct-2012 08:06:
CAPWAP(R) 2006-3
OP: GR

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2631.6	-119.3	186.6	-8.46	21.76	3.6	15.390
2	2.1	2535.9	-167.2	179.9	-11.86	21.24	3.6	14.787
3	3.1	2431.9	-220.1	172.5	-15.61	20.66	3.6	14.147
4	4.2	2384.5	-271.9	169.1	-19.28	20.03	3.6	13.467
5	5.2	2426.6	-319.2	172.1	-22.64	19.35	3.6	12.772
6	6.2	2487.8	-333.9	176.4	-23.68	18.69	3.6	12.096
7	7.3	2533.4	-317.3	179.7	-22.50	18.15	3.6	11.469
8	8.3	2530.9	-303.5	179.5	-21.52	17.56	3.5	10.803
9	9.4	2464.9	-352.3	174.8	-24.99	16.79	3.5	10.079
10	10.4	2470.8	-351.2	175.2	-24.90	15.95	3.4	9.300
11	11.5	2614.3	-382.9	185.4	-27.15	15.06	3.3	8.499
12	12.5	2577.6	-351.0	182.8	-24.90	13.23	3.3	7.693
13	13.5	2573.6	-283.5	182.5	-20.11	11.55	3.2	6.877
14	14.6	2569.6	-235.1	182.2	-16.67	10.06	3.6	6.026
15	15.6	2609.1	-196.2	185.0	-13.91	8.59	3.8	5.149
16	16.7	2789.6	-144.1	197.8	-10.22	7.11	3.5	4.238
17	17.7	2953.1	-102.1	209.4	-7.24	6.27	2.5	3.353
Absolute	17.7			209.4			(T =	24.8 ms)
	11.5				-27.15		(T =	45.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3219.0	3120.5	3022.0	2923.6	2825.1	2726.6	2628.2	2529.7	2431.2	2332.8
RX	3290.0	3253.0	3216.0	3178.9	3141.9	3104.9	3067.9	3030.8	2993.8	2956.8
RU	3241.0	3144.7	3048.4	2952.1	2855.9	2759.6	2663.3	2567.1	2470.8	2374.5

RAU = 2189.4 (kN); RA2 = 2811.9 (kN)

Current CAPWAP Ru = 2542.9 (kN); Corresponding J(RP)= 0.69;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.64	21.14	2071.7	2131.9	2836.0	15.763	0.082	0.500	21.9	2698.6

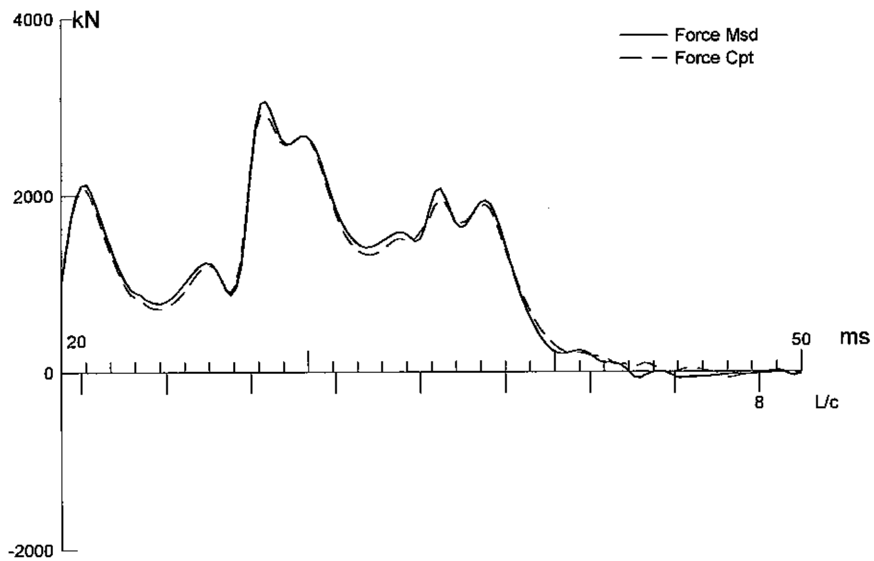
PILE PROFILE AND PILE MODEL

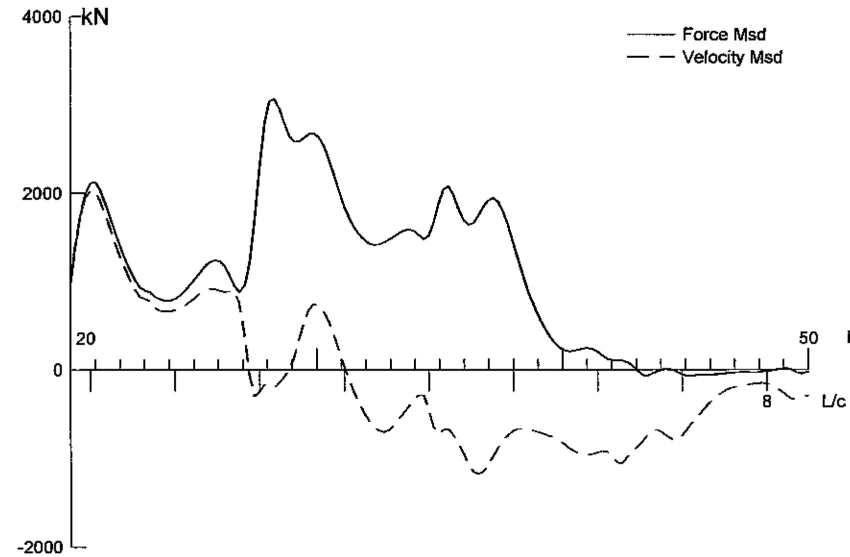
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.70	141.00	206842.7	77.287	1.236

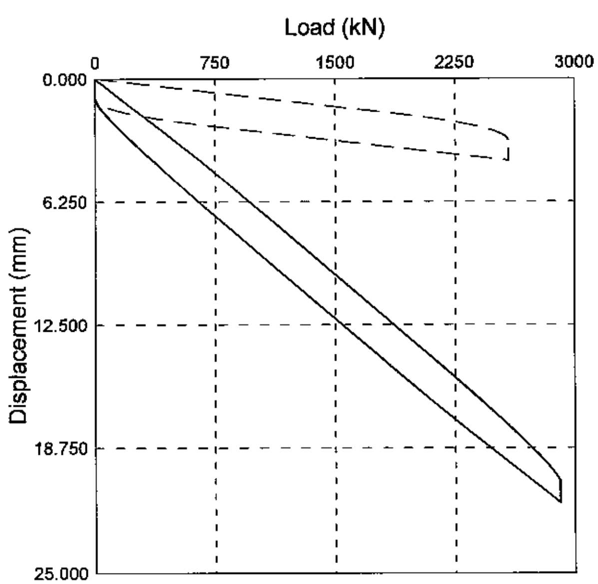
Toe Area 0.095 m^2

Top Segment Length 1.04 m, Top Impedance 569.29 kN/m/s

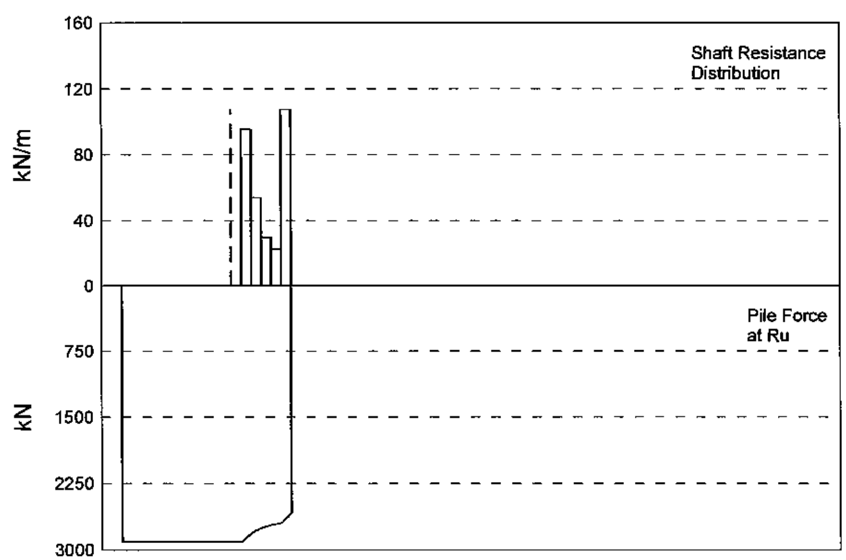
Pile Damping 1.0 %, Time Incr 0.203 ms, Wave Speed 5123.0 m/s, 2L/c 6.9 ms







Ru =	2907.2	kN
Rs =	320.5	kN
Rb =	2586.6	kN
Dy =	20.5	mm
Dx =	21.5	mm



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RSTRK; Blow: 24
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CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2907.2; along Shaft 320.5; at Toe 2586.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2907.2				
1	13.5	1.2	98.6	2808.6	98.6	85.07	68.83	0.304
2	14.5	2.2	55.8	2752.8	154.4	53.90	43.61	0.304
3	15.5	3.2	31.0	2721.8	185.4	29.94	24.22	0.304
4	16.6	4.3	23.6	2698.2	208.9	22.75	18.41	0.304
5	17.6	5.3	111.6	2586.6	320.5	107.79	87.21	0.304
Avg. Shaft			64.1			60.48	48.93	0.304
Toe			2586.6				27090.67	0.730

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.075	2.454
Case Damping Factor		0.171	3.317
Damping Type			Smith
Unloading Quake	(% of loading quake)	83	87
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	64	
Resistance Gap (included in Toe Quake) (mm)			0.100
Soil Plug Weight	(kN)		0.66

CAPWAP match quality = 3.89 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 204.2 MPa (T= 28.5 ms, max= 1.129 x Top)
max. Comp. Stress = 230.5 MPa (Z= 17.6 m, T= 24.7 ms)
max. Tens. Stress = -30.39 MPa (Z= 12.4 m, T= 45.1 ms)
max. Energy (EMX) = 22.37 kJ; max. Measured Top Displ. (DMX)=15.37 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2879.3	-94.5	204.2	-6.70	22.37	3.7	15.418
2	2.1	2726.0	-131.9	193.3	-9.35	21.80	3.7	14.782
3	3.1	2567.0	-192.6	182.1	-13.66	21.16	3.7	14.117
4	4.1	2577.0	-260.8	182.8	-18.50	20.49	3.7	13.429
5	5.2	2612.7	-327.7	185.3	-23.24	19.81	3.7	12.732
6	6.2	2656.4	-357.5	188.4	-25.35	19.14	3.7	12.041
7	7.2	2677.7	-344.7	189.9	-24.44	18.55	3.7	11.389
8	8.3	2679.6	-331.0	190.0	-23.47	17.98	3.6	10.752
9	9.3	2660.6	-367.8	188.7	-26.09	17.23	3.6	10.020
10	10.4	2593.8	-396.7	184.0	-28.13	16.28	3.6	9.201
11	11.4	2763.0	-408.8	196.0	-28.99	15.25	3.5	8.332
12	12.4	2867.8	-428.5	203.4	-30.39	14.07	3.4	7.432
13	13.5	2936.5	-409.2	208.3	-29.02	12.81	3.4	6.478
14	14.5	2898.6	-312.2	205.6	-22.14	10.36	3.6	5.511
15	15.5	2929.4	-251.8	207.8	-17.86	8.42	3.4	4.516
16	16.6	3078.4	-203.5	218.3	-14.43	6.74	2.8	3.492
17	17.6	3250.7	-139.3	230.5	-9.88	5.66	1.8	2.490
Absolute	17.6			230.5			(T =	24.7 ms)
	12.4				-30.39		(T =	45.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3469.0	3393.3	3317.7	3242.0	3166.3	3090.6	3014.9	2939.2	2863.6	2787.9
RX	3650.5	3604.6	3563.2	3521.9	3484.3	3455.6	3426.8	3398.0	3369.3	3340.5
RU	3506.5	3434.6	3362.6	3290.7	3218.8	3146.8	3074.9	3003.0	2931.0	2859.1

RAU = 2262.9 (kN); RA2 = 3176.8 (kN)

Current CAPWAP Ru = 2907.2 (kN); Corresponding J(RP)= 0.74;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.65	21.02	2080.3	2145.6	3112.7	15.372	0.678	1.000	22.3	2727.4

PILE PROFILE AND PILE MODEL

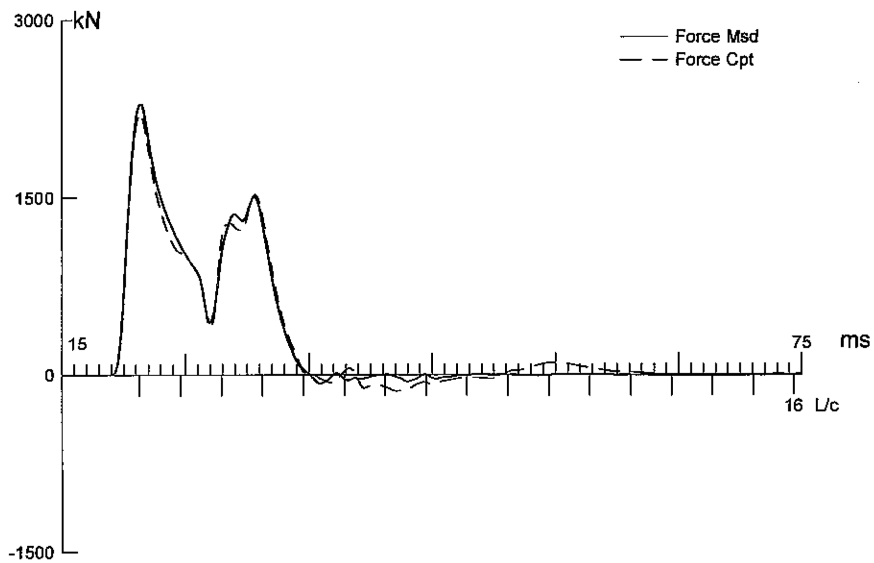
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.60	141.00	206842.7	77.287	1.236

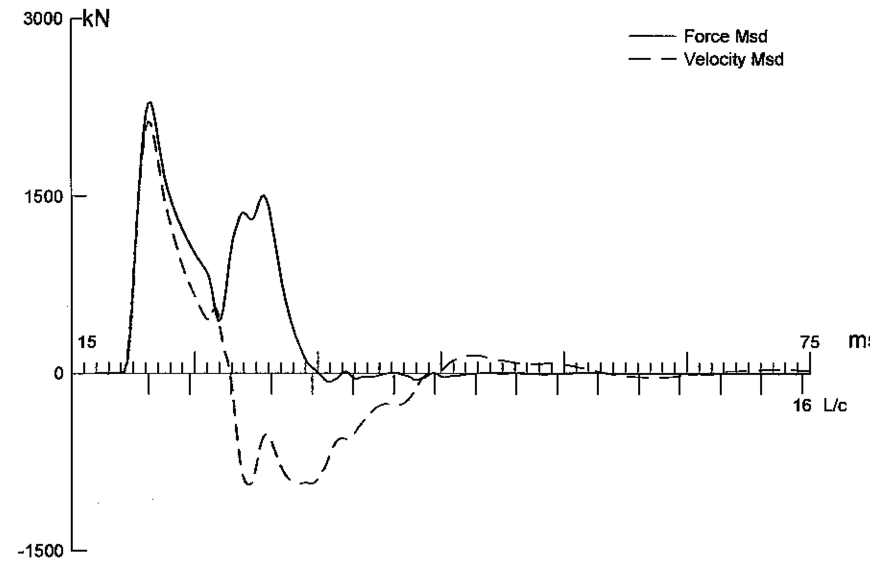
Toe Area 0.095 m^2

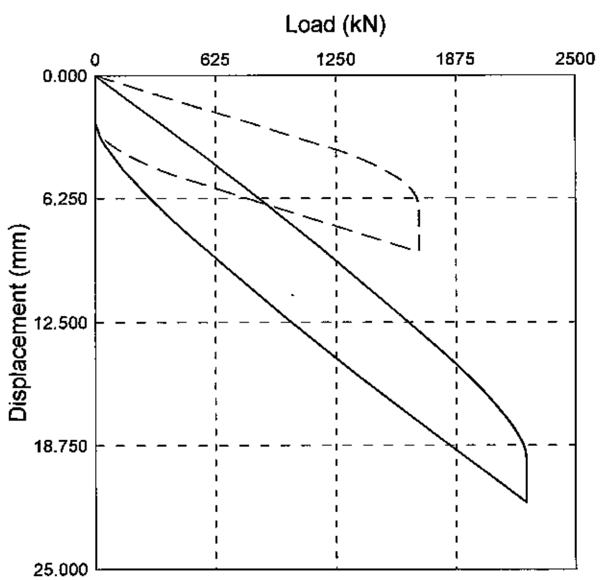
Top Segment Length 1.04 m, Top Impedance 569.29 kN/m/s

Pile Damping 1.0 %, Time Incr 0.202 ms, Wave Speed 5123.0 m/s, 2L/c 6.9 ms

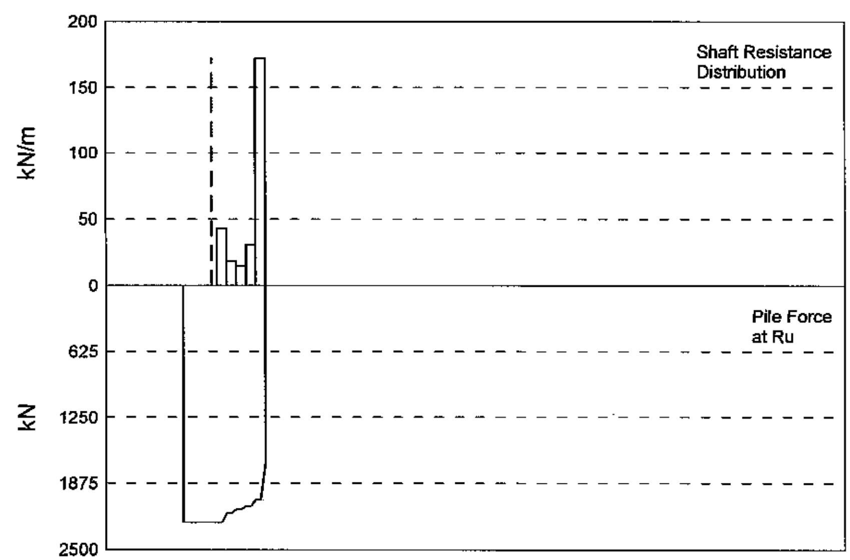
Site 5







Ru = 2243.3 kN
Rs = 558.4 kN
Rb = 1684.8 kN
Dy = 19.4 mm
Dx = 21.7 mm



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[redacted] Blow: 10 CAPWAP(R) 2006-3
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2243.3; along Shaft 558.4; at Toe 1684.8 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2243.3				
1	9.0	2.8	86.1	2157.2	86.1	30.75	24.88	0.744
2	11.0	4.8	36.9	2120.3	123.0	18.45	14.93	0.744
3	13.0	6.8	29.5	2090.7	152.5	14.76	11.94	0.744
4	15.0	8.8	61.5	2029.2	214.0	30.75	24.88	0.744
5	17.0	10.8	344.4	1684.8	558.4	172.20	139.32	0.744
Avg. Shaft			111.7			51.71	41.83	0.744
Toe			1684.8				17645.99	0.080

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	3.674	5.123
Case Damping Factor		0.730	0.237
Damping Type		Smith	
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	42	
Resistance Gap (included in Toe Quake) (mm)			0.044
Soil Plug Weight	(kN)		0.14

CAPWAP match quality = 3.74 (Wave Up Match) ; RSA = 0
Observed: final set = 2.300 mm; blow count = 435 b/m
Computed: final set = 1.292 mm; blow count = 774 b/m

max. Top Comp. Stress = 157.0 MPa (T= 21.7 ms, max= 1.131 x Top)
max. Comp. Stress = 177.6 MPa (Z= 17.0 m, T= 25.4 ms)
max. Tens. Stress = -24.17 MPa (Z= 9.0 m, T= 37.9 ms)
max. Energy (EMX) = 22.90 kJ; max. Measured Top Displ. (DMX)=14.74 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2213.8	-154.0	157.0	-10.92	22.90	3.9	15.114
2	2.0	2213.1	-169.3	157.0	-12.01	22.77	3.9	14.820
3	3.0	2212.4	-210.7	156.9	-14.94	22.59	3.9	14.461
4	4.0	2211.7	-226.7	156.9	-16.08	22.35	3.9	14.064
5	5.0	2212.8	-238.4	156.9	-16.91	22.07	3.9	13.621
6	6.0	2224.0	-238.2	157.7	-16.90	21.74	3.9	13.143
7	7.0	2258.1	-241.0	160.1	-17.09	21.35	3.8	12.632
8	8.0	2317.8	-283.1	164.4	-20.07	20.90	3.8	12.077
9	9.0	2386.1	-340.8	169.2	-24.17	20.40	3.6	11.487
10	10.0	2107.4	-281.8	149.5	-19.99	17.54	3.6	10.892
11	11.0	2138.7	-289.2	151.7	-20.51	16.97	3.5	10.260
12	12.0	2038.6	-256.6	144.6	-18.20	15.45	3.5	9.610
13	13.0	2006.5	-267.7	142.3	-18.99	14.79	3.8	8.939
14	14.0	2072.2	-252.2	147.0	-17.89	13.50	4.0	8.259
15	15.0	2229.4	-268.4	158.1	-19.04	12.81	3.9	7.567
16	16.0	2312.5	-233.4	164.0	-16.55	11.17	3.7	6.881
17	17.0	2504.1	-250.2	177.6	-17.75	7.93	3.2	6.187
Absolute	17.0			177.6			(T =	25.4 ms)
	9.0				-24.17		(T =	37.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	2721.4	2548.2	2375.0	2201.8	2028.6	1855.3	1682.1	1508.9	1335.7	1162.5
RX	2933.0	2831.2	2744.2	2672.3	2604.5	2542.0	2483.1	2428.6	2376.7	2329.1
RU	2721.4	2548.2	2375.0	2201.8	2028.6	1855.3	1682.1	1508.9	1335.7	1162.5

RAU = 1910.2 (kN); RA2 = 2387.7 (kN)

Current CAPWAP Ru = 2243.3 (kN); Corresponding J(RP)= 0.28; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.77	21.47	2145.8	2307.7	2314.3	14.741	2.301	2.300	23.0	2701.9

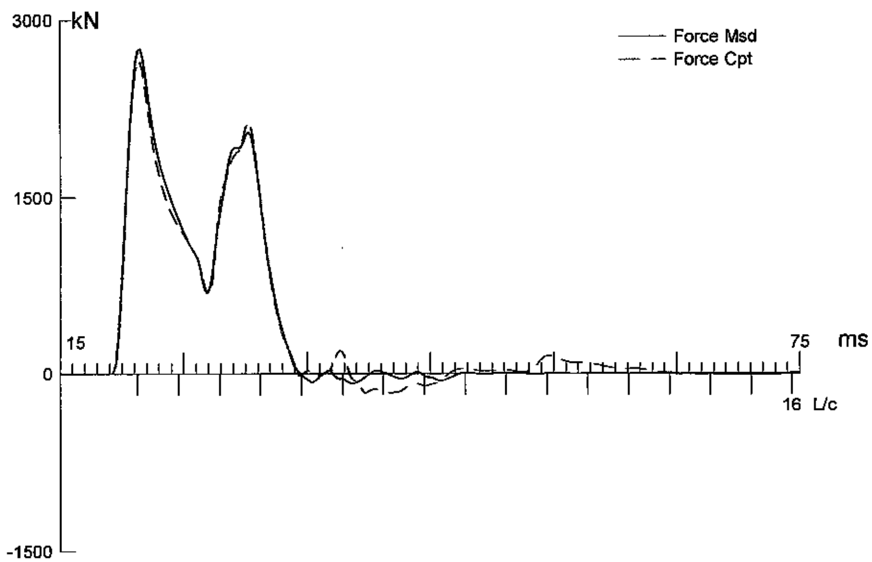
PILE PROFILE AND PILE MODEL

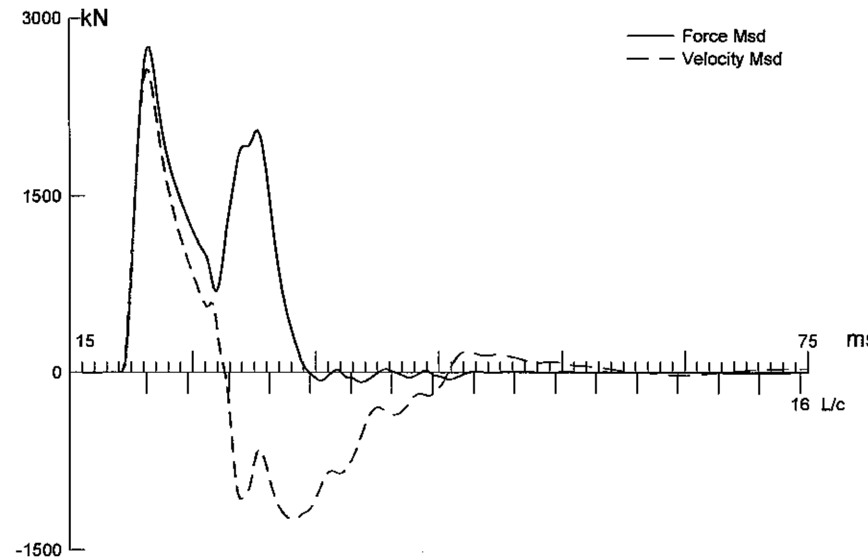
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.00	141.00	206842.7	77.287	1.236

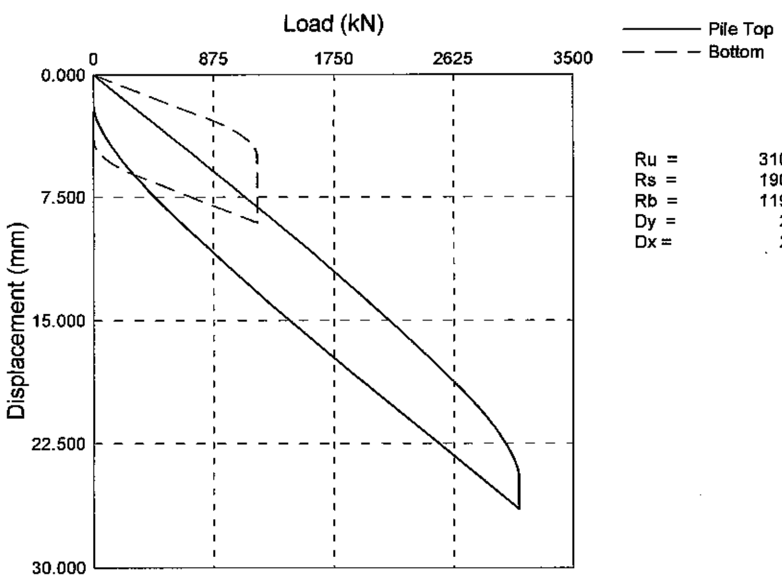
Toe Area 0.095 m^2

Top Segment Length 1.00 m, Top Impedance 569.29 kN/m/s

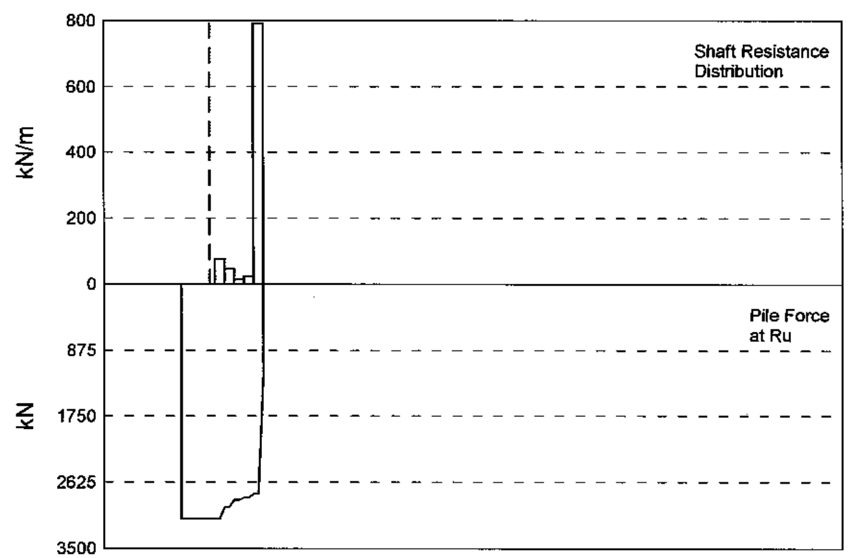
Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 6.6 ms







Ru = 3103.6 kN
Rs = 1908.5 kN
Rb = 1195.2 kN
Dy = 24.6 mm
Dx = 26.5 mm



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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3103.6; along Shaft 1908.5; at Toe 1195.2 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				3103.6				
1	9.0	2.8	153.6	2950.1	153.6	54.84	44.37	0.166
2	11.0	4.8	92.6	2857.5	246.2	46.31	37.47	0.166
3	13.0	6.8	29.3	2828.2	275.5	14.63	11.84	0.166
4	15.0	8.8	48.7	2779.4	324.2	24.37	19.72	0.166
5	17.0	10.8	1584.3	1195.2	1908.5	792.14	640.89	0.166
Avg. Shaft			381.7			176.71	142.97	0.166
Toe			1195.2				12517.32	0.080

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	5.507	3.879
Case Damping Factor		0.556	0.168
Unloading Quake	(% of loading quake)	38	37
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	6	
Resistance Gap (included in Toe Quake) (mm)			0.130

CAPWAP match quality = 4.25 (Wave Up Match) ; RSA = 0
Observed: final set = 1.900 mm; blow count = 526 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 188.5 MPa (T= 21.7 ms, max= 1.255 x Top)
max. Comp. Stress = 236.6 MPa (Z= 17.0 m, T= 25.2 ms)
max. Tens. Stress = -36.58 MPa (Z= 13.0 m, T= 37.5 ms)
max. Energy (EMX) = 33.15 kJ; max. Measured Top Displ. (DMX)=17.65 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2658.4	-206.5	188.5	-14.65	33.15	4.7	17.982
2	2.0	2657.6	-268.8	188.5	-19.06	32.95	4.7	17.612
3	3.0	2657.4	-326.9	188.5	-23.19	32.67	4.7	17.171
4	4.0	2658.9	-369.9	188.6	-26.24	32.31	4.7	16.670
5	5.0	2671.4	-402.6	189.5	-28.55	31.88	4.7	16.138
6	6.0	2694.3	-434.3	191.1	-30.80	31.38	4.6	15.552
7	7.0	2729.6	-431.7	193.6	-30.62	30.80	4.6	14.908
8	8.0	2771.1	-421.5	196.5	-29.89	30.09	4.5	14.225
9	9.0	2814.1	-423.8	199.6	-30.06	29.29	4.4	13.473
10	10.0	2602.7	-427.7	184.6	-30.34	25.86	4.4	12.734
11	11.0	2626.0	-479.8	186.2	-34.03	24.95	4.3	11.939
12	12.0	2501.4	-495.6	177.4	-35.15	22.62	4.3	11.121
13	13.0	2566.3	-515.8	182.0	-36.58	21.53	4.5	10.261
14	14.0	2675.8	-508.8	189.8	-36.09	20.01	4.5	9.368
15	15.0	2868.8	-498.1	203.5	-35.32	18.77	4.3	8.447
16	16.0	3091.4	-478.6	219.2	-33.94	17.06	4.0	7.514
17	17.0	3335.7	-475.5	236.6	-33.72	7.00	3.4	6.563
Absolute	17.0			236.6			(T =	25.2 ms)
	13.0				-36.58		(T =	37.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3438.4	3247.2	3056.0	2864.7	2673.5	2482.3	2291.1	2099.9	1908.7	1717.5
RX	3730.1	3624.6	3534.6	3452.3	3377.7	3307.1	3237.4	3176.2	3115.0	3053.8
RU	3450.3	3260.2	3070.2	2880.2	2690.2	2500.1	2310.1	2120.1	1930.1	1740.0

RAU = 2579.9 (kN); RA2 = 3099.8 (kN)

Current CAPWAP Ru = 3103.6 (kN); Corresponding J(RP)= 0.18; J(RX) = 0.82

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.54	21.47	2586.5	2764.0	2775.1	17.645	1.901	1.900	33.3	3406.2

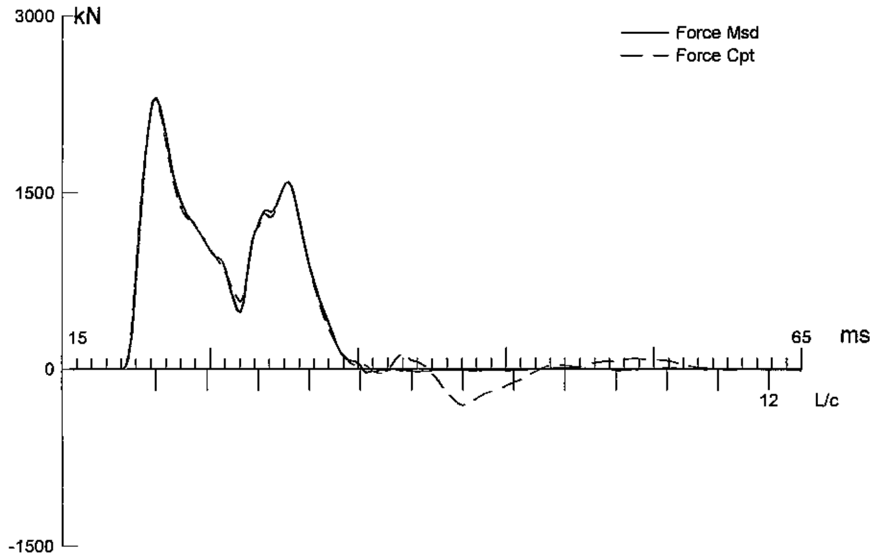
PILE PROFILE AND PILE MODEL

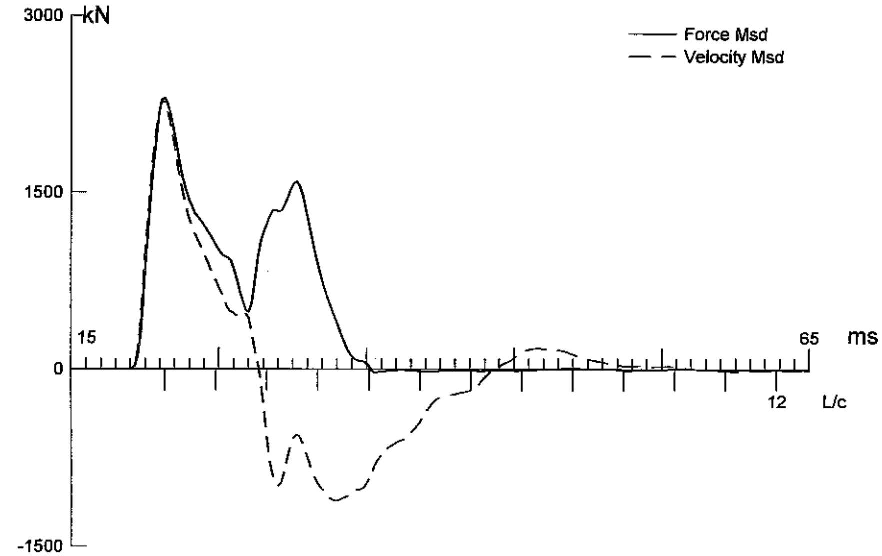
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.00	141.00	206842.7	77.287	1.236

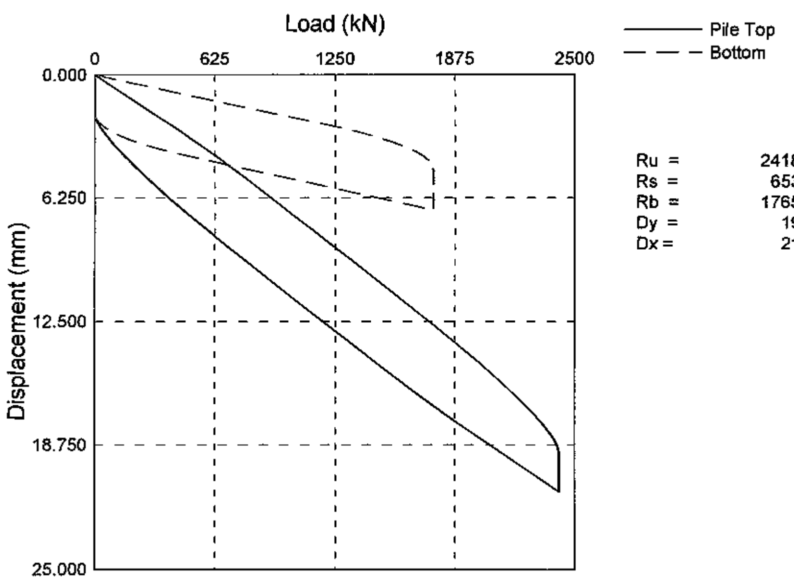
Toe Area 0.095 m^2

Top Segment Length 1.00 m, Top Impedance 569.29 kN/m/s

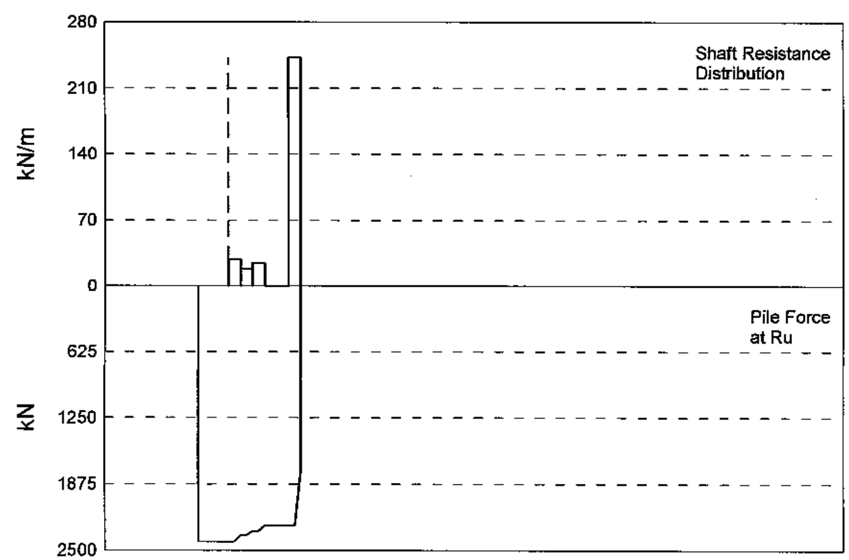
Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 6.6 ms







Ru =	2418.6	kN
Rs =	653.2	kN
Rb =	1765.3	kN
Dy =	19.1	mm
Dx =	21.1	mm



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[redacted]Pile: P5 — Test: 29-Feb-2012 20:23:
RSTRK; Blow: 10 — CAPWAP(R) 2006-3
[redacted] — OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2418.6; along Shaft 653.2; at Toe 1765.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2418.6				
1	7.3	1.3	59.6	2359.0	59.6	46.26	37.43	0.497
2	9.4	3.4	38.2	2320.8	97.8	18.35	14.85	0.497
3	11.5	5.5	51.0	2269.8	148.8	24.47	19.80	0.497
4	13.5	7.5	0.0	2269.8	148.8	0.00	0.00	0.000
5	15.6	9.6	0.0	2269.8	148.8	0.00	0.00	0.000
6	17.7	11.7	504.5	1765.3	653.2	242.26	196.00	0.497
Avg. Shaft			108.9			55.83	45.17	0.497
Toe			1765.3				18488.97	0.128

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.269	3.742
Case Damping Factor		0.570	0.397
Unloading Quake	(% of loading quake)	30	102
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	69	
Resistance Gap (included in Toe Quake) (mm)			0.253

CAPWAP match quality = 3.59 (Wave Up Match) ; RSA = 0
Observed: final set = 2.000 mm; blow count = 500 b/m
Computed: final set = 1.015 mm; blow count = 985 b/m

max. Top Comp. Stress = 162.8 MPa (T= 21.7 ms, max= 1.308 x Top)
max. Comp. Stress = 213.1 MPa (Z= 17.7 m, T= 25.4 ms)
max. Tens. Stress = -36.59 MPa (Z= 17.7 m, T= 38.8 ms)
max. Energy (EMX) = 23.94 kJ; max. Measured Top Displ. (DMX)=15.41 mm

[redacted] Pile: P5 Test: 29-Feb-2012 20:23:
RSTRK; Blow: 10 CAPWAP(R) 2006-3
[redacted] OP: GR

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2295.9	-309.9	162.8	-21.98	23.94	4.0	15.043
2	2.1	2296.4	-317.3	162.9	-22.51	23.79	4.0	14.709
3	3.1	2304.0	-323.0	163.4	-22.91	23.60	4.0	14.349
4	4.2	2324.7	-324.5	164.9	-23.01	23.37	4.0	13.947
5	5.2	2354.6	-334.0	167.0	-23.69	23.07	3.9	13.479
6	6.2	2393.8	-358.8	169.8	-25.45	22.70	3.9	12.972
7	7.3	2421.0	-394.9	171.7	-28.01	22.28	3.8	12.415
8	8.3	2282.6	-392.7	161.9	-27.85	20.20	3.8	11.843
9	9.4	2306.9	-417.9	163.6	-29.64	19.67	3.7	11.218
10	10.4	2219.8	-414.9	157.4	-29.43	18.13	3.7	10.564
11	11.5	2225.7	-432.2	157.9	-30.65	17.44	3.7	9.856
12	12.5	2095.7	-398.4	148.6	-28.25	15.59	3.7	9.128
13	13.5	2174.2	-412.1	154.2	-29.23	14.76	3.7	8.363
14	14.6	2293.8	-429.7	162.7	-30.48	13.92	3.7	7.580
15	15.6	2494.2	-459.9	176.9	-32.61	13.04	3.5	6.786
16	16.7	2762.5	-498.8	195.9	-35.38	12.13	3.1	5.980
17	17.7	3004.1	-515.9	213.1	-36.59	7.78	2.5	5.164
Absolute	17.7			213.1			(T =	25.4 ms)
	17.7				-36.59		(T =	38.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3135.8	2985.3	2834.7	2684.1	2533.6	2383.0	2232.4	2081.8	1931.3	1780.7
RX	3283.6	3169.5	3059.9	2950.3	2849.4	2748.5	2651.4	2560.3	2477.3	2398.0
RU	3146.2	2996.7	2847.2	2697.6	2548.1	2398.6	2249.0	2099.5	1950.0	1800.4

RAU = 1882.1 (kN); RA2 = 2287.8 (kN)

Current CAPWAP Ru = 2418.6 (kN); Corresponding J(RP)= 0.48; J(RX) = 0.87

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.06	21.54	2308.6	2333.0	2333.0	15.408	2.001	2.000	24.2	2780.8

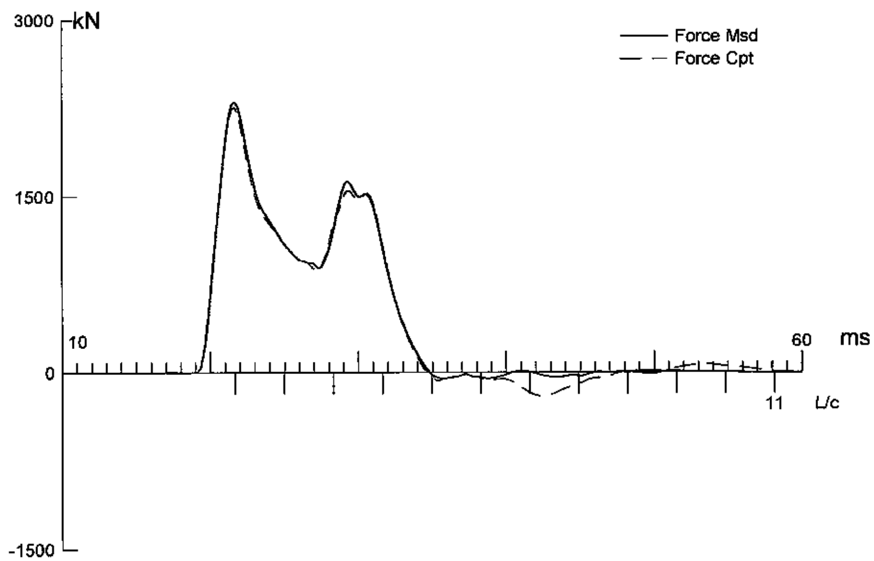
PILE PROFILE AND PILE MODEL

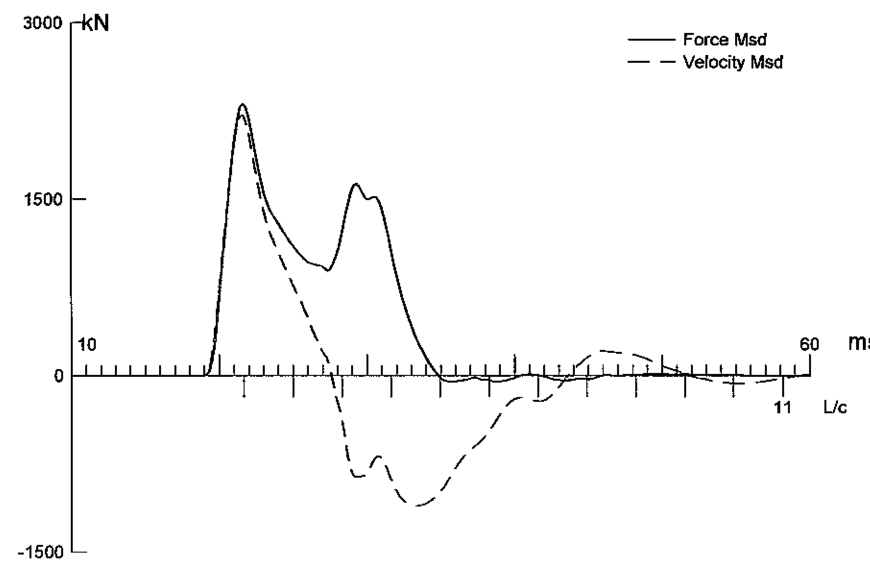
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.70	141.00	206842.7	77.287	1.236

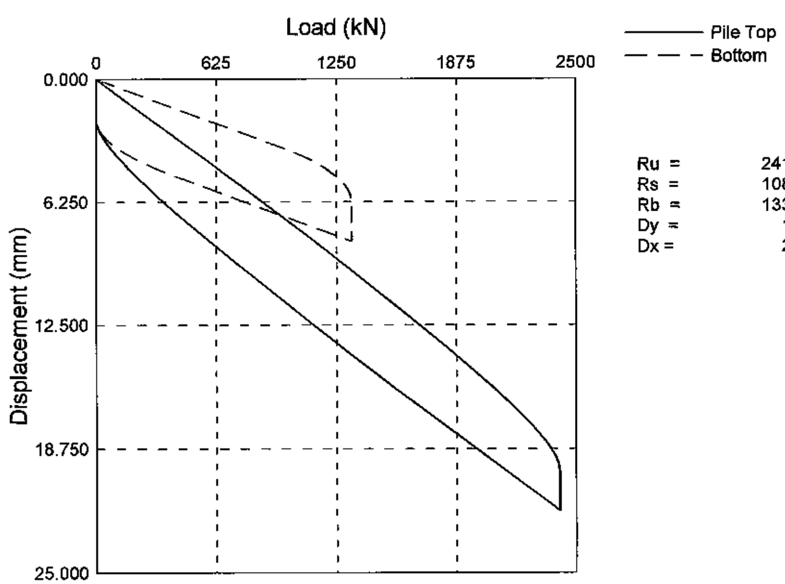
Toe Area 0.095 m^2

Top Segment Length 1.04 m, Top Impedance 569.29 kN/m/s

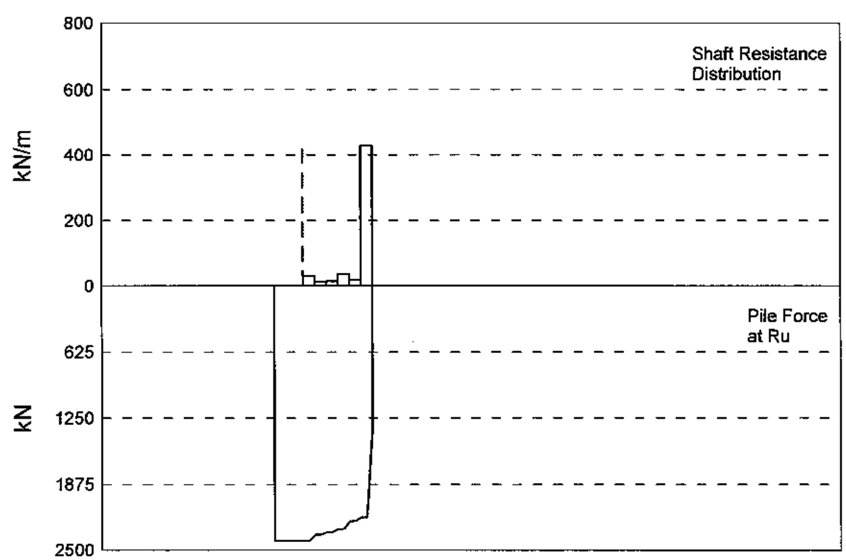
Pile Damping 1.0 %, Time Incr 0.203 ms, Wave Speed 5123.0 m/s, 2L/c 6.9 ms







Ru =	2413.4	kN
Rs =	1081.6	kN
Rb =	1331.9	kN
Dy =	19.9	mm
Dx =	21.9	mm



[redacted] Pile: P11 | Test: 29-Feb-2012 20:06:
[redacted] | CAPWAP(R) 2006-3
OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2413.4; along Shaft 1081.6; at Toe 1331.9 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2413.4				
1	7.0	1.4	60.0	2353.4	60.0	42.88	34.69	0.548
2	9.0	3.4	24.5	2328.9	84.5	12.25	9.91	0.548
3	11.0	5.4	29.4	2299.5	113.9	14.70	11.89	0.548
4	13.0	7.4	73.5	2226.0	187.4	36.75	29.73	0.548
5	15.0	9.4	36.8	2189.3	224.2	18.38	14.87	0.548
6	17.0	11.4	857.4	1331.9	1081.6	428.69	346.84	0.548
Avg. Shaft			180.3			94.87	76.76	0.548
Toe			1331.9				13949.41	0.167

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	4.525	4.815
Case Damping Factor		1.041	0.391
Damping Type			Smith
Unloading Quake	(% of loading quake)	87	27
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	76	
Resistance Gap (included in Toe Quake) (mm)			0.527
Soil Plug Weight	(kN)		0.94

CAPWAP match quality = 2.51 (Wave Up Match) ; RSA = 0
Observed: final set = 2.000 mm; blow count = 500 b/m
Computed: final set = 0.989 mm; blow count = 1011 b/m

max. Top Comp. Stress = 160.4 MPa (T= 22.1 ms, max= 1.208 x Top)
max. Comp. Stress = 193.7 MPa (Z= 17.0 m, T= 25.6 ms)
max. Tens. Stress = -24.50 MPa (Z= 17.0 m, T= 38.6 ms)
max. Energy (EMX) = 24.32 kJ; max. Measured Top Displ. (DMX)=15.17 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2261.1	-216.0	160.4	-15.32	24.32	4.0	15.082
2	2.0	2268.4	-230.0	160.9	-16.31	24.12	4.0	14.703
3	3.0	2283.3	-240.3	161.9	-17.04	23.88	3.9	14.302
4	4.0	2303.7	-248.9	163.4	-17.65	23.60	3.9	13.861
5	5.0	2328.6	-256.9	165.1	-18.22	23.28	3.8	13.394
6	6.0	2356.6	-262.3	167.1	-18.60	22.91	3.8	12.880
7	7.0	2383.5	-285.5	169.0	-20.25	22.46	3.7	12.335
8	8.0	2229.8	-255.7	158.1	-18.14	20.38	3.7	11.763
9	9.0	2260.0	-281.9	160.3	-19.99	19.85	3.7	11.161
10	10.0	2221.2	-281.3	157.5	-19.95	18.67	3.6	10.523
11	11.0	2264.6	-300.3	160.6	-21.30	18.01	3.5	9.849
12	12.0	2256.3	-299.1	160.0	-21.21	16.67	3.4	9.146
13	13.0	2294.8	-333.4	162.8	-23.65	15.88	3.3	8.404
14	14.0	2204.9	-287.4	156.4	-20.38	13.82	3.3	7.655
15	15.0	2358.9	-318.4	167.3	-22.58	12.96	3.2	6.885
16	16.0	2516.4	-321.8	178.5	-22.82	11.67	2.9	6.130
17	17.0	2731.6	-345.5	193.7	-24.50	4.46	2.5	5.379
Absolute	17.0			193.7			(T =	25.6 ms)
	17.0				-24.50		(T =	38.6 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	2979.6	2823.4	2667.2	2511.1	2354.9	2198.8	2042.6	1886.5	1730.3	1574.2
RX	3146.4	3055.1	2967.0	2890.4	2823.1	2758.8	2701.1	2645.0	2595.5	2550.6
RU	2995.3	2840.8	2686.2	2531.6	2377.0	2222.5	2067.9	1913.3	1758.7	1604.2

RAU = 2071.4 (kN); RA2 = 2498.0 (kN)

Current CAPWAP Ru = 2413.5 (kN); RMX requires J > 0.9;

Check with PDA-W; RA2 may be a better Case Method

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.91	21.86	2223.3	2317.7	2317.7	15.167	2.001	2.000	24.5	2853.0

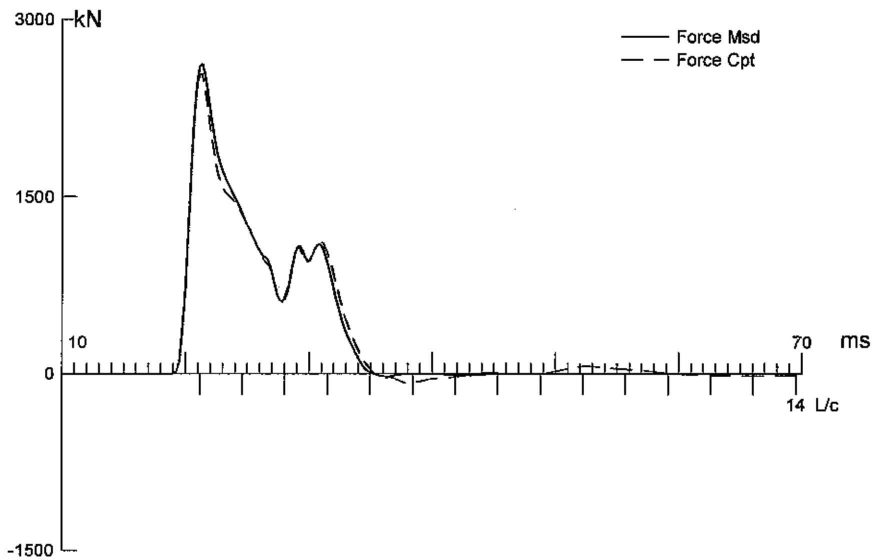
PILE PROFILE AND PILE MODEL

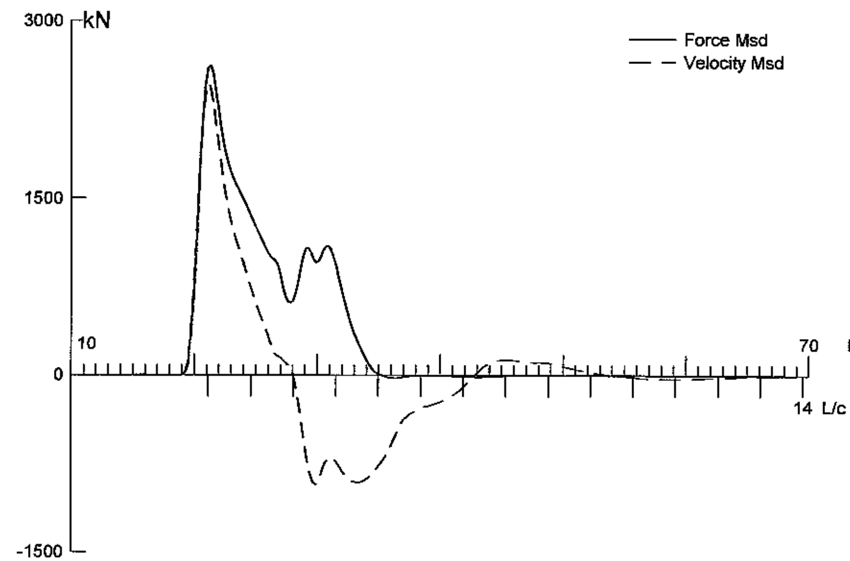
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.00	141.00	206842.7	77.287	1.236

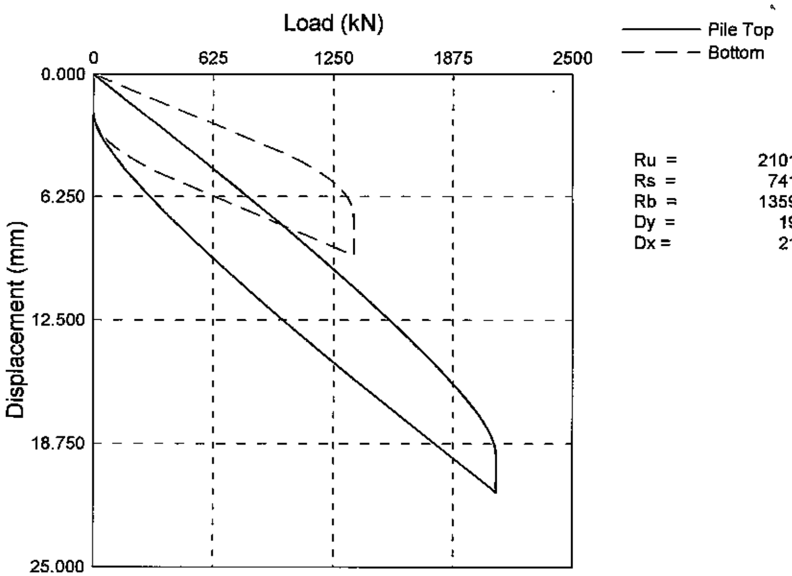
Toe Area 0.095 m^2

Top Segment Length 1.00 m, Top Impedance 569.29 kN/m/s

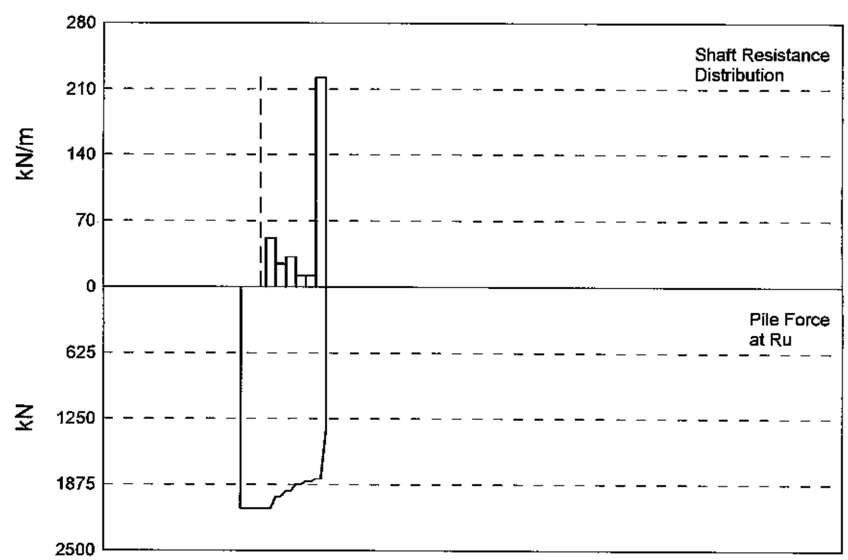
Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 6.6 ms







Ru = 2101.2 kN
Rs = 741.6 kN
Rb = 1359.6 kN
Dy = 19.3 mm
Dx = 21.2 mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2101.2; along Shaft 741.6; at Toe 1359.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2101.2				
1	7.3	2.1	108.4	1992.8	108.4	51.91	42.00	0.694
2	9.4	4.2	51.5	1941.3	159.9	24.72	20.00	0.694
3	11.5	6.3	66.9	1874.4	226.8	32.14	26.00	0.694
4	13.5	8.3	25.7	1848.7	252.5	12.36	10.00	0.694
5	15.6	10.4	25.7	1822.9	278.3	12.36	10.00	0.694
6	17.7	12.5	463.3	1359.6	741.6	222.50	180.02	0.694
Avg. Shaft			123.6			59.33	48.00	0.694
Toe			1359.6				14239.63	0.079

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	5.410	5.566
Case Damping Factor		0.904	0.189
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake)	(mm)		1.009
Soil Plug Weight	(kN)		0.62

```
CAPWAP match quality    =    2.13         (Wave Up Match) ; RSA = 0
Observed: final set     =    2.000 mm;    blow count      =     500 b/m
Computed: final set     =    2.654 mm;    blow count      =     377 b/m

max. Top Comp. Stress   =    180.0 MPa     (T=  21.7 ms, max= 1.098 x Top)
max. Comp. Stress       =    197.7 MPa     (Z=   7.3 m, T=  23.0 ms)
max. Tens. Stress       =   -21.52 MPa     (Z=   7.3 m, T=  36.6 ms)
max. Energy (EMX)       =    28.24 kJ;     max. Measured Top Displ. (DMX)=15.05 mm
```

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2538.6	-94.0	180.0	-6.67	28.24	4.5	15.464
2	2.1	2540.2	-136.5	180.2	-9.68	28.14	4.5	15.198
3	3.1	2556.6	-181.8	181.3	-12.89	28.00	4.4	14.881
4	4.2	2600.4	-221.6	184.4	-15.72	27.81	4.4	14.524
5	5.2	2665.0	-255.9	189.0	-18.15	27.58	4.3	14.113
6	6.2	2732.1	-282.7	193.8	-20.05	27.30	4.2	13.674
7	7.3	2787.7	-303.4	197.7	-21.52	26.97	4.1	13.180
8	8.3	2463.3	-207.0	174.7	-14.68	22.98	4.0	12.685
9	9.4	2508.9	-210.7	177.9	-14.94	22.58	3.9	12.138
10	10.4	2375.5	-162.2	168.5	-11.50	20.53	3.8	11.574
11	11.5	2403.6	-167.0	170.5	-11.84	20.02	3.8	10.970
12	12.5	2213.1	-116.6	157.0	-8.27	17.66	3.7	10.359
13	13.5	2253.6	-112.2	159.8	-7.96	17.11	3.7	9.736
14	14.6	2183.1	-101.8	154.8	-7.22	15.93	3.6	9.107
15	15.6	2147.3	-111.8	152.3	-7.93	15.36	3.7	8.467
16	16.7	2109.2	-114.7	149.6	-8.14	14.26	3.8	7.831
17	17.7	2343.3	-127.4	166.2	-9.04	7.14	3.6	7.187
Absolute	7.3			197.7			(T =	23.0 ms)
	7.3				-21.52		(T =	36.6 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	2874.3	2648.3	2422.3	2196.4	1970.4	1744.4	1518.5	1292.5	1066.5	840.6
RX	2949.7	2763.4	2618.9	2503.4	2405.3	2315.3	2228.5	2146.3	2064.1	1987.2
RU	2874.3	2648.3	2422.3	2196.4	1970.4	1744.4	1518.5	1292.5	1066.5	840.6

RAU = 1049.2 (kN); RA2 = 2305.6 (kN)

Current CAPWAP Ru = 2101.2 (kN); Corresponding J(RP)= 0.34; J(RX) = 0.75

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.39	21.34	2496.9	2637.0	2653.6	15.045	2.001	2.000	28.3	3318.9

PILE PROFILE AND PILE MODEL

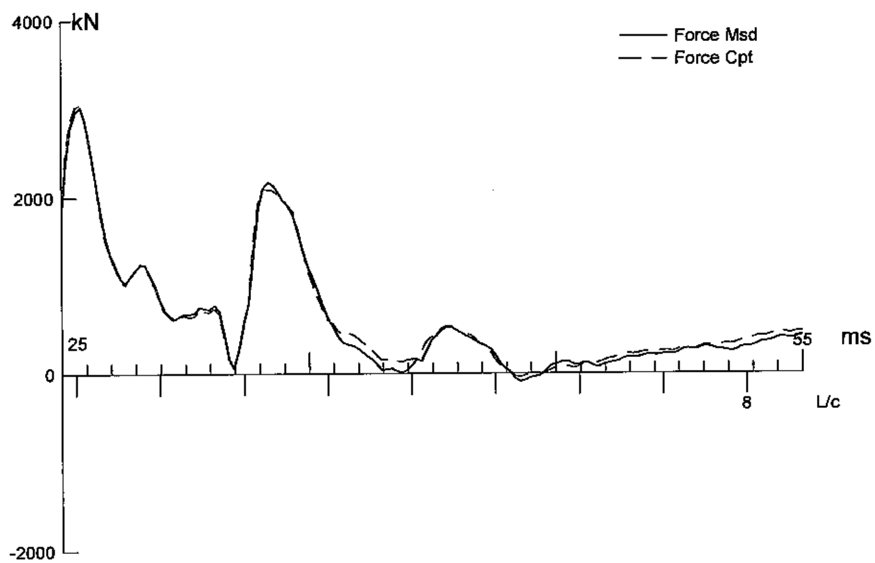
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
17.70	141.00	206842.7	77.287	1.236

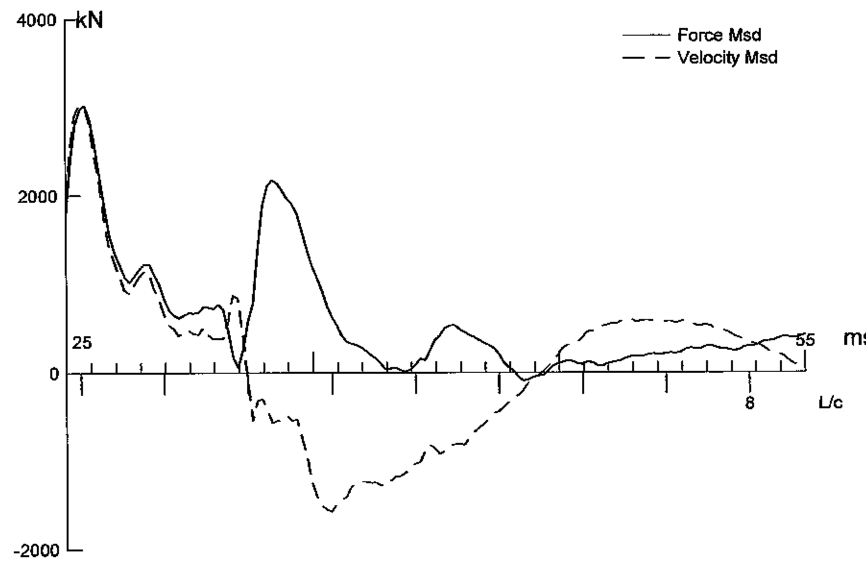
Toe Area 0.095 m^2

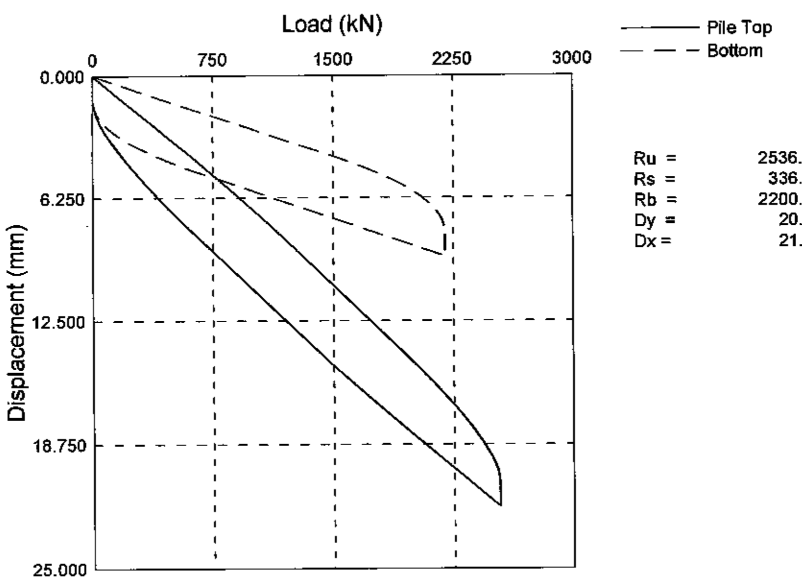
Top Segment Length 1.04 m, Top Impedance 569.29 kN/m/s

Pile Damping 1.0 %, Time Incr 0.203 ms, Wave Speed 5123.0 m/s, 2L/c 6.9 ms

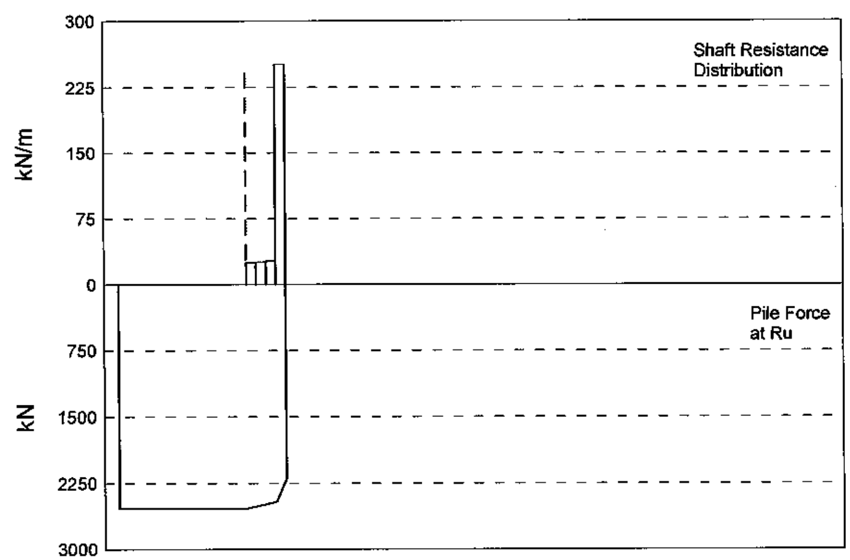
Site 6







Ru =	2536.9	kN
Rs =	336.9	kN
Rb =	2200.0	kN
Dy =	20.6	mm
Dx =	21.9	mm



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EOID; Blow: 231
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Test: 05-Jul-2012 16:33:
CAPWAP(R) 2006-3
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2536.9; along Shaft 336.9; at Toe 2200.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2536.9				
1	14.3	0.9	25.9	2511.0	25.9	27.87	19.25	0.500
2	15.4	2.0	26.3	2484.7	52.2	25.70	17.75	0.500
3	16.4	3.0	27.5	2457.2	79.7	26.87	18.56	0.500
4	17.4	4.0	257.2	2200.0	336.9	251.29	173.54	0.500
Avg. Shaft			84.2			84.22	58.17	0.500
Toe			2200.0				16803.77	0.382

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.833	6.101
Case Damping Factor		0.248	1.239
Damping Type			Smith
Unloading Quake	(% of loading quake)	75	98
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			0.319
Soil Plug Weight	(kN)		0.03

CAPWAP match quality = 2.18 (Wave Up Match) ; RSA = 0
Observed: final set = 1.300 mm; blow count = 769 b/m
Computed: final set = 0.756 mm; blow count = 1323 b/m

max. Top Comp. Stress = 181.8 MPa (T= 26.2 ms, max= 1.209 x Top)
max. Comp. Stress = 219.7 MPa (Z= 17.4 m, T= 29.6 ms)
max. Tens. Stress = -36.59 MPa (Z= 17.4 m, T= 42.4 ms)
max. Energy (EMX) = 22.90 kJ; max. Measured Top Displ. (DMX)=13.79 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3053.5	-98.0	181.8	-5.83	22.90	4.4	14.024
2	2.0	3059.4	-155.2	182.1	-9.24	22.75	4.4	13.735
3	3.1	3064.7	-214.9	182.4	-12.79	22.57	4.4	13.421
4	4.1	3069.3	-253.5	182.7	-15.09	22.37	4.4	13.083
5	5.1	3073.7	-292.6	183.0	-17.42	22.14	4.4	12.731
6	6.1	3078.0	-313.1	183.2	-18.63	21.95	4.4	12.387
7	7.2	3082.5	-327.3	183.5	-19.48	21.75	4.4	12.047
8	8.2	3087.3	-325.8	183.8	-19.39	21.56	4.4	11.721
9	9.2	3092.3	-329.0	184.1	-19.58	21.28	4.3	11.332
10	10.2	3097.4	-336.1	184.4	-20.01	20.84	4.3	10.830
11	11.3	3104.2	-411.0	184.8	-24.46	20.22	4.3	10.217
12	12.3	3124.9	-463.7	186.0	-27.60	19.40	4.3	9.536
13	13.3	3162.8	-521.3	188.3	-31.03	18.71	4.2	8.898
14	14.3	3141.5	-578.3	187.0	-34.42	18.09	4.4	8.280
15	15.4	2824.5	-603.5	168.1	-35.92	16.75	5.0	7.596
16	16.4	3156.7	-607.8	187.9	-36.18	15.31	4.9	6.860
17	17.4	3690.6	-614.7	219.7	-36.59	11.78	4.2	6.119
Absolute	17.4			219.7			(T =	29.6 ms)
	17.4				-36.59		(T =	42.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3277.4	2989.0	2700.6	2412.1	2123.7	1835.3	1546.9	1258.5	970.1	681.7
RX	3915.0	3693.8	3472.7	3313.6	3186.2	3069.6	2994.4	2965.1	2935.7	2906.4
RU	3277.4	2989.0	2700.6	2412.1	2123.7	1835.3	1546.9	1258.5	970.1	681.7

RAU = 2611.3 (kN); RA2 = 2720.4 (kN)

Current CAPWAP Ru = 2536.9 (kN); Corresponding J(RP)= 0.26;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.65	25.77	3152.1	3009.3	3104.2	13.792	1.336	1.300	23.1	3061.3

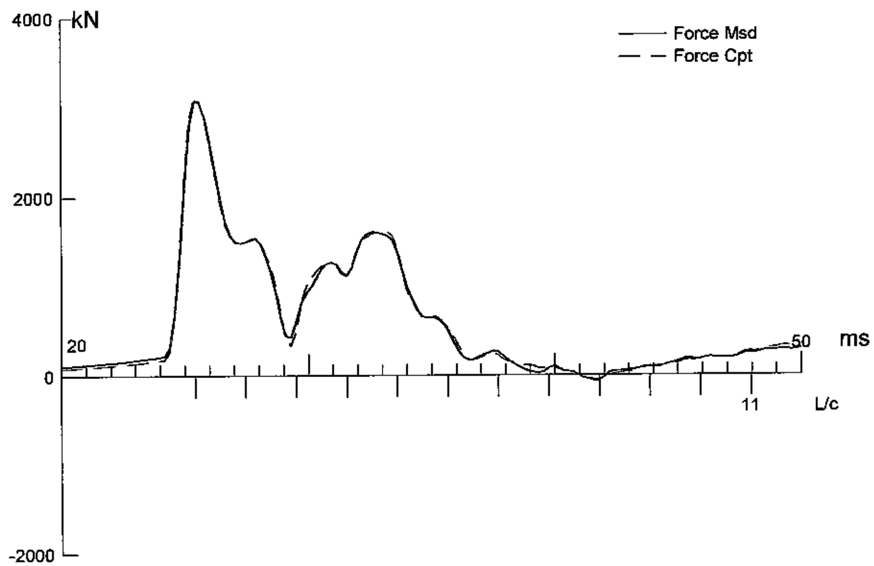
PILE PROFILE AND PILE MODEL

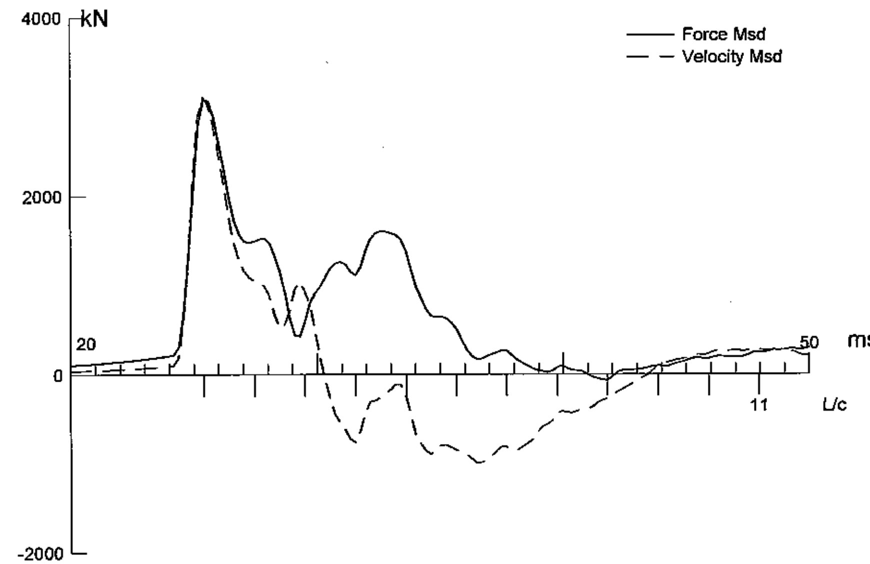
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
17.40	168.00	206842.7	77.287	1.448

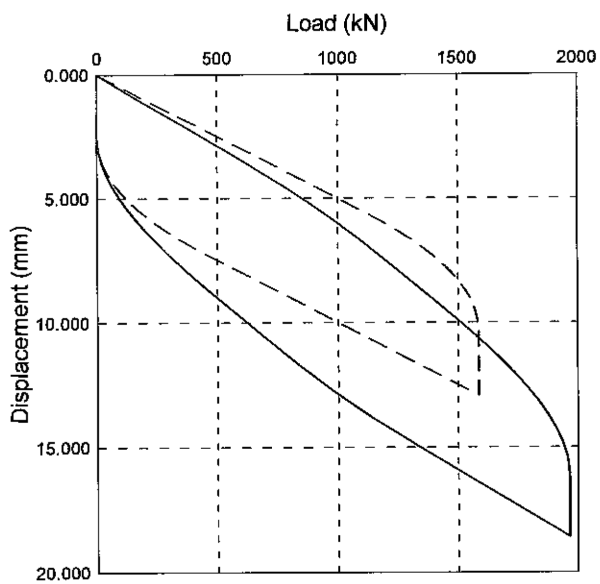
Toe Area 0.131 m^2

Top Segment Length 1.02 m, Top Impedance 678.30 kN/m/s

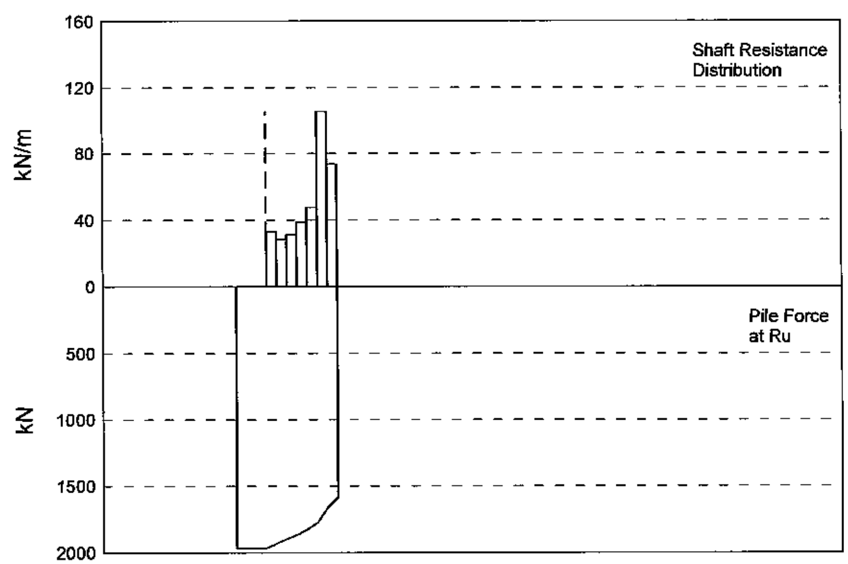
Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 5123.0 m/s, 2L/c 6.8 ms







Ru =	1964.2	kN
Rs =	377.0	kN
Rb =	1587.2	kN
Dy =	16.0	mm
Dx =	18.6	mm



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[redacted] Pile: 11 SU1 | Test: 13-Jul-2012 10:00:
EOID; Blow: 20 | CAPWAP(R) 2006-3
[redacted] | OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1964.2; along Shaft 377.0; at Toe 1587.2 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				1964.2				
1	4.2	0.3	35.0	1929.2	35.0	116.67	80.57	0.794
2	5.3	1.3	30.0	1899.2	65.0	28.57	19.73	0.794
3	6.3	2.4	33.0	1866.2	98.0	31.43	21.70	0.794
4	7.4	3.5	41.0	1825.2	139.0	39.05	26.97	0.794
5	8.4	4.5	50.0	1775.2	189.0	47.62	32.89	0.794
6	9.5	5.6	111.0	1664.2	300.0	105.71	73.01	0.794
7	10.5	6.6	77.0	1587.2	377.0	73.33	50.64	0.794
Avg. Shaft			53.9			57.12	39.45	0.794
Toe			1587.2				12123.16	0.311

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	3.452	7.935
Case Damping Factor		0.441	0.728
Damping Type			Smith
Unloading Quake	(% of loading quake)	61	84
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	55	
Resistance Gap (included in Toe Quake)	(mm)		0.660
Soil Plug Weight	(kN)		0.14

CAPWAP match quality = 1.59 (Wave Up Match) ; RSA = 0
Observed: final set = 2.600 mm; blow count = 385 b/m
Computed: final set = 2.551 mm; blow count = 392 b/m

max. Top Comp. Stress = 186.5 MPa (T= 25.8 ms, max= 1.039 x Top)
max. Comp. Stress = 193.8 MPa (Z= 4.2 m, T= 26.4 ms)
max. Tens. Stress = -8.85 MPa (Z= 4.2 m, T= 42.8 ms)
max. Energy (EMX) = 22.86 kJ; max. Measured Top Displ. (DMX)=12.41 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.1	3132.9	-77.7	186.5	-4.63	22.86	4.5	12.405
2	2.1	3148.1	-100.3	187.4	-5.97	22.52	4.5	11.959
3	3.2	3192.6	-124.7	190.0	-7.42	22.14	4.4	11.488
4	4.2	3256.3	-148.7	193.8	-8.85	21.84	4.3	11.058
5	5.3	3171.5	-143.2	188.8	-8.52	20.40	4.2	10.609
6	6.3	3122.6	-142.7	185.9	-8.49	19.09	4.1	10.117
7	7.4	3078.2	-138.1	183.2	-8.22	17.68	4.0	9.587
8	8.4	2827.5	-118.3	168.3	-7.04	16.06	4.6	9.055
9	9.5	2119.1	-94.2	126.1	-5.60	14.23	5.0	8.531
10	10.5	2200.2	-27.0	131.0	-1.61	10.02	4.8	8.035
Absolute	4.2			193.8			(T =	26.4 ms)
	4.2				-8.85		(T =	42.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	2912.1	2576.1	2240.1	1904.2	1568.2	1232.2	896.2	560.2	224.3	0.0
RX	3098.9	2822.4	2560.3	2315.4	2158.1	2086.0	2031.4	1999.0	1966.7	1934.4
RU	2912.1	2576.1	2240.1	1904.2	1568.2	1232.2	896.2	560.2	224.3	0.0

RAU = 1869.5 (kN); RA2 = 2227.5 (kN)

Current CAPWAP Ru = 1964.2 (kN); Corresponding J(RP)= 0.28; J(RX) = 0.81

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.65	25.62	3153.2	3118.6	3149.0	12.415	2.598	2.600	23.4	3119.8

PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
10.50	168.00	206842.7	77.287	1.448

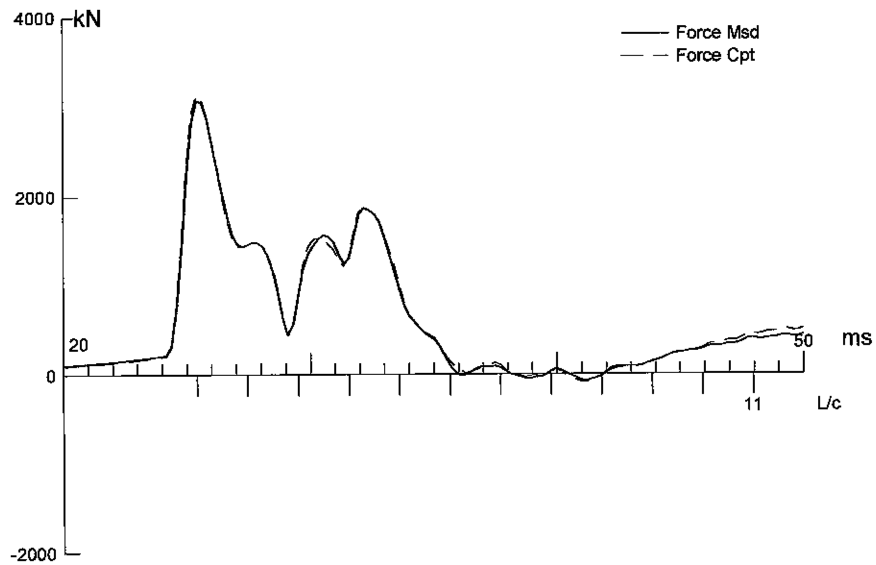
Toe Area 0.131 m^2

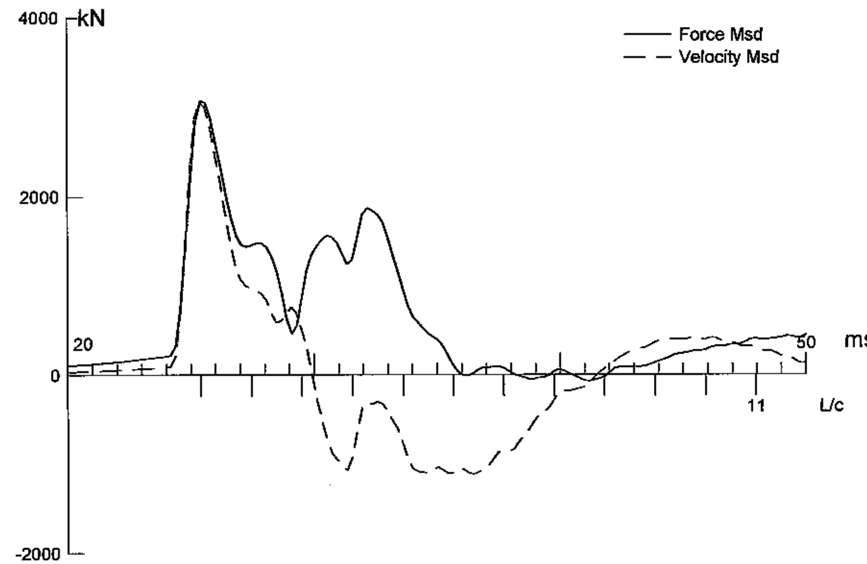
Top Segment Length 1.05 m, Top Impedance 678.30 kN/m/s

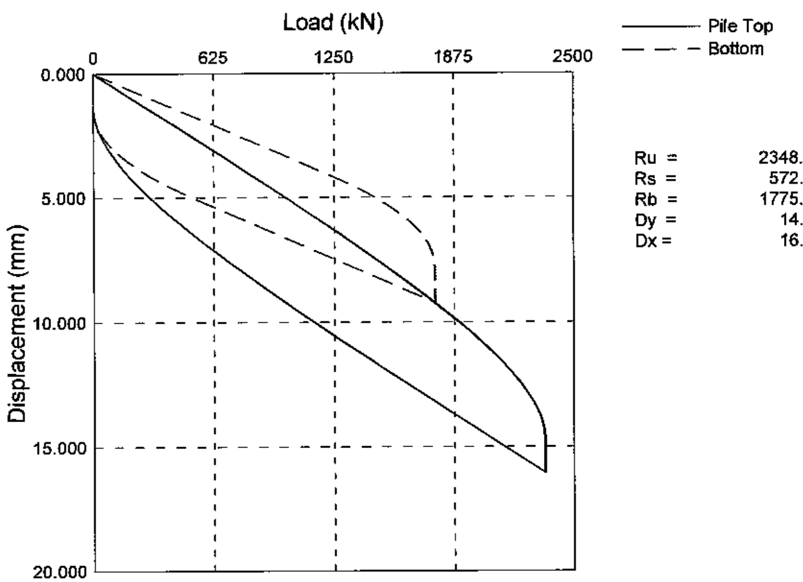
Pile Damping 1.0 %, Time Incr 0.205 ms, Wave Speed 5123.0 m/s, 2L/c 4.1 ms

[redacted] 11A SU1; EOID; Blow: 125 (Test: 13-Jul-2012 10:13:)

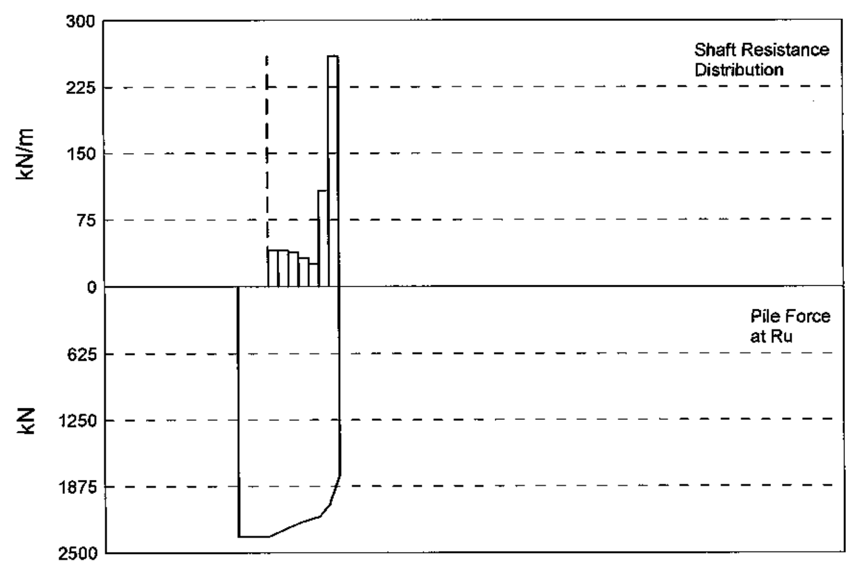
13-Jul-2012
CAPWAP(R) 2006-3







Ru =	2348.2	kN
Rs =	572.5	kN
Rb =	1775.7	kN
Dy =	14.6	mm
Dx =	16.1	mm



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[redacted] Pile: 11A SU1
EOID: Blow: 125
[redacted]

Test: 13-Jul-2012 10:13:
CAPWAP(R) 2006-3
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2348.2; along Shaft 572.5; at Toe 1775.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2348.2				
1	4.2	0.3	42.7	2305.5	42.7	142.34	98.30	0.576
2	5.3	1.3	42.3	2263.2	85.0	40.28	27.82	0.576
3	6.3	2.4	40.7	2222.5	125.7	38.76	26.77	0.576
4	7.4	3.5	33.6	2188.9	159.3	32.00	22.10	0.576
5	8.4	4.5	26.9	2162.0	186.2	25.62	17.69	0.576
6	9.5	5.6	113.2	2048.8	299.4	107.80	74.45	0.576
7	10.5	6.6	273.1	1775.7	572.5	260.09	179.62	0.576
Avg. Shaft			81.8			86.74	59.90	0.576
Toe			1775.7				13562.93	0.297

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	4.321	5.951
Case Damping Factor		0.486	0.778
Damping Type			Smith
Unloading Quake	(% of loading quake)	53	98
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	43	

CAPWAP match quality = 1.50 (Wave Up Match) ; RSA = 0
Observed: final set = 1.500 mm; blow count = 667 b/m
Computed: final set = 0.684 mm; blow count = 1463 b/m

max. Top Comp. Stress = 186.6 MPa (T= 25.8 ms, max= 1.036 x Top)
max. Comp. Stress = 193.3 MPa (Z= 4.2 m, T= 26.4 ms)
max. Tens. Stress = -19.76 MPa (Z= 9.5 m, T= 40.2 ms)
max. Energy (EMX) = 22.31 kJ; max. Measured Top Displ. (DMX)=11.59 mm

Analysis: 13-Jul-2012

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.1	3135.3	-121.1	186.6	-7.21	22.31	4.4	11.237
2	2.1	3147.2	-164.4	187.3	-9.79	22.07	4.4	10.846
3	3.2	3187.5	-219.2	189.7	-13.05	21.76	4.3	10.434
4	4.2	3246.7	-271.1	193.3	-16.14	21.33	4.3	9.925
5	5.3	3174.5	-281.8	189.0	-16.78	19.82	4.2	9.347
6	6.3	3099.2	-292.7	184.5	-17.42	18.27	4.1	8.715
7	7.4	3037.8	-304.6	180.8	-18.13	16.74	4.0	8.056
8	8.4	2805.2	-317.5	167.0	-18.90	15.38	4.5	7.392
9	9.5	2641.2	-332.0	157.2	-19.76	14.19	4.7	6.734
10	10.5	2903.6	-281.3	172.8	-16.75	9.20	4.3	6.108
Absolute	4.2			193.3			(T =	26.4 ms)
	9.5				-19.76		(T =	40.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3212.3	2912.9	2613.5	2314.1	2014.7	1715.3	1416.0	1116.6	817.2	517.8
RX	3555.2	3317.9	3090.4	2919.0	2776.6	2687.8	2632.0	2578.6	2531.1	2494.8
RU	3212.3	2912.9	2613.5	2314.1	2014.7	1715.3	1416.0	1116.6	817.2	517.8

RAU = 2144.5 (kN); RA2 = 2707.5 (kN)

Current CAPWAP Ru = 2348.2 (kN); Corresponding J(RP)= 0.29;

RMX requires higher damping; see PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.58	25.62	3109.4	3096.9	3120.2	11.591	1.499	1.500	22.9	3497.9

PILE PROFILE AND PILE MODEL

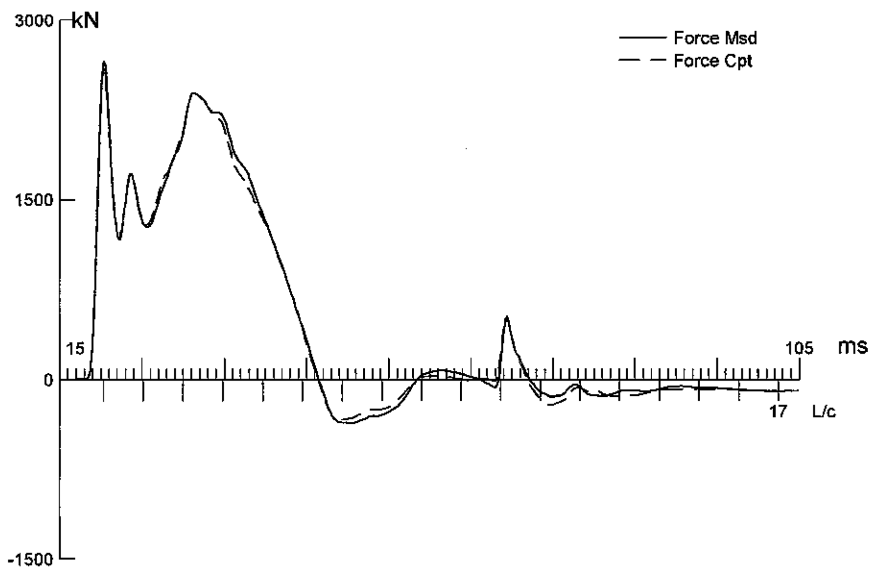
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	168.00	206842.7	77.287	1.448
10.50	168.00	206842.7	77.287	1.448

Toe Area 0.131 m^2

Top Segment Length 1.05 m, Top Impedance 678.30 kN/m/s

Pile Damping 1.0 %, Time Incr 0.205 ms, Wave Speed 5123.0 m/s, 2L/c 4.1 ms

Site 7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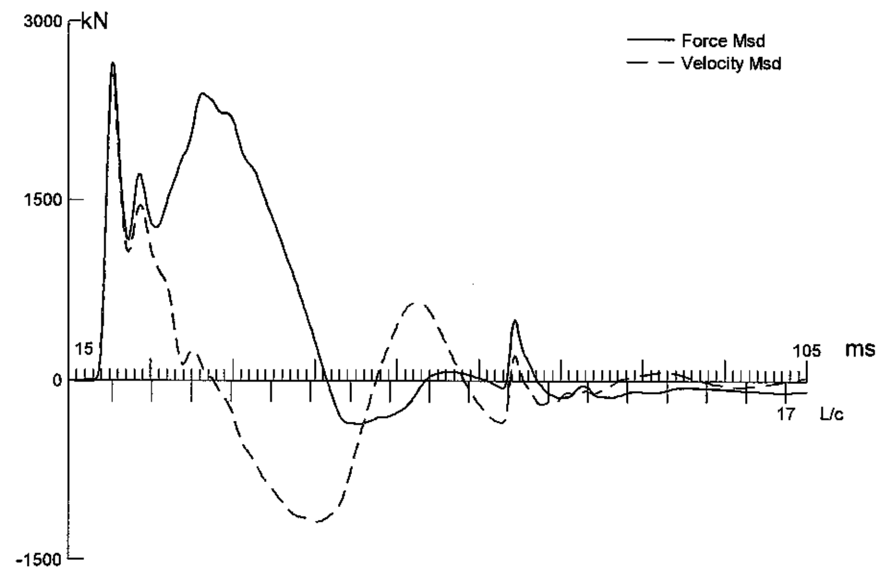
Force Cpt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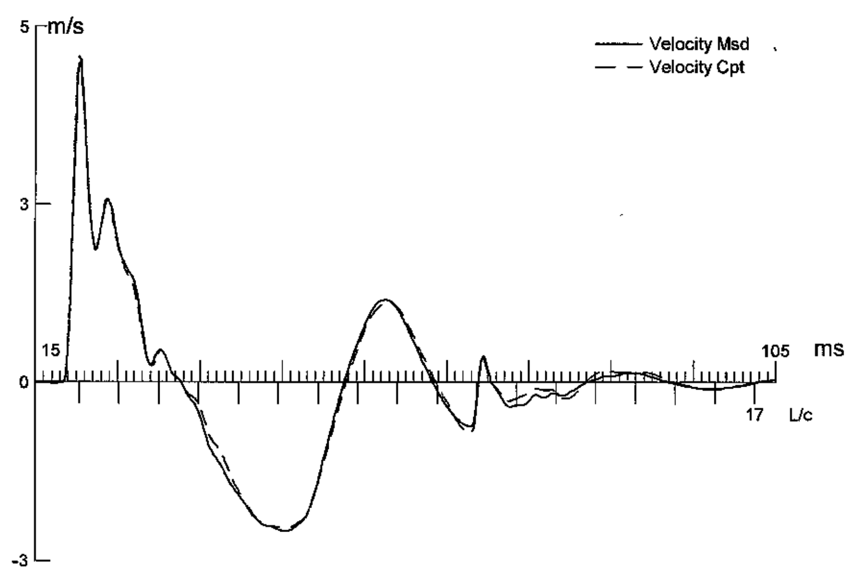
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m/s
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ms
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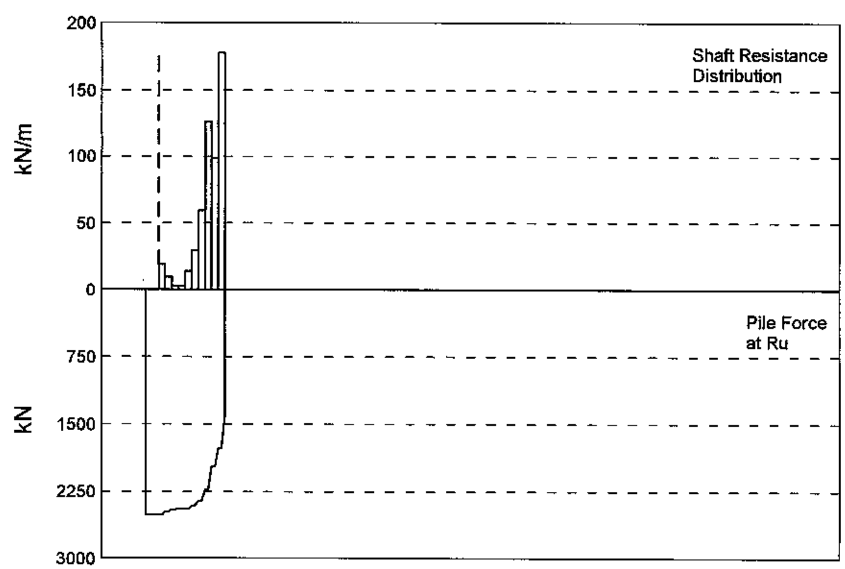
L/c
Velocity Msd
Velocity Cpt


200
150
100
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0
kN/m
Shaft Resistance
Distribution
750
1500
2250
3000
kN
Pile Force
at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2513.5; along Shaft 1113.3; at Toe 1400.2 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2513.5					
1	6.2	1.5	38.5	2475.0	38.5	25.88	21.22	0.827	2.833
2	8.3	3.6	20.3	2454.7	58.8	9.84	8.07	0.827	2.832
3	10.3	5.6	6.1	2448.6	64.9	2.96	2.42	0.827	2.832
4	12.4	7.7	6.1	2442.5	71.0	2.96	2.42	0.827	2.832
5	14.4	9.7	28.3	2414.2	99.3	13.72	11.25	0.827	2.832
6	16.5	11.8	60.8	2353.4	160.1	29.48	24.16	0.827	2.832
7	18.6	13.9	122.6	2230.8	282.7	59.44	48.72	0.827	2.832
8	20.6	15.9	261.4	1969.4	544.1	126.74	103.88	0.827	2.832
9	22.7	18.0	202.9	1766.5	747.0	98.38	80.64	0.827	2.832
10	24.8	20.1	366.3	1400.2	1113.3	177.60	145.57	0.827	2.600
Avg. Shaft			111.3			55.53	45.51	0.827	2.756
Toe			1400.2				15051.87	0.366	3.192

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.622	0.903
Unloading Quake	(% of loading quake)	92	76
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			0.707
Soil Plug Weight	(kN)		0.79

CAPWAP match quality = 2.35 (Wave Up Match) ; RSA = 0
Observed: final set = 1.600 mm; blow count = 625 b/m
Computed: final set = 1.203 mm; blow count = 831 b/m

max. Top Comp. Stress = 185.0 MPa (T= 20.7 ms, max= 1.037 x Top)
max. Comp. Stress = 191.9 MPa (Z= 6.2 m, T= 21.7 ms)
max. Tens. Stress = -59.88 MPa (Z= 16.5 m, T= 52.1 ms)
max. Energy (EMX) = 36.08 kJ; max. Measured Top Displ. (DMX)=21.53 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2601.2	-385.3	185.0	-27.40	36.08	4.6	20.679
2	2.1	2601.4	-430.6	185.0	-30.62	35.12	4.6	19.851
4	4.1	2632.0	-519.4	187.2	-36.94	33.20	4.5	18.195
6	6.2	2698.6	-606.5	191.9	-43.14	31.29	4.4	16.546
7	7.2	2540.3	-579.6	180.7	-41.23	28.51	4.4	15.787
8	8.3	2554.2	-622.3	181.7	-44.26	27.70	4.3	15.028
9	9.3	2468.4	-627.9	175.6	-44.66	25.95	4.3	14.255
10	10.3	2475.1	-669.5	176.0	-47.61	25.08	4.3	13.469
11	11.3	2457.6	-700.6	174.8	-49.83	23.97	4.3	12.683
12	12.4	2483.3	-741.4	176.6	-52.73	23.16	4.2	11.919
13	13.4	2498.9	-771.8	177.7	-54.89	22.13	4.2	11.161
14	14.4	2557.4	-810.3	181.9	-57.63	21.30	4.1	10.388
15	15.5	2516.6	-808.1	179.0	-57.48	19.62	4.0	9.611
16	16.5	2622.4	-841.9	186.5	-59.88	18.73	3.8	8.820
17	17.5	2517.7	-785.8	179.1	-55.89	16.45	3.6	8.040
18	18.6	2678.8	-806.0	190.5	-57.32	15.59	3.3	7.256
19	19.6	2417.2	-668.5	171.9	-47.55	12.72	3.0	6.513
20	20.6	2565.7	-681.1	182.5	-48.44	11.95	2.7	5.771
21	21.7	2043.0	-404.5	145.3	-28.77	8.36	2.5	5.101
22	22.7	2140.2	-416.3	152.2	-29.61	7.73	2.2	4.426
23	23.7	1847.4	-278.6	131.4	-19.81	5.65	2.0	3.809
24	24.8	1870.6	-284.9	133.0	-20.26	3.68	1.9	3.186
Absolute	6.2			191.9			(T =	21.7 ms)
	16.5				-59.88		(T =	52.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3591.4	3412.8	3234.2	3055.6	2877.0	2698.3	2519.7	2341.1	2162.5	1983.9
RX	3591.4	3412.8	3234.2	3055.6	2877.0	2698.3	2533.8	2457.7	2424.3	2423.5
RU	3591.4	3412.8	3234.2	3055.6	2877.0	2698.3	2519.7	2341.1	2162.5	1983.9

RAU = 2256.2 (kN); RA2 = 2312.4 (kN)

Current CAPWAP Ru = 2513.5 (kN); Corresponding J(RP)= 0.60; J(RX) = 0.63

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
4.65	20.53	2636.9	2740.6	2740.6	21.532	1.559	1.600	37.7	3259.0

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	140.60	206842.7	77.287	1.220
24.75	140.60	206842.7	77.287	1.220

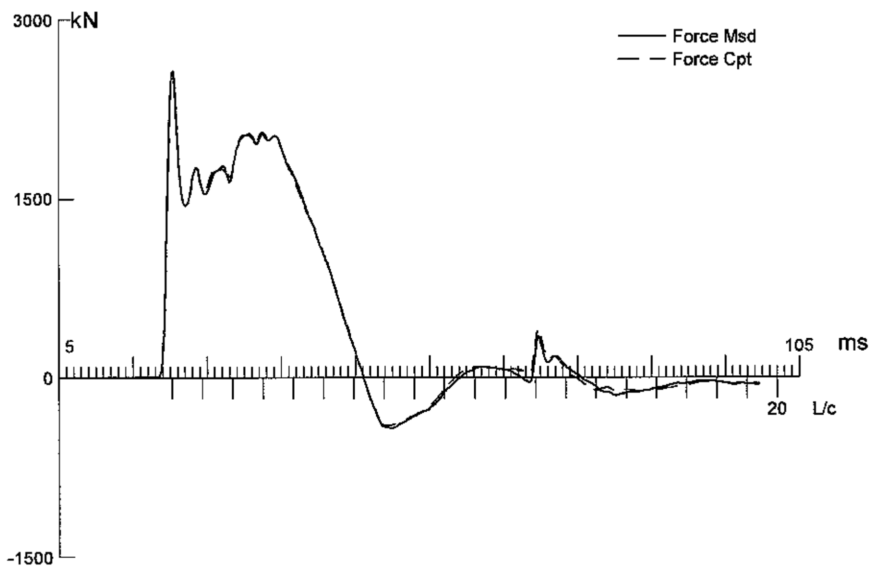
Toe Area 0.093 m^2

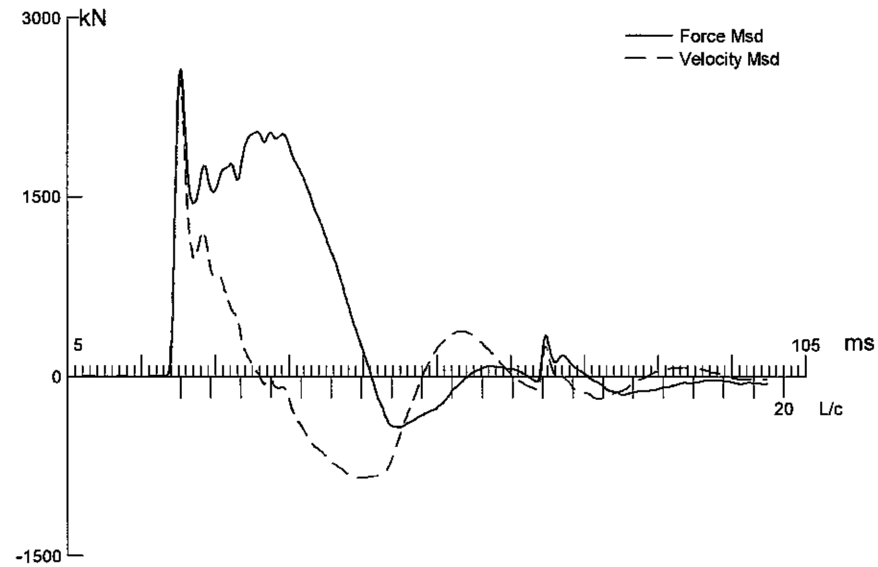
Top Segment Length 1.03 m, Top Impedance 567.67 kN/m/s

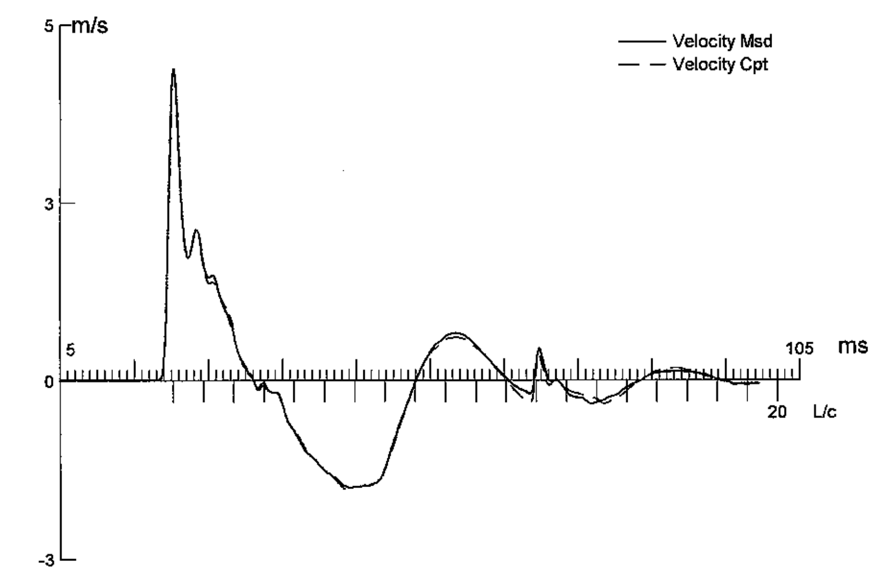
Pile Damping 1.0 %, Time Incr 0.201 ms, Wave Speed 5123.0 m/s, 2L/c 9.7 ms

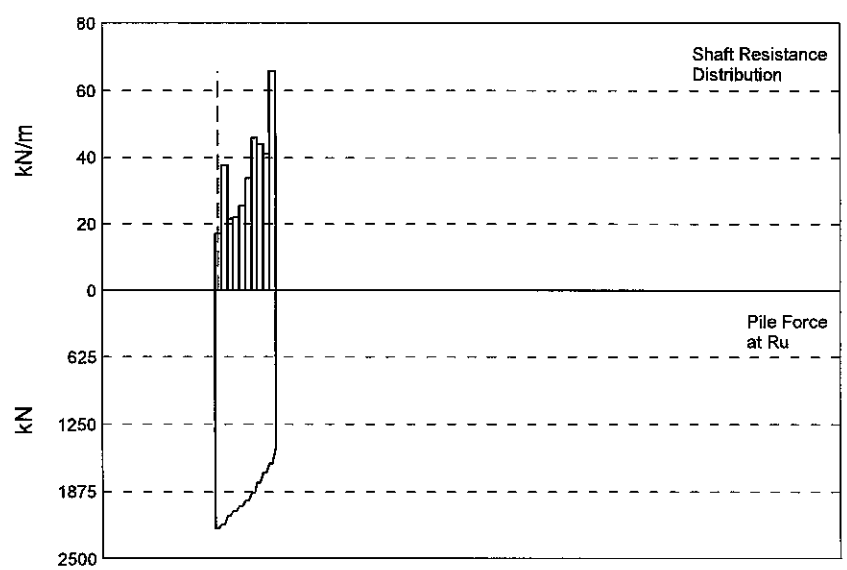
[redacted] Pile: PILE 112; RE-STRIKE; Blow: 9 (Test: 11-Feb-2011 14:35:)

12-Feb-2011
CAPWAP(R) 2006-2









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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2216.6; along Shaft 740.7; at Toe 1475.9 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2216.6				
1	2.1	0.8	35.7	2180.9	35.7	42.25	34.18	0.690
2	4.2	2.9	79.1	2101.8	114.8	37.76	30.55	0.690
3	6.3	5.0	45.3	2056.5	160.1	21.62	17.49	0.690
4	8.4	7.1	45.7	2010.8	205.8	21.81	17.65	0.690
5	10.5	9.2	53.1	1957.7	258.9	25.35	20.51	0.690
6	12.6	11.3	70.7	1887.0	329.6	33.75	27.30	0.690
7	14.7	13.4	95.8	1791.2	425.4	45.73	37.00	0.690
8	16.8	15.5	91.7	1699.5	517.1	43.77	35.41	0.690
9	18.9	17.6	86.1	1613.4	603.2	41.10	33.25	0.690
10	20.9	19.7	137.5	1475.9	740.7	65.63	53.10	0.690
Avg. Shaft			74.1			37.60	30.42	0.690
Toe			1475.9				15457.69	0.364

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	4.771	4.819
Case Damping Factor		0.900	0.946
Unloading Quake	(% of loading quake)	95	78
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake)	(mm)		0.919

CAPWAP match quality = 1.38 (Wave Up Match) ; RSA = 0
Observed: final set = 1.800 mm; blow count = 556 b/m
Computed: final set = 1.637 mm; blow count = 611 b/m

max. Top Comp. Stress = 184.6 MPa (T= 20.9 ms, max= 1.021 x Top)
max. Comp. Stress = 188.4 MPa (Z= 2.1 m, T= 21.1 ms)
max. Tens. Stress = -33.77 MPa (Z= 4.2 m, T= 50.1 ms)
max. Energy (EMX) = 32.45 kJ; max. Measured Top Displ. (DMX)=18.61 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2594.8	-436.3	184.6	-31.03	32.45	4.3	17.647
2	2.1	2648.7	-470.1	188.4	-33.44	31.69	4.3	16.904
3	3.1	2581.8	-442.2	183.6	-31.45	29.53	4.2	16.166
4	4.2	2627.6	-474.8	186.9	-33.77	28.76	4.1	15.422
5	5.2	2400.0	-388.5	170.7	-27.63	25.29	4.0	14.700
6	6.3	2436.1	-420.1	173.3	-29.88	24.56	3.9	13.972
7	7.3	2325.8	-388.7	165.4	-27.65	22.44	3.9	13.258
8	8.4	2364.6	-418.5	168.2	-29.77	21.73	3.8	12.540
9	9.4	2263.7	-388.8	161.0	-27.65	19.81	3.7	11.837
10	10.5	2310.8	-417.3	164.4	-29.68	19.16	3.6	11.150
11	11.5	2204.8	-379.1	156.8	-26.97	17.29	3.6	10.477
12	12.6	2263.4	-405.0	161.0	-28.80	16.65	3.5	9.796
13	13.6	2125.2	-343.9	151.2	-24.46	14.66	3.4	9.138
14	14.7	2183.5	-367.3	155.3	-26.12	14.06	3.2	8.477
15	15.7	1985.6	-293.6	141.2	-20.88	11.98	3.1	7.847
16	16.8	2037.7	-300.7	144.9	-21.39	11.43	3.0	7.213
17	17.8	1860.2	-247.5	132.3	-17.61	9.74	3.0	6.605
18	18.9	1910.4	-252.0	135.9	-17.92	9.22	2.9	5.994
19	19.9	1751.6	-223.5	124.6	-15.90	7.87	2.8	5.410
20	20.9	1931.5	-226.3	137.4	-16.10	6.86	2.6	4.823
Absolute	2.1			188.4			(T =	21.1 ms)
	4.2				-33.77		(T =	50.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3332.7	3143.7	2954.6	2765.5	2576.4	2387.3	2198.2	2009.2	1820.1	1631.0
RX	3332.7	3143.7	2954.6	2765.5	2577.9	2410.2	2242.4	2182.8	2162.2	2141.7
RU	3332.7	3143.7	2954.6	2765.5	2576.4	2387.3	2198.2	2009.2	1820.1	1631.0

RAU = 2107.4 (kN); RA2 = 2219.4 (kN)

Current CAPWAP Ru = 2216.6 (kN); Corresponding J(RP)= 0.59; J(RX) = 0.64

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.52	20.45	2568.4	2655.1	2655.1	18.614	1.898	1.800	33.7	3305.7

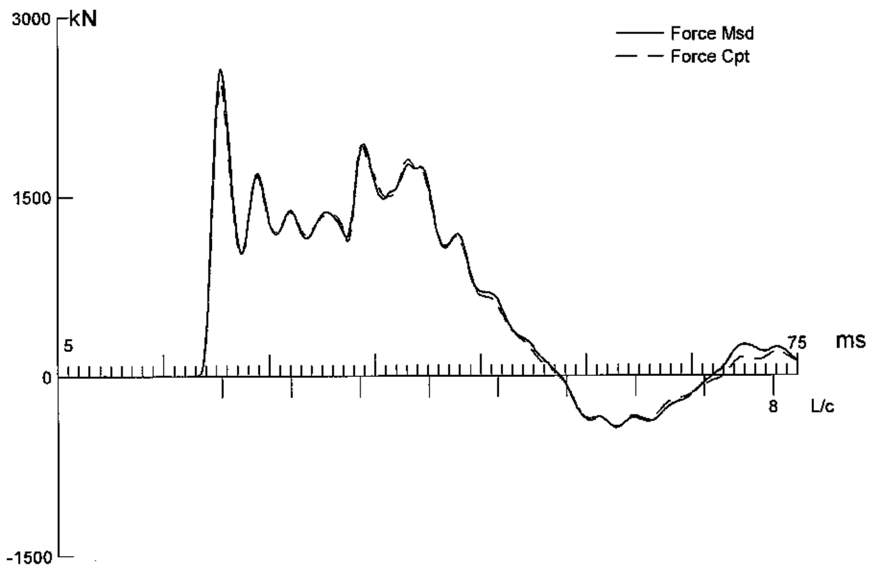
PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	140.60	206842.7	77.287	1.236
20.95	140.60	206842.7	77.287	1.236

Toe Area 0.095 m^2

Top Segment Length 1.05 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.204 ms, Wave Speed 5123.0 m/s, 2L/c 8.2 ms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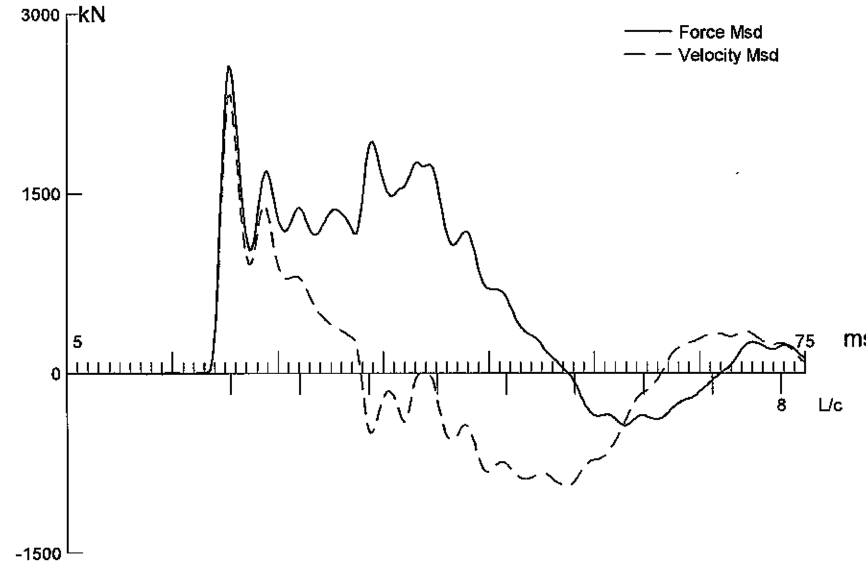
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ms
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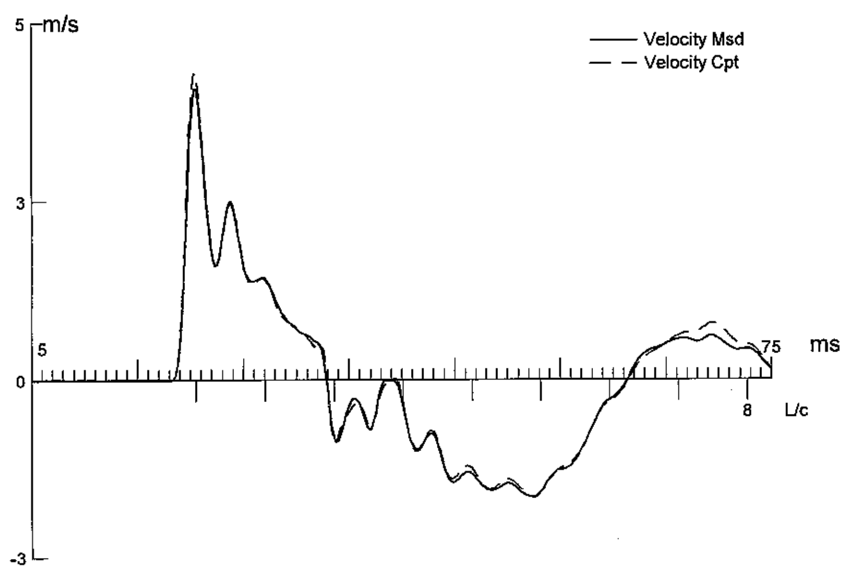
L/c
Force Msd
Velocity Msd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m/s
3
0
-3
5
75
ms
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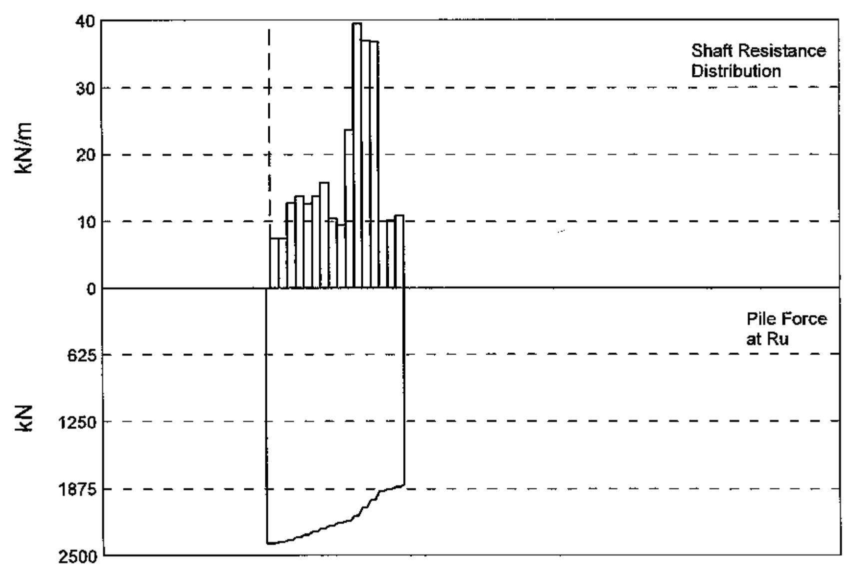
L/c
Velocity Msd
Velocity Cpt


40
30
20
10
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

[redacted] Pile: 122 Test: 11-Feb-2011 13:11:

RE-STRIKE; Blow: 18 CAPWAP(R) 2006-2

[redacted] OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2384.1; along Shaft 549.1; at Toe 1835.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2384.1					
1	3.0	1.8	15.1	2369.0	15.1	8.36	6.85	1.150	6.375
2	5.1	3.8	15.1	2353.9	30.2	7.46	6.11	1.150	6.381
3	7.1	5.9	25.7	2328.2	55.9	12.70	10.41	1.150	6.381
4	9.1	7.9	27.8	2300.4	83.7	13.73	11.26	1.150	6.381
5	11.1	9.9	25.6	2274.8	109.3	12.65	10.37	1.150	6.381
6	13.2	11.9	27.8	2247.0	137.1	13.73	11.26	1.150	6.381
7	15.2	14.0	31.8	2215.2	168.9	15.71	12.88	1.150	6.381
8	17.2	16.0	21.1	2194.1	190.0	10.42	8.54	1.150	6.381
9	19.2	18.0	19.2	2174.9	209.2	9.49	7.77	1.150	6.381
10	21.3	20.0	47.9	2127.0	257.1	23.66	19.40	1.150	6.381
11	23.3	22.0	80.0	2047.0	337.1	39.52	32.39	1.150	6.381
12	25.3	24.1	74.8	1972.2	411.9	36.95	30.29	1.150	6.381
13	27.3	26.1	74.5	1897.7	486.4	36.80	30.17	1.150	6.381
14	29.4	28.1	20.3	1877.4	506.7	10.03	8.22	1.150	5.835
15	31.4	30.1	20.5	1856.9	527.2	10.13	8.30	1.150	4.632
16	33.4	32.2	21.9	1835.0	549.1	10.82	8.87	1.150	3.059
Avg. Shaft			34.3			17.07	13.99	1.150	6.162
Toe			1835.0				19725.88	0.687	2.561

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.112	2.221
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	61	
Resistance Gap (included in Toe Quake) (mm)			0.076
Soil Plug Weight	(kN)		0.79

CAPWAP match quality = 2.79 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 173.6 MPa (T= 20.7 ms, max= 1.203 x Top)
max. Comp. Stress = 208.8 MPa (Z= 33.4 m, T= 27.3 ms)
max. Tens. Stress = -39.17 MPa (Z= 11.1 m, T= 59.7 ms)
max. Energy (EMX) = 34.23 kJ; max. Measured Top Displ. (DMX)=21.79 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2440.4	-463.6	173.6	-32.97	34.23	4.3	21.823
2	2.0	2444.9	-490.9	173.9	-34.92	33.86	4.3	21.326
4	4.0	2423.9	-505.7	172.4	-35.96	32.21	4.2	20.247
6	6.1	2407.8	-511.7	171.3	-36.40	30.47	4.2	19.090
8	8.1	2376.2	-515.8	169.0	-36.69	28.27	4.1	17.914
10	10.1	2340.9	-534.2	166.5	-37.99	26.15	4.1	16.789
12	12.1	2311.0	-532.9	164.4	-37.90	24.29	4.0	15.708
14	14.2	2280.4	-523.5	162.2	-37.23	22.32	4.0	14.558
16	16.2	2239.4	-506.6	159.3	-36.03	20.14	3.9	13.287
18	18.2	2214.5	-507.1	157.5	-36.07	18.52	3.9	12.082
20	20.2	2204.2	-507.2	156.8	-36.08	17.15	3.8	10.969
22	22.3	2167.8	-478.0	154.2	-33.99	15.02	3.7	9.713
24	24.3	2217.6	-433.5	157.7	-30.83	12.17	3.6	8.278
25	25.3	2239.3	-440.5	159.3	-31.33	11.47	3.5	7.571
26	26.3	2102.7	-406.5	149.5	-28.91	10.16	3.5	7.063
27	27.3	2055.1	-411.7	146.2	-29.28	9.95	3.4	6.685
28	28.3	1931.7	-389.3	137.4	-27.69	8.97	3.4	6.317
29	29.4	1922.5	-393.8	136.7	-28.01	8.69	3.4	5.882
30	30.4	1829.9	-392.3	130.2	-27.90	8.06	3.6	5.319
31	31.4	2184.0	-395.4	155.3	-28.12	7.37	3.6	4.632
32	32.4	2600.2	-394.5	184.9	-28.06	6.27	3.2	3.842
33	33.4	2935.7	-395.4	208.8	-28.12	6.51	2.3	3.108
Absolute	33.4			208.8			(T =	27.3 ms)
	11.1				-39.17		(T =	59.7 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3595.6	3468.8	3342.0	3215.3	3088.5	2961.7	2834.9	2708.1	2581.3	2454.5
RX	3595.6	3468.8	3342.0	3215.3	3088.5	2977.3	2867.9	2758.4	2649.0	2539.6
RU	3687.5	3569.9	3452.3	3334.7	3217.1	3099.5	2981.9	2864.2	2746.6	2629.0

RAU = 1510.3 (kN); RA2 = 2174.9 (kN)

Current CAPWAP Ru = 2384.1 (kN);

Case Method matching requires higher damping factor - please check with PDA-W

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
4.09	20.74	2321.2	2542.3	2563.3	21.786	0.385	1.000	34.5	3028.7

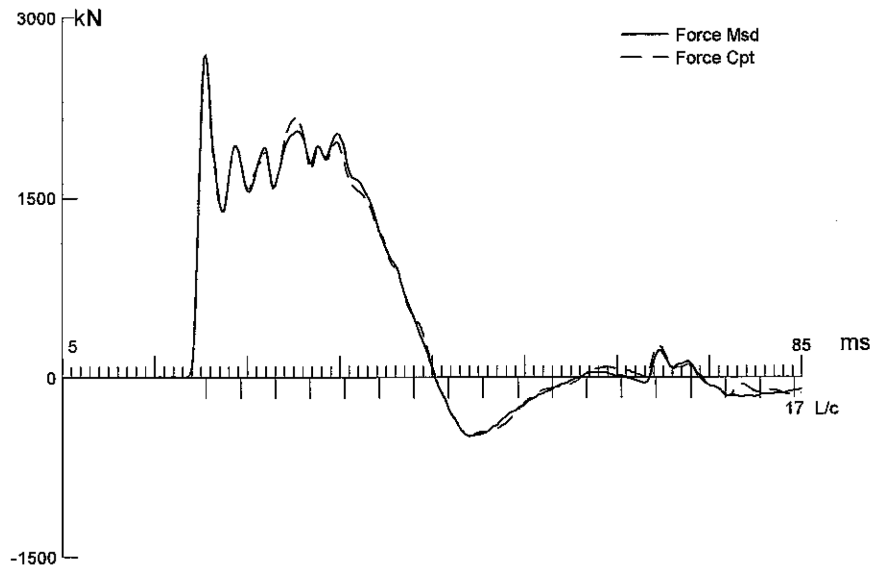
PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	140.60	206842.7	77.287	1.220
33.40	140.60	206842.7	77.287	1.220

Toe Area 0.093 m^2

Top Segment Length 1.01 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 13.0 ms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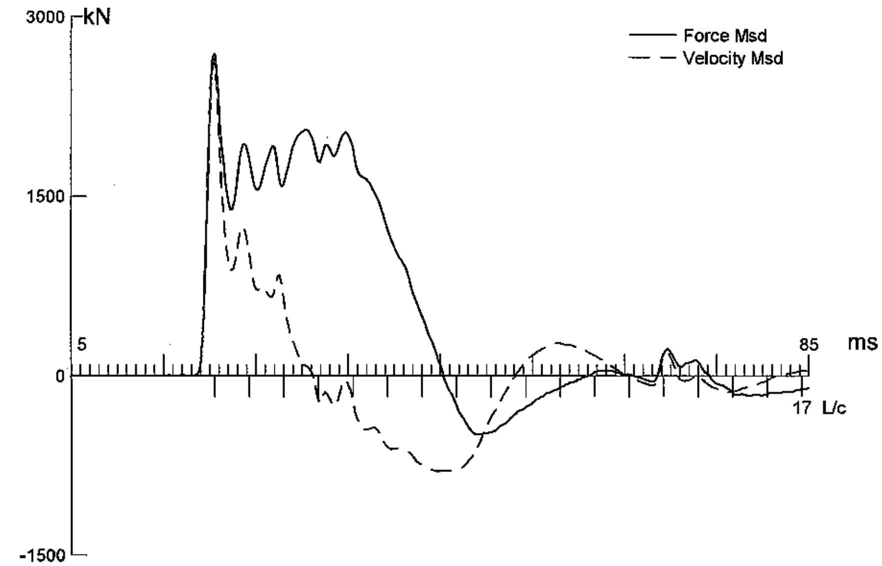
1500
0
-1500
5
85
ms
17 L/c
Force Msd
Force Cpt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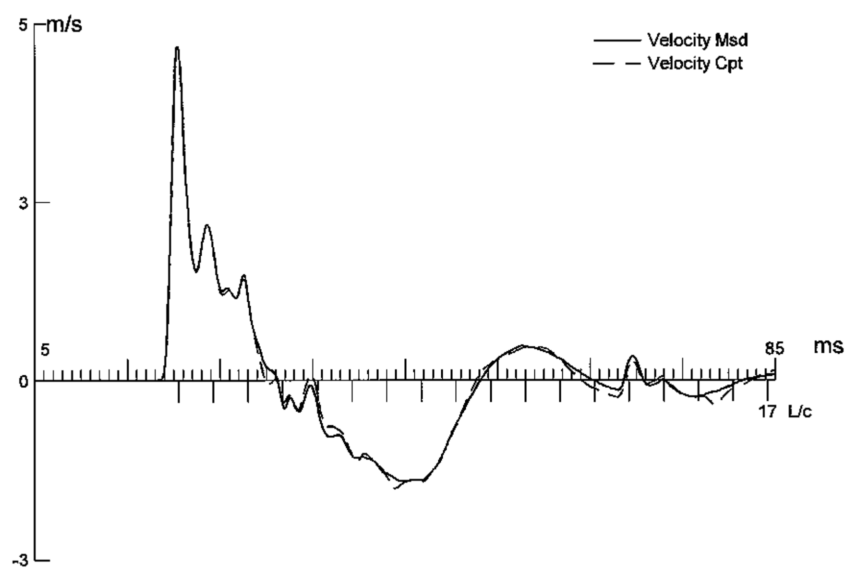
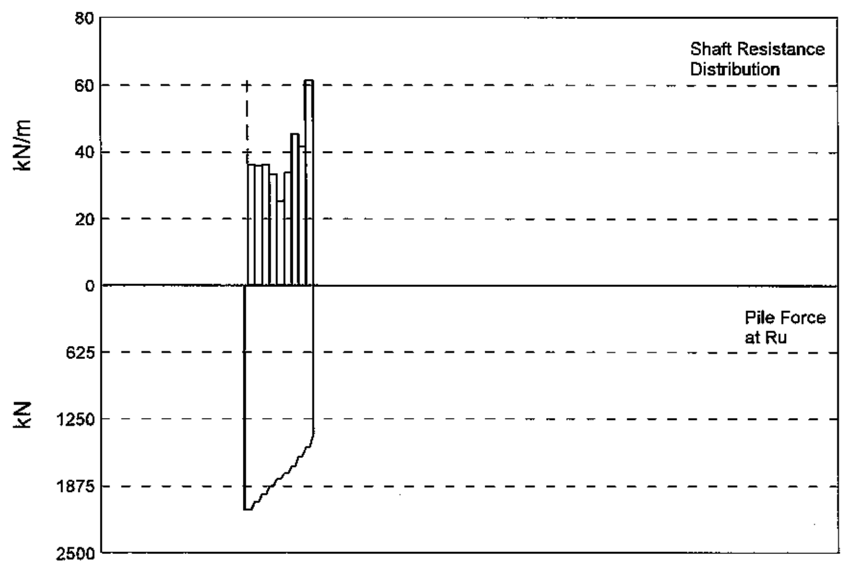
5
85
ms
17 L/c
Force Msd
Velocity Msd


5 m/s
3
0
-3
5
85
ms
17 L/c
Velocity Msd
Velocity Cpt


80
60
40
20
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2094.7; along Shaft 704.3; at Toe 1390.4 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2094.7				
1	3.0	1.7	73.0	2021.7	73.0	43.41	35.58	0.642
2	5.1	3.7	72.0	1949.7	145.0	35.63	29.20	0.642
3	7.1	5.7	73.0	1876.7	218.0	36.12	29.61	0.642
4	9.1	7.7	67.2	1809.5	285.2	33.25	27.25	0.642
5	11.1	9.8	51.4	1758.1	336.6	25.43	20.85	0.642
6	13.1	11.8	68.2	1689.9	404.8	33.74	27.66	0.642
7	15.2	13.8	91.7	1598.2	496.5	45.37	37.19	0.642
8	17.2	15.8	84.1	1514.1	580.6	41.61	34.11	0.642
9	19.2	17.9	123.7	1390.4	704.3	61.21	50.17	0.642
Avg. Shaft			78.3			39.46	32.34	0.642
Toe			1390.4				14946.52	0.103

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	4.576	2.494
Case Damping Factor		0.797	0.252
Unloading Quake	(% of loading quake)	74	124
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	95	
Resistance Gap (included in Toe Quake) (mm)			0.486
Soil Plug Weight	(kN)		1.26

CAPWAP match quality = 2.20 (Wave Up Match) ; RSA = 0
Observed: final set = 2.200 mm; blow count = 455 b/m
Computed: final set = 2.955 mm; blow count = 338 b/m

max. Top Comp. Stress = 194.3 MPa (T= 20.9 ms, max= 1.044 x Top)
max. Comp. Stress = 202.9 MPa (Z= 3.0 m, T= 21.3 ms)
max. Tens. Stress = -40.53 MPa (Z= 3.0 m, T= 49.7 ms)
max. Energy (EMX) = 34.70 kJ; max. Measured Top Displ. (DMX)=19.01 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2732.1	-525.7	194.3	-37.39	34.70	4.6	18.045
2	2.0	2792.9	-548.3	198.6	-39.00	34.05	4.5	17.369
3	3.0	2852.8	-569.8	202.9	-40.53	33.36	4.4	16.683
4	4.0	2660.8	-496.3	189.2	-35.30	29.93	4.3	15.998
5	5.1	2717.7	-530.5	193.3	-37.73	29.22	4.2	15.296
6	6.1	2537.8	-460.5	180.5	-32.75	26.12	4.1	14.607
7	7.1	2589.5	-492.3	184.2	-35.02	25.43	4.0	13.917
8	8.1	2408.9	-430.5	171.3	-30.62	22.67	3.9	13.273
9	9.1	2450.8	-461.6	174.3	-32.83	22.11	3.8	12.644
10	10.1	2289.0	-405.0	162.8	-28.81	19.83	3.8	12.053
11	11.1	2332.5	-422.4	165.9	-30.04	19.33	3.7	11.461
12	12.1	2233.8	-365.7	158.9	-26.01	17.62	3.6	10.877
13	13.1	2288.4	-365.2	162.8	-25.98	17.11	3.5	10.278
14	14.1	2158.9	-280.0	153.5	-19.91	15.26	3.4	9.696
15	15.2	2217.3	-279.5	157.7	-19.88	14.78	3.3	9.117
16	16.2	2048.7	-187.2	145.7	-13.32	12.82	3.2	8.570
17	17.2	2044.2	-182.3	145.4	-12.96	12.39	3.2	8.021
18	18.2	1787.5	-125.2	127.1	-8.91	10.75	3.6	7.497
19	19.2	1745.0	-127.5	124.1	-9.07	9.46	3.5	6.972
Absolute	3.0			202.9			(T =	21.3 ms)
	3.0				-40.53		(T =	49.7 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3137.8	2907.1	2676.5	2445.8	2215.2	1984.6	1753.9	1523.3	1292.6	1062.0
RX	3137.8	2907.1	2676.5	2446.6	2336.0	2273.7	2211.4	2183.0	2161.0	2139.1
RU	3137.8	2907.1	2676.5	2445.8	2215.2	1984.6	1753.9	1523.3	1292.6	1062.0

RAU = 1830.7 (kN); RA2 = 2143.9 (kN)

Current CAPWAP Ru = 2094.7 (kN); Corresponding J(RP)= 0.45; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.77	20.71	2710.0	2734.2	2734.2	19.014	2.391	2.200	36.3	3424.4

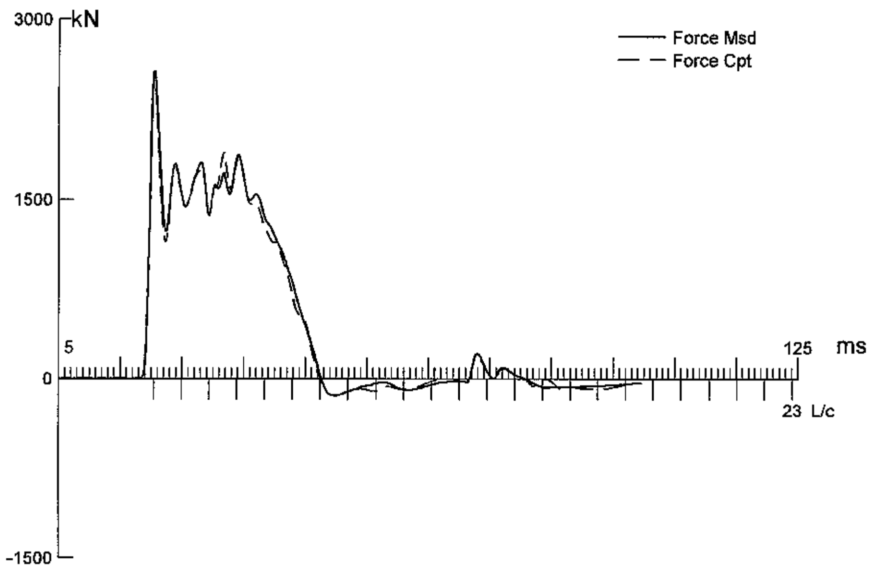
PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	140.60	206842.7	77.287	1.220
19.20	140.60	206842.7	77.287	1.220

Toe Area 0.093 m^2

Top Segment Length 1.01 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.197 ms, Wave Speed 5123.0 m/s, 2L/c 7.5 ms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kN
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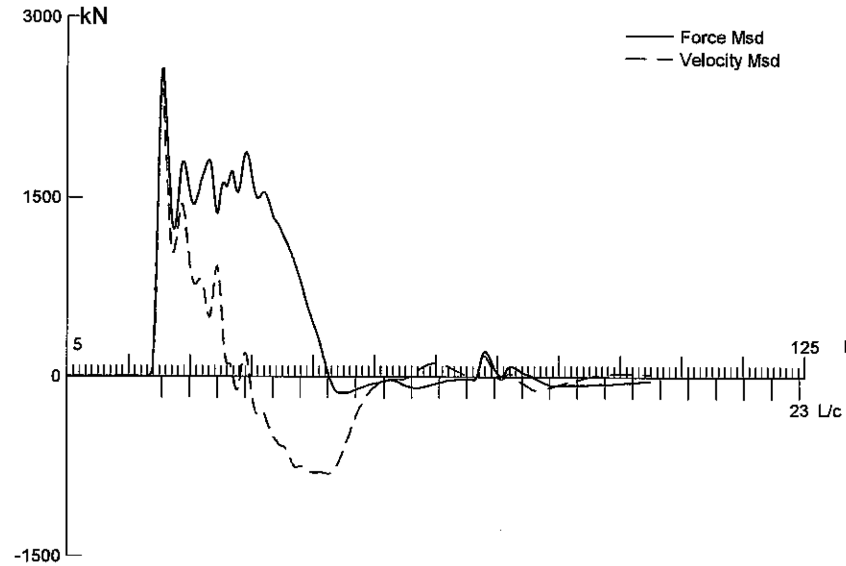
5
125
ms
23 L/c
Force Msd
Force Cpt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ms
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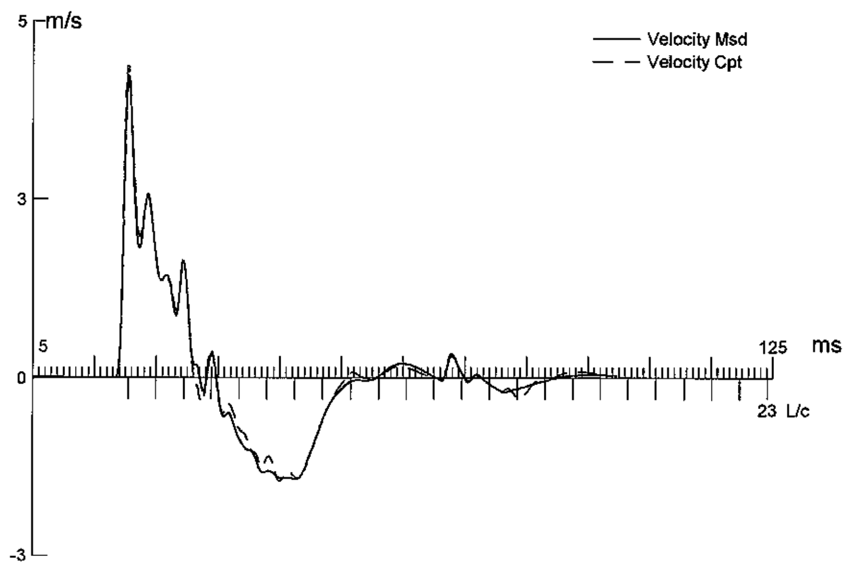
Force Msd
Velocity Msd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m/s
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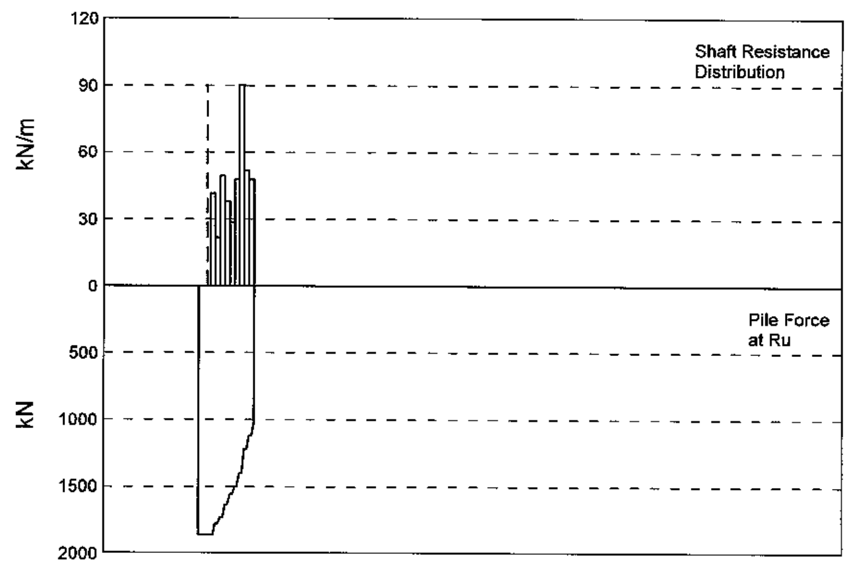
ms
23 L/c
Velocity Msd
Velocity Cpt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90
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30
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500
1000
1500
2000
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

[redacted]Pile: PILE 39 Test: 23-Mar-2011 10:12:

RE-STRIKE; Blow: 27 CAPWAP(R) 2006-2

[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1860.0; along Shaft 837.4; at Toe 1022.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				1860.0				
1	7.0	2.3	83.1	1776.9	83.1	36.44	29.87	0.521
2	9.0	4.3	43.8	1733.1	126.9	21.81	17.87	0.521
3	11.0	6.3	99.5	1633.6	226.4	49.53	40.60	0.521
4	13.1	8.3	76.3	1557.3	302.7	37.98	31.14	0.521
5	15.1	10.3	57.7	1499.6	360.4	28.73	23.55	0.521
6	17.1	12.3	96.0	1403.6	456.4	47.79	39.17	0.521
7	19.1	14.3	181.2	1222.4	637.6	90.21	73.94	0.521
8	21.1	16.3	103.8	1118.6	741.4	51.68	42.36	0.521
9	23.1	18.3	96.0	1022.6	837.4	47.79	39.17	0.521
Avg. Shaft			93.0			45.63	37.41	0.521
Toe			1022.6				10992.74	0.098

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.766	3.428
Case Damping Factor		0.769	0.177
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	100	102
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	24	
Soil Plug Weight	(kN)		1.49

CAPWAP match quality = 2.64 (Wave Up Match) ; RSA = 0

Observed: final set = 6.000 mm; blow count = 167 b/m

Computed: final set = 5.009 mm; blow count = 200 b/m

max. Top Comp. Stress = 177.1 MPa (T= 20.8 ms, max= 1.053 x Top)

max. Comp. Stress = 186.5 MPa (Z= 7.0 m, T= 22.2 ms)

max. Tens. Stress = -21.41 MPa (Z= 7.0 m, T= 50.8 ms)

max. Energy (EMX) = 37.33 kJ; max. Measured Top Displ. (DMX)=22.16 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2490.6	-167.4	177.1	-11.90	37.33	4.4	21.839
2	2.0	2488.4	-193.0	177.0	-13.73	36.75	4.4	21.206
4	4.0	2485.9	-237.9	176.8	-16.92	35.63	4.4	19.942
5	5.0	2502.6	-258.1	178.0	-18.36	35.13	4.4	19.353
6	6.0	2555.3	-277.7	181.7	-19.75	34.63	4.3	18.766
7	7.0	2621.7	-301.0	186.5	-21.41	34.13	4.2	18.178
8	8.0	2393.2	-266.2	170.2	-18.93	30.86	4.1	17.615
9	9.0	2442.8	-289.7	173.7	-20.60	30.40	4.1	17.051
10	10.0	2367.8	-278.2	168.4	-19.79	28.61	4.0	16.505
11	11.0	2447.2	-296.7	174.1	-21.10	28.18	3.8	15.967
12	12.1	2225.3	-258.1	158.3	-18.36	25.06	3.8	15.464
13	13.1	2281.1	-279.1	162.2	-19.85	24.68	3.7	14.945
14	14.1	2129.3	-258.8	151.4	-18.41	22.34	3.7	14.433
15	15.1	2173.2	-278.1	154.6	-19.78	21.92	3.6	13.898
16	16.1	2080.6	-266.2	148.0	-18.93	20.19	3.5	13.386
17	17.1	2151.0	-284.0	153.0	-20.20	19.82	3.4	12.881
18	18.1	2012.2	-253.7	143.1	-18.05	17.54	3.3	12.420
19	19.1	2115.4	-270.1	150.5	-19.21	17.27	3.2	11.997
20	20.1	1811.2	-189.8	128.8	-13.50	13.88	3.1	11.617
21	21.1	1797.5	-191.6	127.8	-13.63	13.65	3.3	11.217
22	22.1	1484.4	-133.8	105.6	-9.52	11.67	3.8	10.847
23	23.1	1394.8	-122.0	99.2	-8.68	10.27	4.0	10.491
Absolute	7.0			186.5			(T =	22.2 ms)
	7.0				-21.41		(T =	50.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	2754.9	2524.3	2293.8	2063.2	1832.6	1602.1	1371.5	1141.0	910.4	679.8
RX	2754.9	2524.3	2303.8	2215.1	2134.8	2054.5	1974.3	1929.1	1909.2	1889.4
RU	2754.9	2524.3	2293.8	2063.2	1832.6	1602.1	1371.5	1141.0	910.4	679.8

RAU = 1792.3 (kN); RA2 = 2165.2 (kN)

Current CAPWAP Ru = 1860.0 (kN); Corresponding J(RP)= 0.39; matches RX9 within 5%

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
4.31	20.58	2444.3	2616.2	2616.2	22.164	5.635	6.000	38.5	2734.4

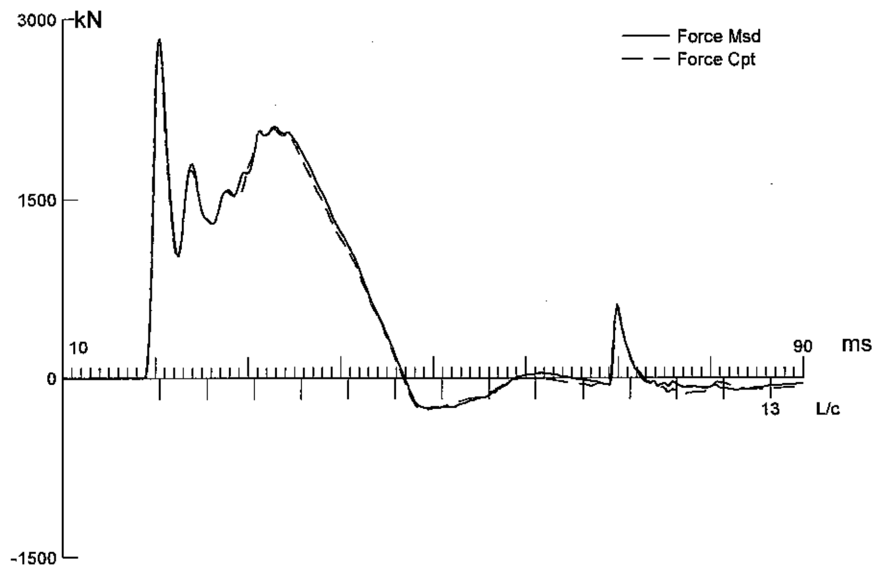
PILE PROFILE AND PILE MODEL

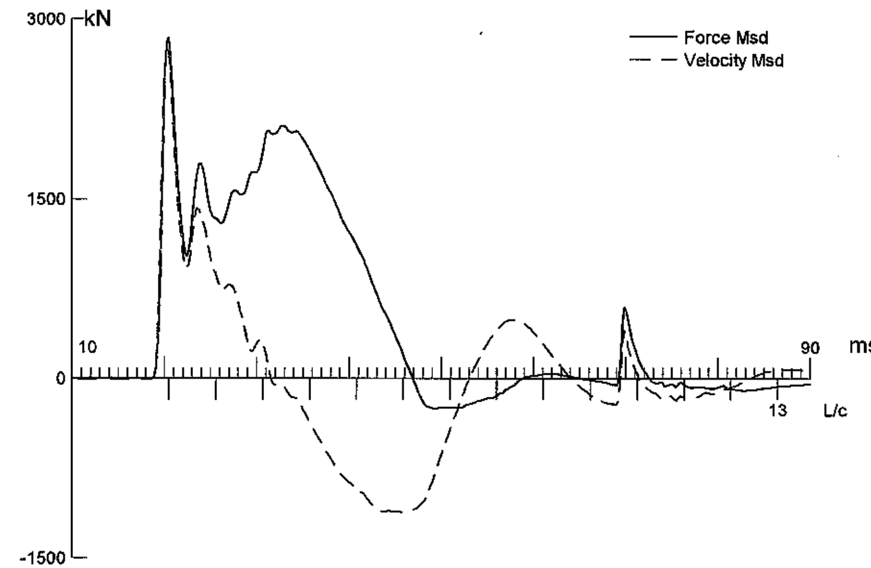
Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	140.60	206842.7	77.287	1.220
23.10	140.60	206842.7	77.287	1.220

Toe Area 0.093 m^2

Top Segment Length 1.00 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 5123.0 m/s, 2L/c 9.0 ms


3000
kN
1500
0
-1500
10
90
ms
13
L/c
Force Msd
Force Cpt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kN
1500
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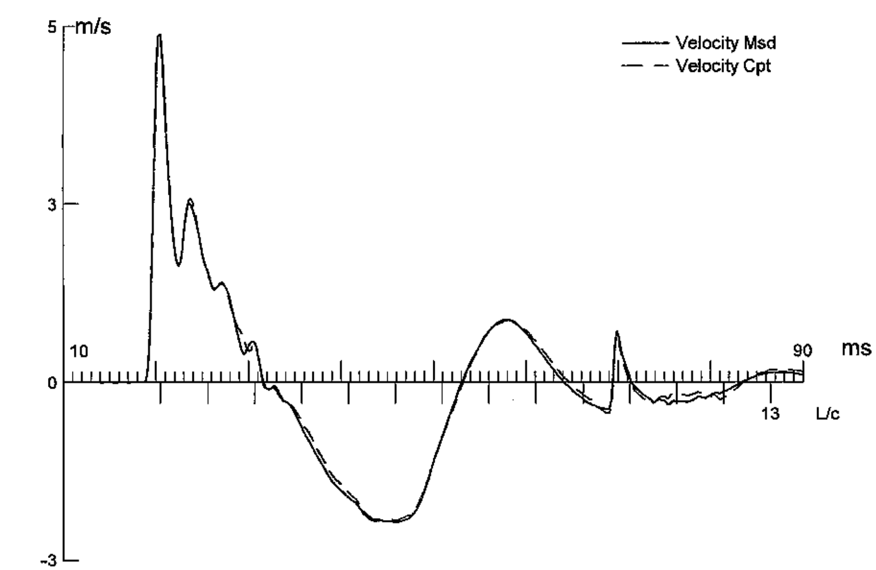
10
90
ms
13
L/c
Force Msd
Velocity Msd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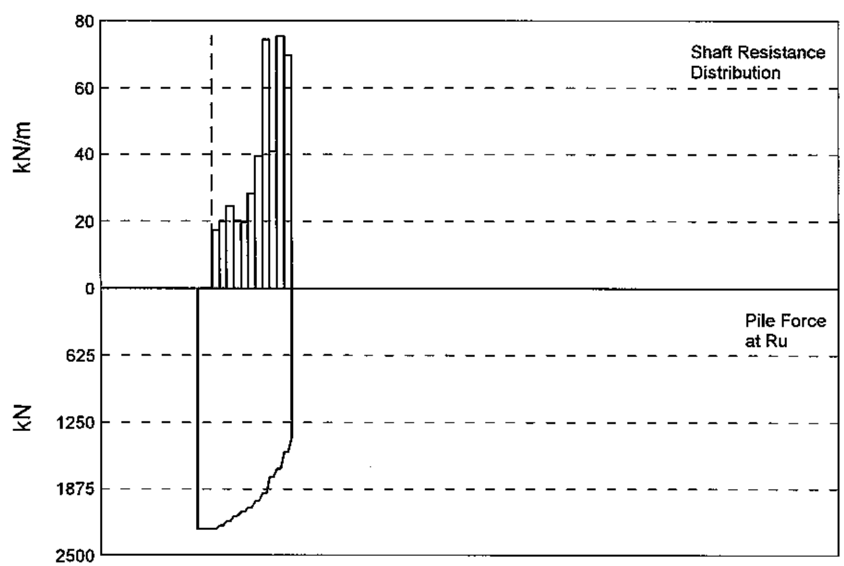
m/s
3
0
-3
10
90
ms
13
L/c
Velocity Msd
Velocity Cpt


80
60
40
20
0
kN/m
Shaft Resistance
Distribution
625
1250
1875
2500
kN
Pile Force
at Ru

[redacted]ile: PILE 33 Test: 23-Mar-2011 10:50:
RE-STRIKE; Blow: 18 CAPWAP(R) 2006-2
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2253.7; along Shaft 858.7; at Toe 1395.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2253.7					
1	6.0	1.1	34.5	2219.2	34.5	30.53	25.03	0.497	4.693
2	8.0	3.1	40.2	2179.0	74.7	20.10	16.48	0.497	4.696
3	10.0	5.1	48.9	2130.1	123.6	24.45	20.04	0.497	4.696
4	12.0	7.1	40.2	2089.9	163.8	20.10	16.48	0.497	4.696
5	14.0	9.1	39.4	2050.5	203.2	19.70	16.15	0.497	4.696
6	16.0	11.1	56.6	1993.9	259.8	28.30	23.20	0.497	4.696
7	18.0	13.1	78.7	1915.2	338.5	39.35	32.25	0.497	4.696
8	20.0	15.1	148.7	1766.5	487.2	74.35	60.94	0.497	4.696
9	22.0	17.1	81.6	1684.9	568.8	40.80	33.44	0.497	4.696
10	24.0	19.1	150.8	1534.1	719.6	75.40	61.80	0.497	4.472
11	26.0	21.1	139.1	1395.0	858.7	69.55	57.01	0.497	3.583
Avg. Shaft			78.1			40.64	33.31	0.497	4.477
Toe			1395.0				14995.97	0.479	4.222

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.752	1.177
Unloading Quake	(% of loading quake)	100	92
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	95	
Resistance Gap (included in Toe Quake) (mm)			1.163
Soil Plug Weight	(kN)		0.40

CAPWAP match quality = 1.93 (Wave Up Match) ; RSA = 0
Observed: final set = 2.400 mm; blow count = 417 b/m
Computed: final set = 1.400 mm; blow count = 714 b/m

max. Top Comp. Stress = 199.3 MPa (T= 20.9 ms, max= 1.026 x Top)
max. Comp. Stress = 204.6 MPa (Z= 6.0 m, T= 21.9 ms)
max. Tens. Stress = -40.14 MPa (Z= 16.0 m, T= 52.3 ms)
max. Energy (EMX) = 37.28 kJ; max. Measured Top Displ. (DMX)=21.41 mm

[redacted]Pile: PILE 33 Test: 23-Mar-2011 10:50:
RE-STRIKE; Blow: 18 CAPWAP(R) 2006-2
[redacted]
OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2802.4	-306.5	199.3	-21.80	37.28	4.9	21.030
2	2.0	2802.2	-347.3	199.3	-24.70	36.49	4.9	20.295
4	4.0	2822.7	-425.4	200.8	-30.26	34.91	4.9	18.823
6	6.0	2876.4	-498.5	204.6	-35.45	33.46	4.8	17.411
8	8.0	2835.4	-517.0	201.7	-36.77	30.84	4.7	16.043
10	10.0	2781.3	-530.7	197.8	-37.75	28.07	4.6	14.632
11	11.0	2669.2	-500.6	189.8	-35.61	25.90	4.5	13.936
12	12.0	2696.6	-535.3	191.8	-38.07	25.20	4.5	13.238
13	13.0	2612.5	-516.3	185.8	-36.72	23.44	4.4	12.552
14	14.0	2646.0	-549.7	188.2	-39.10	22.74	4.4	11.858
15	15.0	2577.1	-533.2	183.3	-37.92	21.13	4.3	11.169
16	16.0	2622.6	-564.4	186.5	-40.14	20.42	4.2	10.475
17	17.0	2528.7	-527.0	179.9	-37.48	18.66	4.1	9.825
18	18.0	2600.7	-553.4	185.0	-39.36	18.04	4.0	9.175
19	19.0	2482.2	-483.0	176.5	-34.35	16.14	3.9	8.549
20	20.0	2543.8	-495.7	180.9	-35.26	15.56	3.7	7.918
21	21.0	2251.0	-385.1	160.1	-27.39	13.01	3.7	7.328
22	22.0	2318.0	-395.0	164.9	-28.09	12.50	3.6	6.729
23	23.0	2256.8	-358.9	160.5	-25.53	11.09	3.4	6.150
24	24.0	2412.4	-366.4	171.6	-26.06	10.58	3.1	5.557
25	25.0	2204.9	-322.2	156.8	-22.92	8.88	2.9	5.016
26	26.0	2241.8	-325.7	159.4	-23.16	8.06	2.9	4.483
Absolute	6.0			204.6			(T =	21.9 ms)
	16.0				-40.14		(T =	52.3 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3600.8	3389.8	3178.8	2967.8	2756.8	2545.8	2334.9	2123.9	1912.9	1701.9
RX	3635.4	3432.1	3228.9	3025.7	2825.3	2644.6	2463.9	2367.8	2318.3	2276.6
RU	3600.8	3389.8	3178.8	2967.8	2756.8	2545.8	2334.9	2123.9	1912.9	1701.9

RAU = 2105.7 (kN); RA2 = 2105.7 (kN)

Current CAPWAP Ru = 2253.7 (kN); Corresponding J(RP)= 0.64; matches RX9 within 5%

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
5.00	20.69	2840.1	2870.6	2872.7	21.409	1.800	2.400	38.2	3209.2

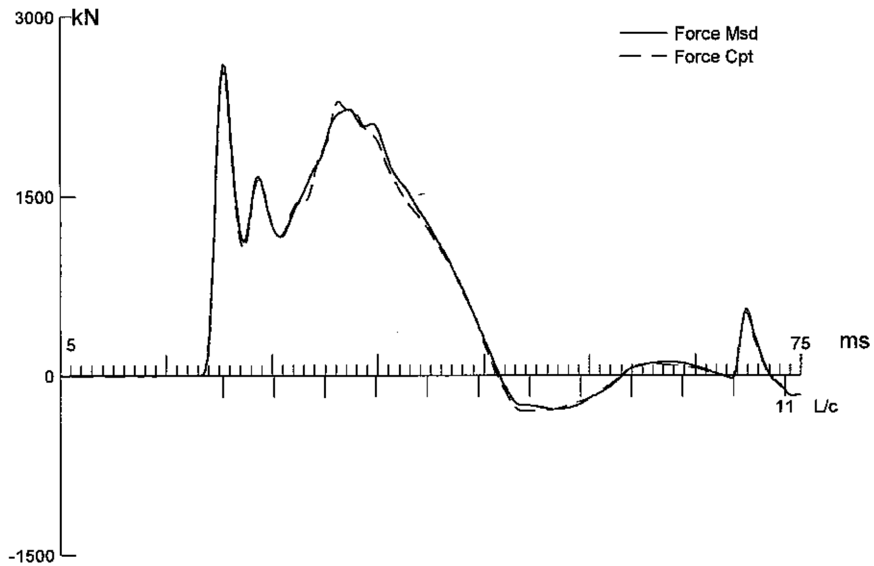
PILE PROFILE AND PILE MODEL

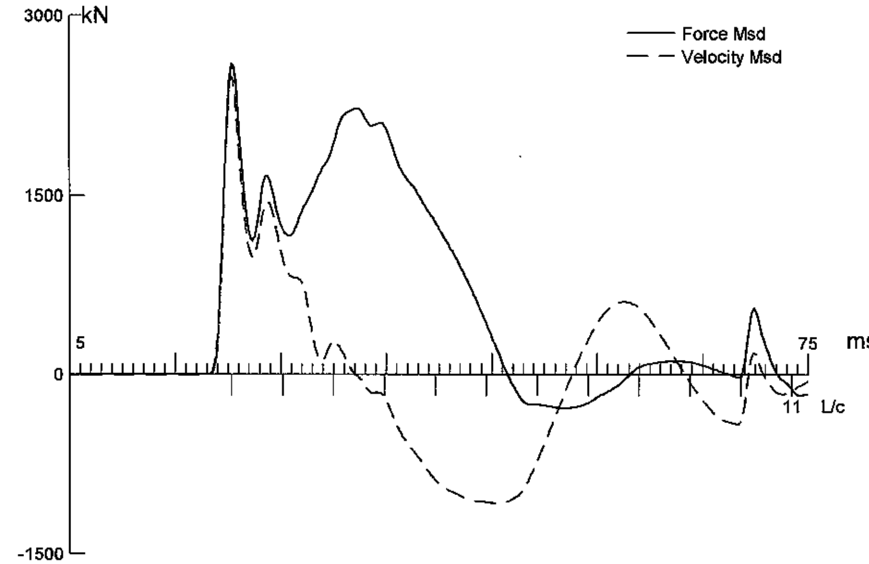
Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	140.60	206842.7	77.287	1.220
26.00	140.60	206842.7	77.287	1.220

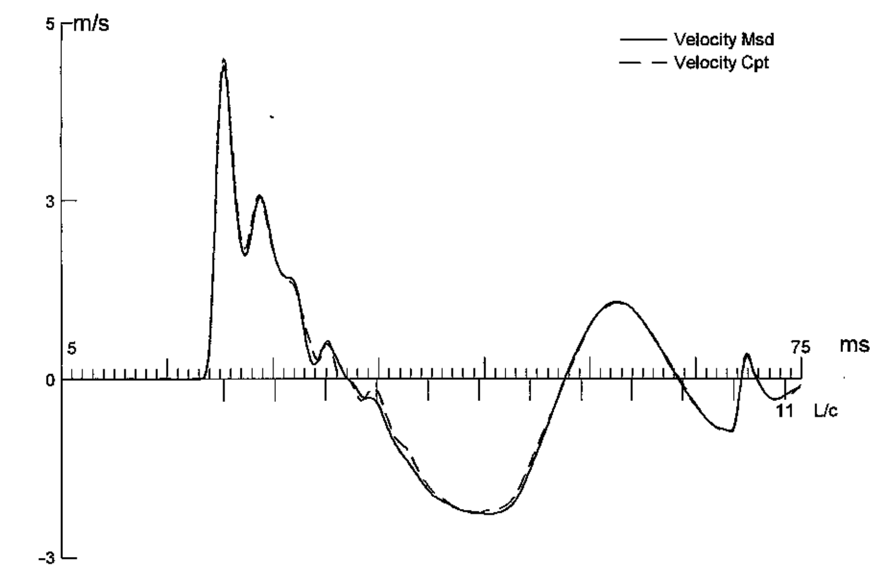
Toe Area 0.093 m^2

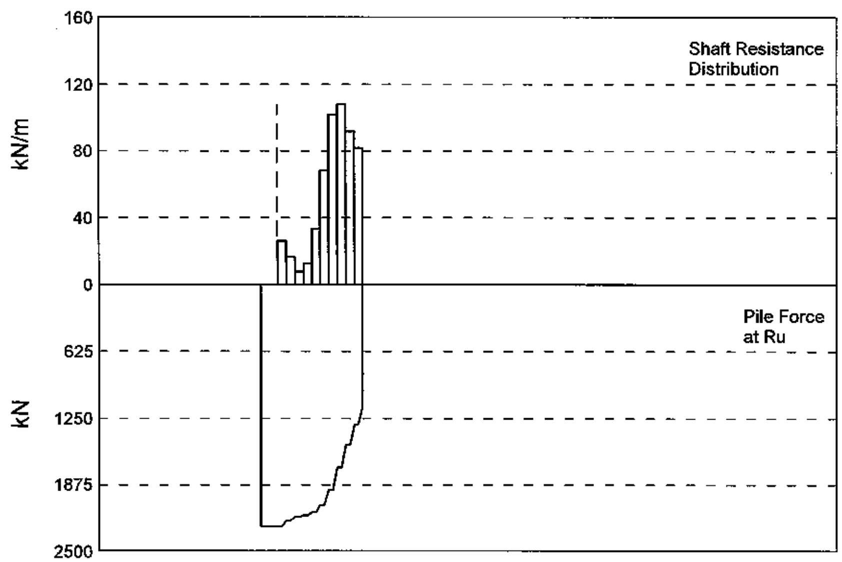
Top Segment Length 1.00 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 10.2 ms


3000 kN
1500
0
-1500
5
75
ms
11 L/c
Force Msd
Force Cpt


3000 kN
1500
0
-1500
5
75
ms
11 L/c
Force Msd
Velocity Msd


5 m/s
3
0
-3
5
75
ms
11 L/c
Velocity Msd
Velocity Cpt


160
120
80
40
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2267.0; along Shaft 1130.1; at Toe 1136.9 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2267.0					
1	6.2	1.0	53.8	2213.2	53.8	51.86	42.50	0.413	5.011
2	8.3	3.1	33.9	2179.3	87.7	16.44	13.47	0.413	5.010
3	10.3	5.2	16.0	2163.3	103.7	7.76	6.36	0.413	5.010
4	12.4	7.2	26.1	2137.2	129.8	12.65	10.37	0.413	5.010
5	14.4	9.3	69.2	2068.0	199.0	33.55	27.50	0.413	5.010
6	16.5	11.4	140.1	1927.9	339.1	67.93	55.68	0.413	5.010
7	18.6	13.4	210.2	1717.7	549.3	101.92	83.54	0.413	5.010
8	20.6	15.5	223.1	1494.6	772.4	108.17	88.66	0.413	5.010
9	22.7	17.5	189.1	1305.5	961.5	91.68	75.15	0.413	5.010
10	24.8	19.6	168.6	1136.9	1130.1	81.75	67.00	0.413	4.444
Avg. Shaft			113.0			57.66	47.26	0.413	4.926
Toe			1136.9				12221.45	1.005	3.374

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.822	2.013
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	94	71
Reloading Level	(% of Ru)	100	100
Soil Plug Weight	(kN)		3.36

CAPWAP match quality = 2.22 (Wave Up Match) ; RSA = 0
Observed: final set = 2.000 mm; blow count = 500 b/m
Computed: final set = 2.187 mm; blow count = 457 b/m

max. Top Comp. Stress = 181.5 MPa (T= 20.7 ms, max= 1.014 x Top)
max. Comp. Stress = 183.9 MPa (Z= 6.2 m, T= 21.7 ms)
max. Tens. Stress = -47.07 MPa (Z= 14.4 m, T= 51.7 ms)
max. Energy (EMX) = 33.59 kJ; max. Measured Top Displ. (DMX)=20.67 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2551.3	-340.2	181.5	-24.20	33.59	4.5	20.267
2	2.1	2549.8	-378.8	181.4	-26.94	32.68	4.5	19.461
4	4.1	2550.3	-459.3	181.4	-32.67	30.95	4.5	17.901
6	6.2	2586.0	-534.8	183.9	-38.03	29.75	4.4	16.595
7	7.2	2512.8	-500.5	178.7	-35.60	27.72	4.4	15.929
8	8.3	2527.2	-538.5	179.7	-38.30	27.03	4.4	15.235
9	9.3	2480.4	-533.0	176.4	-37.91	25.51	4.4	14.515
10	10.3	2487.6	-570.7	176.9	-40.59	24.75	4.3	13.786
11	11.3	2469.4	-585.8	175.6	-41.66	23.73	4.3	13.086
12	12.4	2483.0	-622.0	176.6	-44.24	23.09	4.3	12.411
13	13.4	2460.5	-626.4	175.0	-44.55	21.98	4.3	11.742
14	14.4	2494.9	-661.8	177.4	-47.07	21.30	4.2	11.037
15	15.5	2428.9	-615.5	172.8	-43.78	19.53	4.1	10.326
16	16.5	2492.4	-646.2	177.3	-45.96	18.75	4.0	9.581
17	17.5	2352.0	-530.7	167.3	-37.75	16.36	3.9	8.876
18	18.6	2436.1	-549.4	173.3	-39.07	15.69	3.8	8.183
19	19.6	2223.5	-430.7	158.1	-30.63	13.22	3.7	7.543
20	20.6	2322.2	-442.6	165.2	-31.48	12.60	3.5	6.885
21	21.7	2186.7	-377.4	155.5	-26.84	10.61	3.4	6.277
22	22.7	2343.5	-384.0	166.7	-27.31	10.19	3.1	5.711
23	23.7	2180.1	-376.7	155.1	-26.79	8.97	3.0	5.245
24	24.8	2155.9	-378.2	153.3	-26.90	8.38	3.1	4.778
Absolute	6.2			183.9			(T =	21.7 ms)
	14.4				-47.07		(T =	51.7 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3446.8	3265.0	3083.3	2901.5	2719.8	2538.0	2356.3	2174.6	1992.8	1811.1
RX	3446.8	3265.0	3083.3	2901.5	2719.8	2543.9	2416.0	2370.4	2325.8	2281.9
RU	3446.8	3265.0	3083.3	2901.5	2719.8	2538.0	2356.3	2174.6	1992.8	1811.1

RAU = 2137.3 (kN); RA2 = 2184.0 (kN)

Current CAPWAP Ru = 2267.0 (kN); Corresponding J(RP)= 0.65; matches RX9 within 5%

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
4.53	20.53	2572.7	2691.6	2691.6	20.669	1.714	2.000	34.9	3079.2

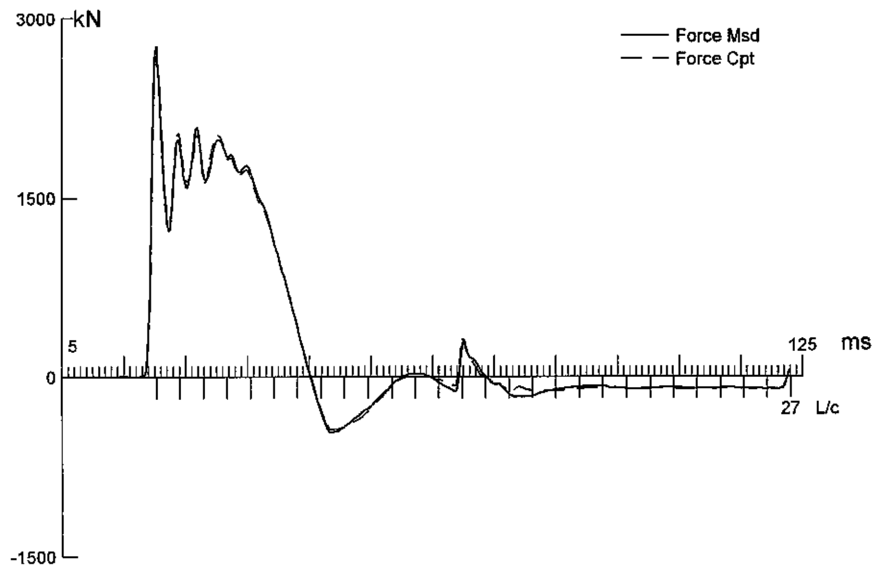
PILE PROFILE AND PILE MODEL

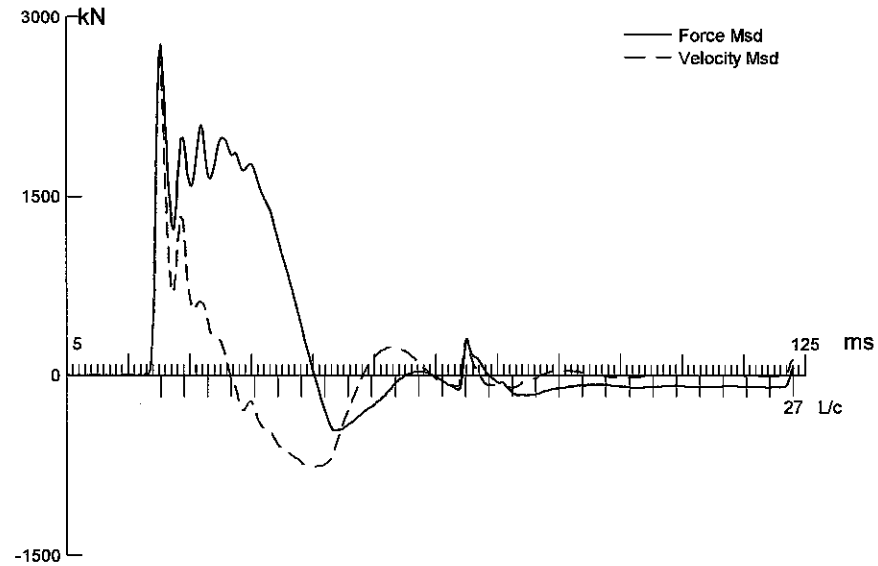
Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	140.60	206842.7	77.287	1.220
24.75	140.60	206842.7	77.287	1.220

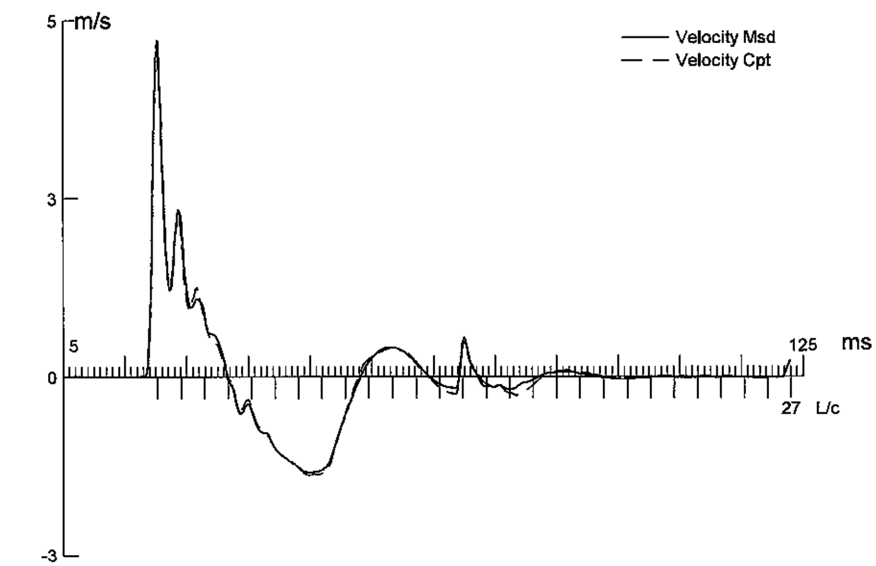
Toe Area 0.093 m^2

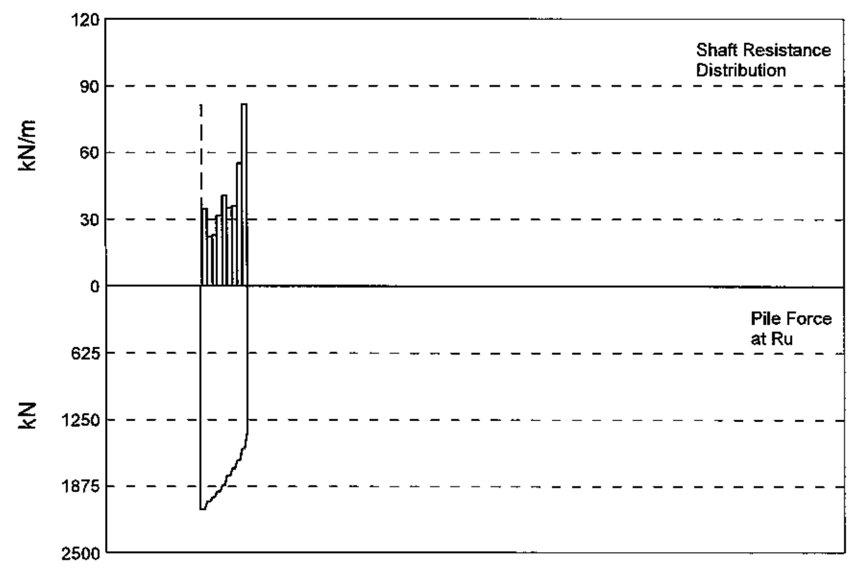
Top Segment Length 1.03 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.201 ms, Wave Speed 5123.0 m/s, 2L/c 9.7 ms


3000
kN
1500
0
-1500
5
125
ms
27
L/c
Force Msd
Force Cpt


3000
kN
1500
0
-1500
5
125
ms
27
L/c
Force Msd
Velocity Msd


5
m/s
3
0
-3
5
125
ms
27
L/c
Velocity Msd
Velocity Cpt


120
90
60
30
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted] Pile: PILE 114 Test: 11-Feb-2011 14:19:
RE-STRIKE; Blow: 7 CAPWAP(R) 2006-2
[redacted] OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2093.8; along Shaft 739.2; at Toe 1354.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2093.8					
1	3.1	1.8	71.5	2022.3	71.5	39.90	32.28	0.973	7.500
2	5.1	3.8	45.2	1977.1	116.7	22.00	17.80	0.973	7.501
3	7.2	5.9	47.2	1929.9	163.9	22.97	18.59	0.973	7.501
4	9.2	8.0	65.4	1864.5	229.3	31.83	25.75	0.973	7.501
5	11.3	10.0	83.3	1781.2	312.6	40.54	32.80	0.973	7.501
6	13.4	12.1	72.1	1709.1	384.7	35.09	28.39	0.973	7.501
7	15.4	14.1	73.5	1635.6	458.2	35.77	28.94	0.973	7.501
8	17.5	16.2	112.8	1522.8	571.0	54.90	44.42	0.973	7.501
9	19.5	18.2	168.2	1354.6	739.2	81.86	66.23	0.973	6.832
Avg. Shaft			82.1			40.55	32.81	0.973	7.348
Toe			1354.6				14187.26	0.277	6.843

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.267	0.661
Unloading Quake	(% of loading quake)	50	49
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	74	
Resistance Gap (included in Toe Quake) (mm)			1.466
Soil Plug Weight	(kN)		0.74

CAPWAP match quality = 1.61 (Wave Up Match) ; RSA = 0
Observed: final set = 2.000 mm; blow count = 500 b/m
Computed: final set = 2.901 mm; blow count = 345 b/m

max. Top Comp. Stress = 197.7 MPa (T= 20.7 ms, max= 1.048 x Top)
max. Comp. Stress = 207.3 MPa (Z= 3.1 m, T= 21.1 ms)
max. Tens. Stress = -38.22 MPa (Z= 3.1 m, T= 49.3 ms)
max. Energy (EMX) = 35.79 kJ; max. Measured Top Displ. (DMX)=18.87 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2780.3	-473.9	197.7	-33.71	35.79	4.6	18.109
2	2.1	2852.8	-506.5	202.9	-36.03	35.16	4.5	17.441
3	3.1	2914.1	-537.4	207.3	-38.22	34.51	4.4	16.759
4	4.1	2633.0	-453.9	187.3	-32.29	30.46	4.3	16.090
5	5.1	2682.7	-484.2	190.8	-34.44	29.81	4.2	15.412
6	6.2	2533.9	-449.5	180.2	-31.97	27.25	4.1	14.744
7	7.2	2592.8	-477.1	184.4	-33.93	26.62	4.0	14.074
8	8.2	2459.2	-448.8	174.9	-31.92	24.27	3.9	13.422
9	9.2	2530.2	-474.0	180.0	-33.71	23.67	3.8	12.773
10	10.3	2343.4	-427.7	166.7	-30.42	21.02	3.6	12.142
11	11.3	2410.3	-450.0	171.4	-32.00	20.44	3.5	11.504
12	12.3	2163.7	-380.7	153.9	-27.08	17.61	3.4	10.887
13	13.4	2224.2	-399.4	158.2	-28.41	17.06	3.3	10.264
14	14.4	2037.5	-328.8	144.9	-23.39	14.90	3.2	9.663
15	15.4	2117.7	-334.9	150.6	-23.82	14.39	3.0	9.066
16	16.4	1993.5	-273.6	141.8	-19.46	12.57	2.9	8.494
17	17.5	2063.0	-278.7	146.7	-19.82	12.11	2.7	7.922
18	18.5	1724.6	-216.3	122.7	-15.38	9.97	2.8	7.386
19	19.5	1627.9	-217.2	115.8	-15.45	7.62	2.9	6.847
Absolute	3.1			207.3			(T =	21.1 ms)
	3.1				-38.22		(T =	49.3 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3378.1	3159.4	2940.6	2721.9	2503.1	2284.3	2065.6	1846.8	1628.1	1409.3
RX	3378.1	3159.4	2940.6	2721.9	2503.1	2284.3	2149.6	2111.6	2076.6	2044.4
RU	3378.1	3159.4	2940.6	2721.9	2503.1	2284.3	2065.6	1846.8	1628.1	1409.3

RAU = 1502.4 (kN); RA2 = 1918.2 (kN)

Current CAPWAP Ru = 2093.8 (kN); Corresponding J(RP)= 0.59; J(RX) = 0.75

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.81	20.45	2728.1	2837.7	2837.7	18.866	2.039	2.000	37.2	3563.8

PILE PROFILE AND PILE MODEL

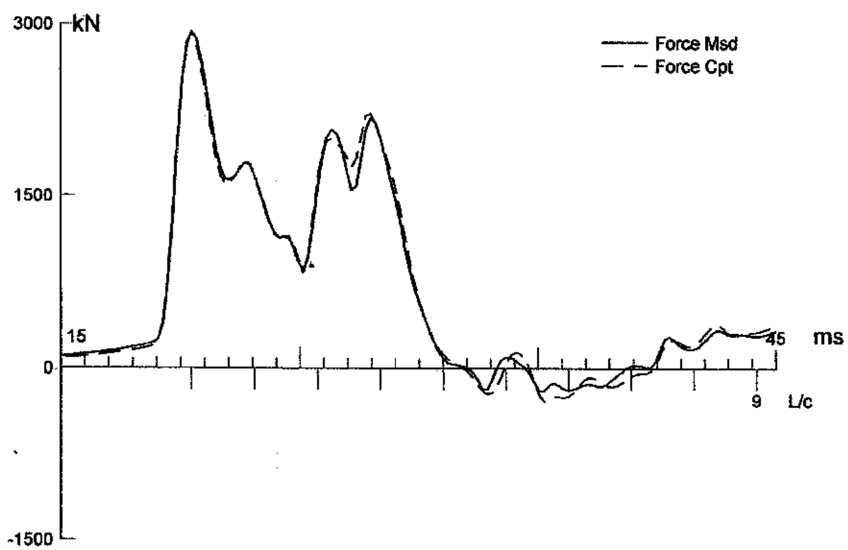
Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	140.60	206842.7	77.287	1.236
19.52	140.60	206842.7	77.287	1.236

Toe Area 0.095 m^2

Top Segment Length 1.03 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.201 ms, Wave Speed 5123.0 m/s, 2L/c 7.6 ms

Site 8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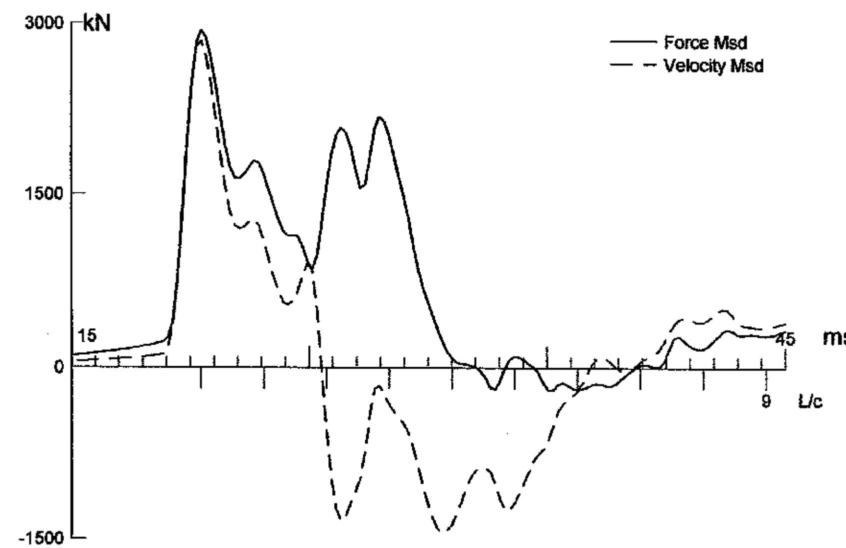
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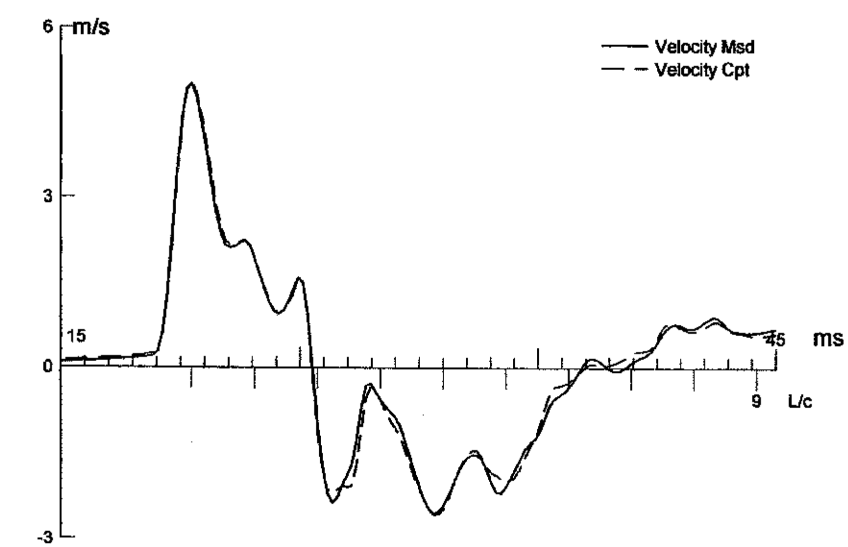
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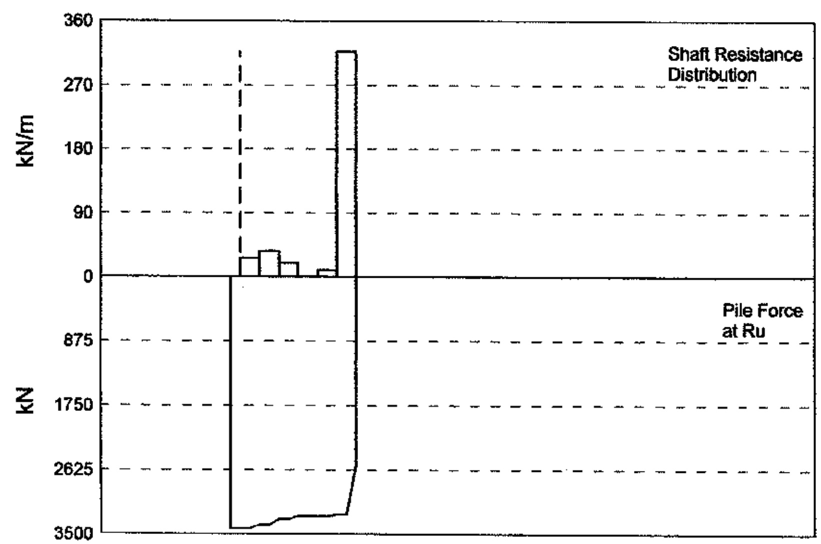
Velocity Cpt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2625
3500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3420.4; along Shaft 850.7; at Toe 2569.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				3420.4					
1	3.1	1.8	55.4	3365.1	55.4	30.50	25.00	0.934	7.500
2	5.2	3.9	76.0	3289.1	131.4	36.60	30.00	0.934	7.500
3	7.3	6.0	40.5	3248.5	171.9	19.52	16.00	0.934	7.500
4	9.3	8.0	0.0	3248.5	171.9	0.00	0.00	0.000	7.500
5	11.4	10.1	20.0	3228.5	191.9	9.63	7.89	0.934	7.104
6	13.5	12.2	658.8	2569.7	850.7	317.21	260.01	0.934	4.658
Avg. Shaft			141.8			69.73	57.16	0.934	5.290
Toe			2569.7				27623.78	0.080	5.250

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.400	0.362
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	98	36
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	98	
Resistance Gap (included in Toe Quake) (mm)			1.669
Soil Plug Weight	(kN)		0.33

CAPWAP match quality = 3.48 (Wave Up Match) ; RSA = 0
Observed: final set = 0.870 mm; blow count = 1150 b/m
Computed: final set = 0.303 mm; blow count = 3302 b/m

max. Top Comp. Stress = 209.1 MPa (T= 20.9 ms, max= 1.342 x Top)
max. Comp. Stress = 280.6 MPa (Z= 13.5 m, T= 23.5 ms)
max. Tens. Stress = -35.33 MPa (Z= 13.5 m, T= 36.1 ms)
max. Energy (EMX) = 28.80 kJ; max. Measured Top Displ. (DMX)=15.71 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2939.9	-409.2	209.1	-29.11	28.80	5.0	15.563
2	2.1	2969.9	-446.0	211.2	-31.72	28.24	5.0	14.919
3	3.1	3023.7	-462.1	215.1	-32.86	27.64	4.9	14.276
4	4.2	2892.3	-410.5	205.7	-29.19	25.19	4.8	13.700
5	5.2	2950.9	-484.3	209.9	-34.45	24.66	4.7	13.066
6	6.2	2751.3	-429.7	195.7	-30.56	21.45	4.6	12.360
7	7.3	2778.7	-451.8	197.6	-32.14	20.48	4.6	11.504
8	8.3	2894.5	-418.5	205.9	-29.77	18.19	4.6	10.588
9	9.3	2948.0	-421.4	209.7	-29.97	17.11	4.6	9.725
10	10.4	2864.3	-436.2	203.7	-31.03	16.09	4.6	8.849
11	11.4	2958.8	-461.7	210.4	-32.84	14.69	5.1	7.881
12	12.5	3446.9	-483.1	245.2	-34.36	12.61	5.1	6.748
13	13.5	3945.7	-496.8	280.6	-35.33	6.15	3.9	5.563
Absolute	13.5			280.6			(T =	23.5 ms)
	13.5				-35.33		(T =	36.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3951.5	3768.3	3585.0	3401.7	3218.4	3035.1	2851.8	2668.5	2485.2	2301.9
RX	4210.4	4114.4	4018.4	3922.4	3826.4	3730.4	3663.3	3603.2	3543.0	3482.9
RU	4032.0	3856.8	3681.5	3506.3	3331.0	3155.8	2980.6	2805.3	2630.1	2454.8

RAU = 1952.8 (kN); RA2 = 3295.0 (kN)

Current CAPWAP Ru = 3420.5 (kN); Corresponding J(RP)= 0.29; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.01	20.68	2845.2	2939.3	2939.3	15.714	3.861	0.870	29.1	3512.3

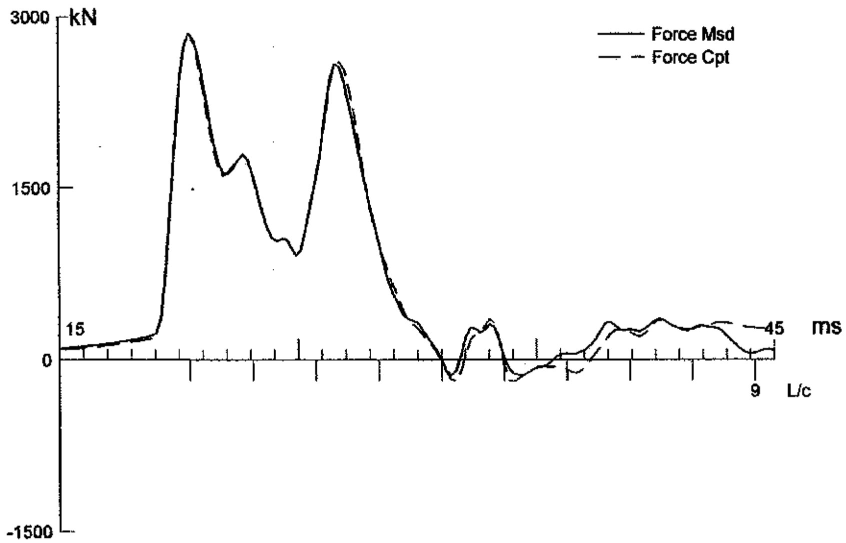
PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	140.60	206842.7	77.287	1.220
13.50	140.60	206842.7	77.287	1.220

Toe Area 0.093 m^2

Top Segment Length 1.04 m, Top Impedance 567.67 kN/m/s

Pile Damping 1.0 %, Time Incr 0.203 ms, Wave Speed 5123.0 m/s, 2L/c 5.3 ms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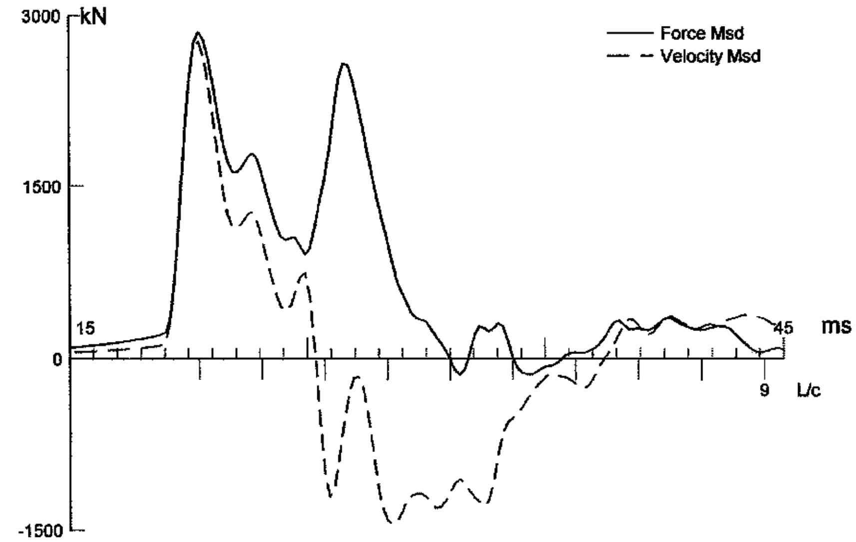
Force Msd
Force Cpt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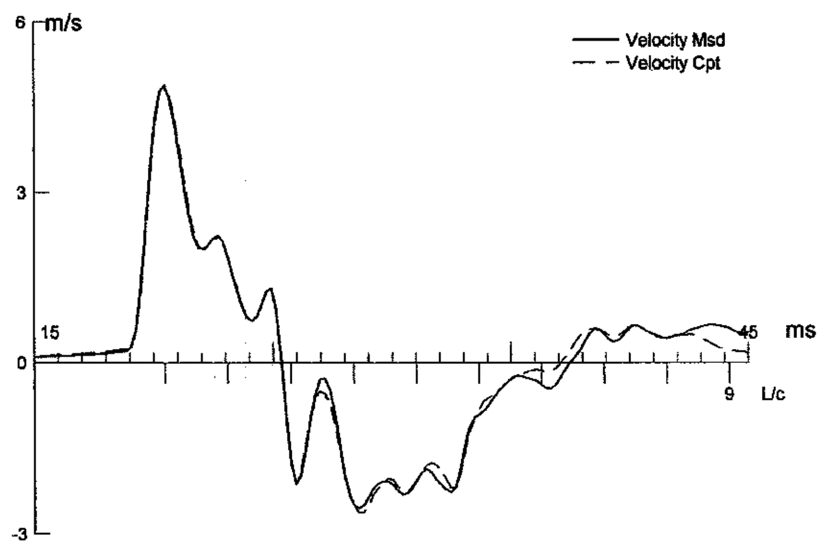
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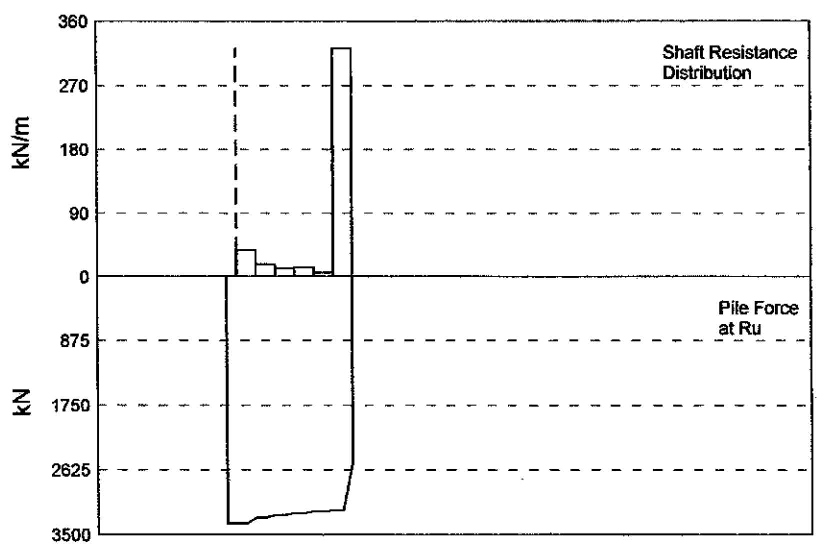
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875
1750
2625
3500
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]

[redacted]ile 002 Test: 14-Aug-2009 16:22:
STEEL HP12-74; Blow: 6; Re-Strike CAPWAP(R) 2006-2
[redacted] OP: GR

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3350.0; along Shaft 847.4; at Toe 2502.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				3350.0					
1	3.1	2.0	78.0	3272.1	78.0	38.69	31.71	1.093	6.544
2	5.2	4.1	36.5	3235.5	114.5	17.58	14.41	1.093	6.542
3	7.3	6.2	24.4	3211.2	138.8	11.72	9.61	1.093	6.542
4	9.3	8.2	26.8	3184.4	165.6	12.88	10.56	1.093	6.542
5	11.4	10.3	12.2	3172.2	177.8	5.87	4.81	1.093	6.542
6	13.5	12.4	669.7	2502.6	847.4	322.43	264.29	1.093	4.818
Avg. Shaft			141.2			68.34	56.02	1.093	5.180
Toe			2502.6				26902.22	0.113	4.600

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.632	0.498
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	47	
Resistance Gap (included in Toe Quake) (mm)			1.143
Soil Plug Weight	(kN)		0.03

CAPWAP match quality = 3.63 (Wave Up Match) ; RSA = 0
Observed: final set = 0.667 mm; blow count = 1500 b/m
Computed: final set = 0.546 mm; blow count = 1830 b/m

max. Top Comp. Stress = 204.3 MPa (T= 20.9 ms, max= 1.359 x Top)
max. Comp. Stress = 277.6 MPa (Z= 13.5 m, T= 23.5 ms)
max. Tens. Stress = -28.81 MPa (Z= 3.1 m, T= 33.4 ms)
max. Energy (EMX) = 26.90 kJ; max. Measured Top Displ. (DMX)=14.98 mm

[redacted] Pile 002 Test: 14-Aug-2009 16:22:

STEEL HP12-74; Blow: 6; Re-Strike CAPWAP(R) 2006-2

[redacted] OP: GR

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2872.1	-200.4	204.3	-14.26	26.90	4.9	14.616
2	2.1	2922.8	-215.5	207.9	-15.33	26.39	4.8	14.032
3	3.1	2999.2	-405.1	213.3	-28.81	25.94	4.6	13.457
4	4.2	2730.6	-318.3	194.2	-22.64	22.61	4.6	12.950
5	5.2	2767.2	-339.8	196.8	-24.17	22.09	4.5	12.383
6	6.2	2652.0	-306.0	188.6	-21.77	20.10	4.5	11.638
7	7.3	2723.9	-333.0	193.7	-23.68	18.99	4.4	10.784
8	8.3	2930.6	-324.3	208.4	-23.06	17.06	4.4	9.850
9	9.3	2971.5	-346.3	211.3	-24.63	16.04	4.4	8.974
10	10.4	2868.4	-337.5	204.0	-24.00	14.36	4.5	8.123
11	11.4	2947.1	-355.3	209.6	-25.27	13.03	4.8	7.132
12	12.5	3437.4	-359.6	244.5	-25.58	11.02	4.4	6.012
13	13.5	3902.8	-379.0	277.6	-26.95	5.79	3.1	4.830
Absolute	13.5			277.6			(T =	23.5 ms)
	3.1				-28.81		(T =	33.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3977.0	3802.2	3627.4	3452.5	3277.7	3102.9	2928.1	2753.3	2578.4	2403.6
RX	4264.0	4112.1	3986.5	3888.2	3809.7	3740.2	3670.8	3601.4	3532.0	3462.5
RU	4018.8	3848.2	3677.5	3506.9	3336.2	3165.6	2995.0	2824.3	2653.7	2483.0

RAU = 2457.8 (kN); RA2 = 3338.2 (kN)

Current CAPWAP Ru = 3350.0 (kN); Corresponding J(RP)= 0.36; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.99	20.68	2831.4	2893.8	2964.1	14.978	0.560	0.667	27.9	3561.0

PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	140.60	206842.7	77.287	1.220
13.50	140.60	206842.7	77.287	1.220

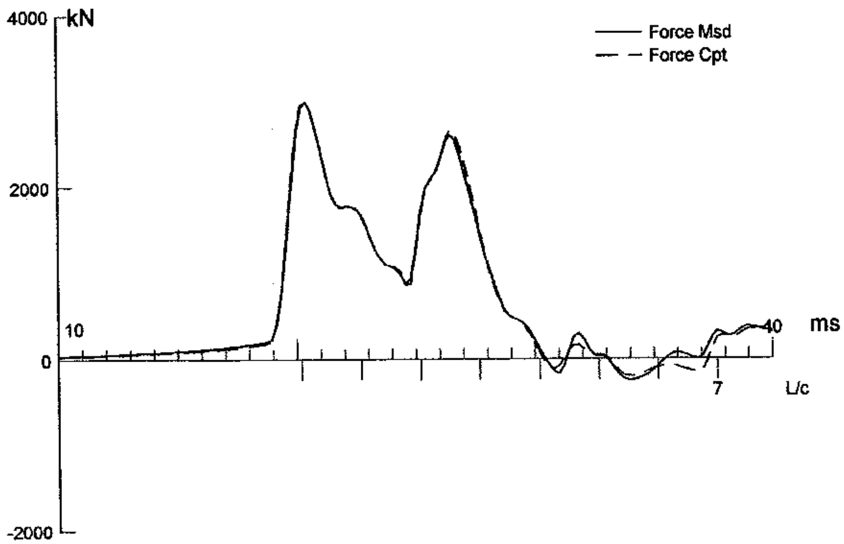
Toe Area 0.093 m^2

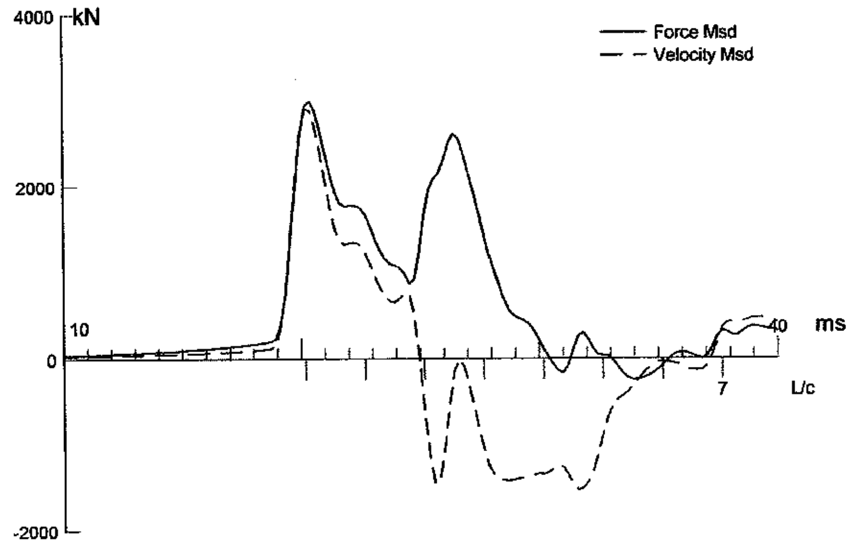
Top Segment Length 1.04 m, Top Impedance 567.67 kN/m/s

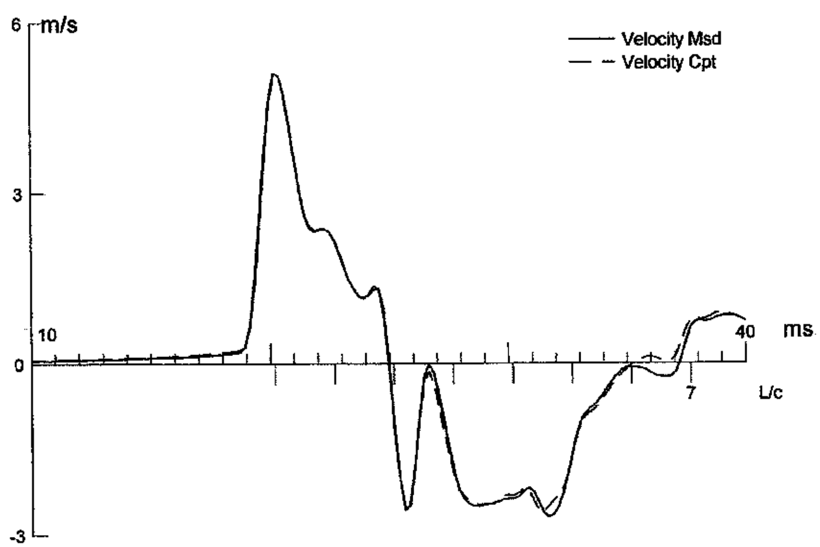
Pile Damping 1.0 %, Time Incr 0.203 ms, Wave Speed 5123.0 m/s, 2L/c 5.3 ms

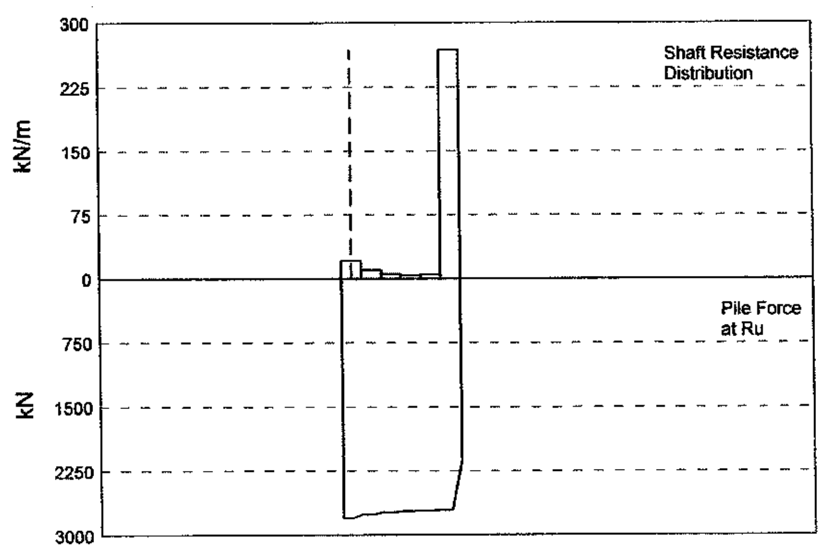
[redacted] PILE #3; STEEL HP 310X110; Blow: 10 (Test: 10-Sep-2009 11:12:); Re-Strike

08-Mar-2010
CAPWAP(R) 2006-2









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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2802.9; along Shaft 668.1; at Toe 2134.8 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2802.9					
1	2.1	0.9	44.9	2758.0	44.9	50.84	41.00	1.304	4.340
2	4.3	3.0	21.2	2736.8	66.1	9.92	8.00	1.304	4.340
3	6.4	5.1	12.0	2724.8	78.1	5.64	4.55	1.304	4.340
4	8.5	7.3	7.9	2716.8	86.0	3.72	3.00	1.304	4.340
5	10.7	9.4	10.0	2706.8	96.0	4.69	3.78	1.304	4.340
6	12.8	11.5	572.1	2134.8	668.1	268.15	216.25	1.304	4.271
Avg. Shaft			111.3			57.84	46.65	1.304	4.281
Toe			2134.8				22213.98	0.678	4.814

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.534	2.549
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	100	57
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			1.527
Soil Plug Weight	(kN)		0.55

CAPWAP match quality	=	3.12	(Wave Up Match) ; RSA = 0
Observed: final set	=	1.000 mm;	blow count = 1000 b/m
Computed: final set	=	0.855 mm;	blow count = 1170 b/m
max. Top Comp. Stress	=	217.5 MPa	(T= 20.8 ms, max= 1.410 x Top)
max. Comp. Stress	=	306.6 MPa	(Z= 12.8 m, T= 23.3 ms)
max. Tens. Stress	=	-41.87 MPa	(Z= 12.8 m, T= 33.9 ms)
max. Energy (EMX)	=	29.12 kJ;	max. Measured Top Displ. (DMX)=15.58 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.1	3058.7	-270.9	217.5	-19.27	29.12	5.0	15.144
2	2.1	3143.7	-312.4	223.6	-22.22	28.59	4.9	14.510
3	3.2	2847.3	-302.9	202.5	-21.54	25.42	4.8	13.903
4	4.3	2888.6	-399.1	205.4	-28.38	24.87	4.8	13.253
5	5.3	2766.2	-427.9	196.7	-30.44	22.93	4.7	12.531
6	6.4	2895.7	-463.2	206.0	-32.95	21.84	4.7	11.633
7	7.5	3148.6	-472.0	223.9	-33.57	19.89	4.7	10.614
8	8.5	3227.3	-518.4	229.5	-36.87	18.42	4.6	9.580
9	9.6	3219.5	-531.8	229.0	-37.83	16.65	4.6	8.537
10	10.7	3400.1	-556.6	241.8	-39.59	14.78	4.8	7.375
11	11.7	3895.9	-563.0	277.1	-40.04	12.11	4.6	6.060
12	12.8	4311.4	-588.7	306.6	-41.87	6.51	3.3	4.883
Absolute	12.8			306.6			(T =	23.3 ms)
	12.8				-41.87		(T =	33.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4680.2	4540.7	4401.2	4261.7	4122.3	3982.8	3843.3	3703.8	3564.3	3424.9
RX	4680.2	4563.0	4463.4	4363.8	4264.1	4164.5	4064.9	3965.3	3865.7	3766.1
RU	4749.0	4616.4	4483.8	4351.2	4218.6	4086.0	3953.4	3820.8	3688.2	3555.6

RAU = 2448.8 (kN); RA2 = 3118.8 (kN)

Current CAPWAP Ru = 2802.9 (kN);

Case Method matching requires higher damping factor - please check with PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.24	20.40	2976.3	3098.6	3098.6	15.581	0.840	1.000	29.9	3609.8

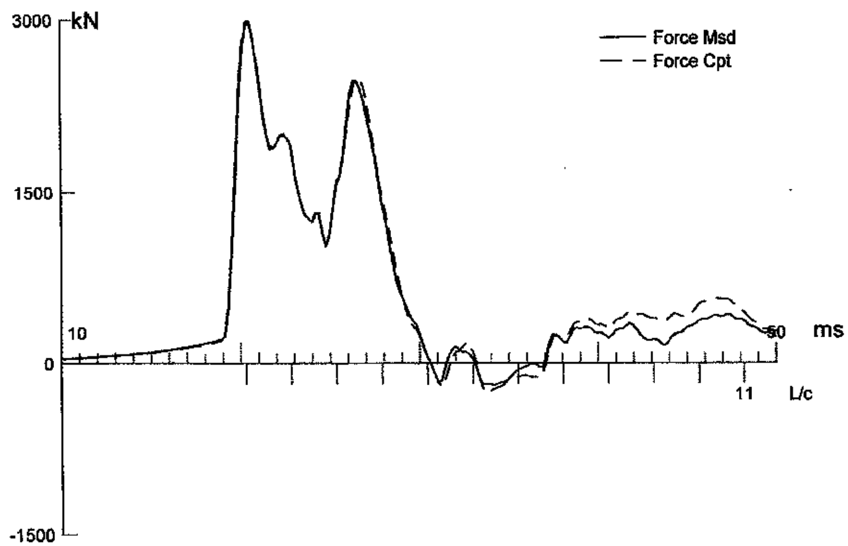
PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	140.60	206874.8	77.300	1.240
12.80	140.60	206874.8	77.300	1.240

Toe Area 0.096 m^2

Top Segment Length 1.07 m, Top Impedance 567.76 kN/m/s

Pile Damping 1.0 %, Time Incr 0.208 ms, Wave Speed 5123.0 m/s, 2L/c 5.0 ms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ms
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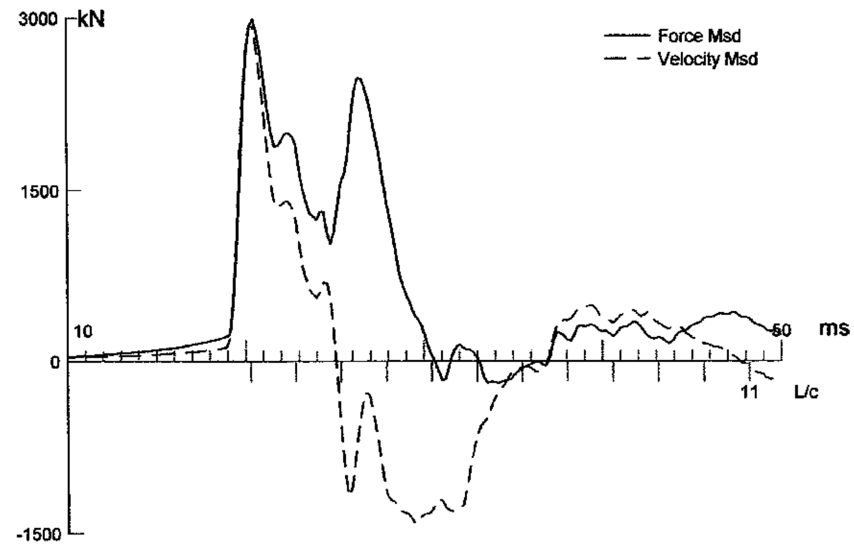
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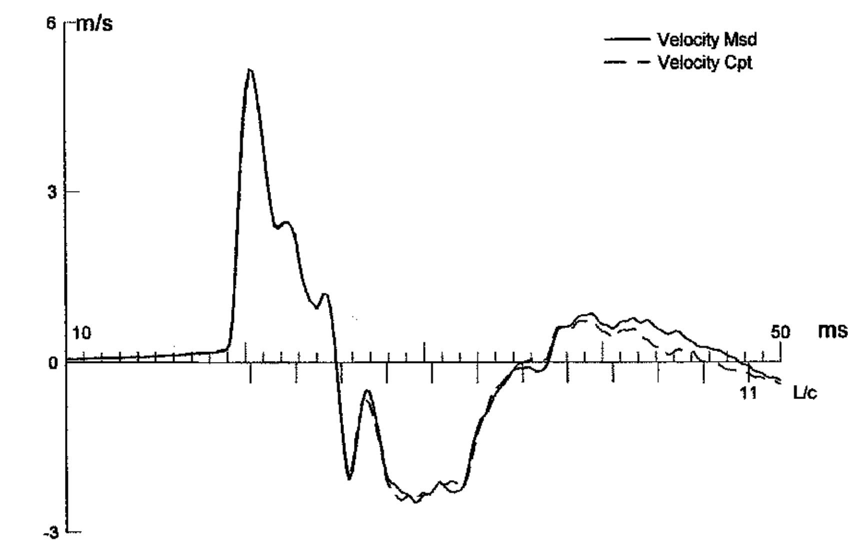
Velocity Msd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m/s
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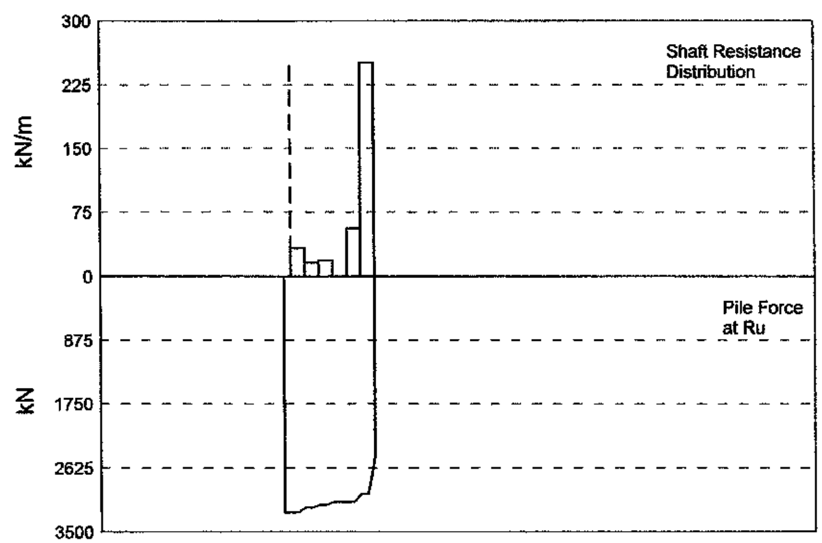
10
50
ms
11
L/c
Velocity Msd
Velocity Cpt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225
150
75
0
875
1750
2625
3500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3231.5; along Shaft 751.5; at Toe 2480.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				3231.5					
1	3.0	1.2	66.4	3165.1	66.4	57.72	46.55	1.208	4.359
2	5.0	3.2	32.6	3132.5	99.0	16.32	13.16	1.208	4.358
3	7.0	5.2	37.6	3094.8	136.7	18.82	15.18	1.208	4.358
4	9.0	7.2	0.0	3094.8	136.7	0.00	0.00	0.000	4.358
5	11.0	9.2	112.9	2981.9	249.6	56.47	45.54	1.208	4.358
6	13.0	11.2	501.9	2480.0	751.5	250.96	202.39	1.208	4.254
Avg. Shaft			125.3			67.40	54.36	1.208	4.289
Toe			2480.0				25806.00	0.080	4.459

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.599	0.349
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	100	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	7	
Resistance Gap (included in Toe Quake) (mm)			1.223
Soil Plug Weight	(kN)		0.13

CAPWAP match quality = 3.05 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.491 mm; blow count = 2036 b/m

max. Top Comp. Stress = 216.4 MPa (T= 20.9 ms, max= 1.176 x Top)
max. Comp. Stress = 254.6 MPa (Z= 13.0 m, T= 23.4 ms)
max. Tens. Stress = -40.95 MPa (Z= 11.0 m, T= 33.6 ms)
max. Energy (EMX) = 30.50 kJ; max. Measured Top Displ. (DMX)=15.64 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3043.2	-323.1	216.4	-22.98	30.50	5.1	15.092
2	2.0	3126.9	-341.5	222.4	-24.29	29.99	5.0	14.469
3	3.0	3232.7	-398.6	229.9	-28.35	29.32	4.8	13.806
4	4.0	2834.8	-438.9	201.6	-31.21	25.34	4.7	13.203
5	5.0	2897.1	-471.6	206.1	-33.54	24.73	4.6	12.557
6	6.0	2741.4	-466.7	195.0	-33.19	22.36	4.6	11.808
7	7.0	2789.0	-517.6	198.4	-36.81	21.19	4.5	10.922
8	8.0	2974.3	-492.0	211.5	-34.99	18.30	4.5	9.932
9	9.0	3052.3	-520.4	217.1	-37.01	16.82	4.4	8.930
10	10.0	3017.4	-546.2	214.6	-38.85	15.49	4.3	7.971
11	11.0	3116.2	-575.8	221.6	-40.95	14.00	4.6	6.961
12	12.0	3313.8	-462.2	235.7	-32.88	10.00	4.5	5.892
13	13.0	3579.8	-486.9	254.6	-34.63	5.54	3.4	4.839
Absolute	13.0			254.6			(T =	23.4 ms)
	11.0				-40.95		(T =	33.6 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4308.8	4130.1	3951.4	3772.6	3593.9	3415.1	3236.4	3057.7	2878.9	2700.2
RX	4308.8	4130.1	4030.0	3933.4	3836.7	3740.1	3643.5	3546.9	3450.3	3353.6
RU	4331.1	4154.6	3978.1	3801.6	3625.1	3448.6	3272.0	3095.5	2919.0	2742.5

RAU = 2470.7 (kN); RA2 = 3578.8 (kN)

Current CAPWAP Ru = 3231.5 (kN); Corresponding J(RP)= 0.60; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.29	20.50	3005.5	3090.8	3090.8	15.643	0.656	1.000	31.2	3754.1

PILE PROFILE AND PILE MODEL

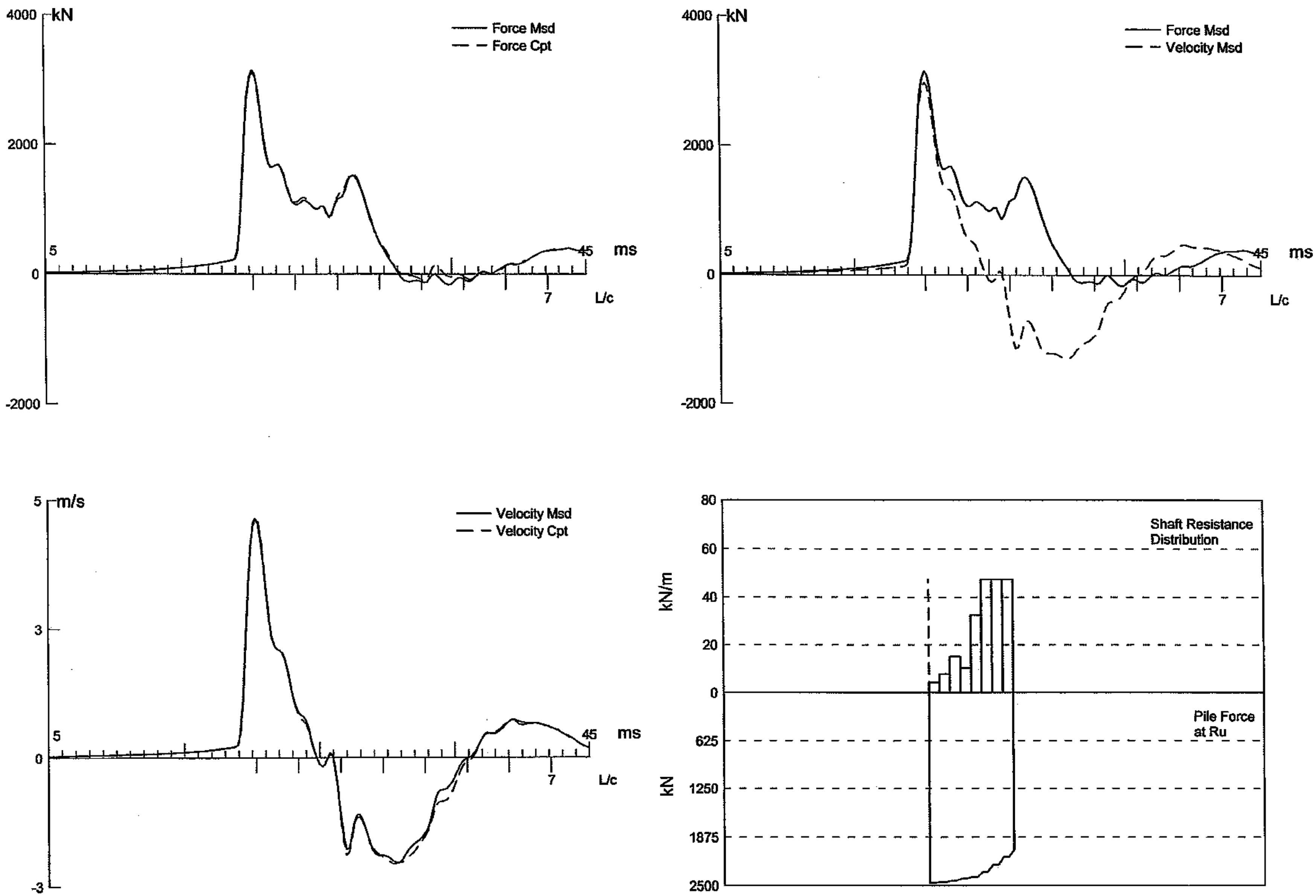
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	140.60	206874.8	77.300	1.240
13.00	140.60	206874.8	77.300	1.240

Toe Area 0.096 m^2

Top Segment Length 1.00 m, Top Impedance 567.76 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 5.1 ms

Site 9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Force Msd
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Force Msd
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m/s
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0
-3
Velocity Msd
Velocity Cpt
80
60
40
20
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2464.1; along Shaft 424.7; at Toe 2039.4 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2464.1				
1	2.0	1.4	8.6	2455.5	8.6	6.14	5.04	1.313
2	4.0	3.4	15.5	2440.0	24.1	7.75	6.35	1.313
3	6.0	5.4	30.0	2410.0	54.1	15.00	12.30	1.313
4	8.0	7.4	20.6	2389.4	74.7	10.30	8.44	1.313
5	10.0	9.4	65.0	2324.4	139.7	32.50	26.64	1.313
6	12.0	11.4	95.0	2229.4	234.7	47.50	38.93	1.313
7	14.0	13.4	95.0	2134.4	329.7	47.50	38.93	1.313
8	16.0	15.4	95.0	2039.4	424.7	47.50	38.93	1.313
Avg. Shaft			53.1			27.58	22.60	1.313
Toe			2039.4				21221.64	0.722

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	3.382	4.556
Case Damping Factor		0.870	2.298
Damping Type			Smith
Unloading Quake	(% of loading quake)	100	118
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	85	
Resistance Gap (included in Toe Quake)	(mm)		1.040
Soil Plug Weight	(kN)		0.45

CAPWAP match quality = 1.37 (Wave Up Match) ; RSA = 0
Observed: final set = 0.600 mm; blow count = 1667 b/m
Computed: final set = 1.372 mm; blow count = 729 b/m

max. Top Comp. Stress = 196.9 MPa (T= 20.7 ms, max= 1.010 x Top)
max. Comp. Stress = 198.8 MPa (Z= 4.0 m, T= 21.3 ms)
max. Tens. Stress = -31.84 MPa (Z= 10.0 m, T= 35.1 ms)
max. Energy (EMX) = 25.58 kJ; max. Measured Top Displ. (DMX)=13.23 mm

[redacted]102, [redacted]
October 24, 2008; Blow: 575
[redacted]

Test: 24-Oct-2008 11:39:
CAPWAP(R) 2006-2
OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3125.1	-169.5	196.9	-10.68	25.58	4.7	12.772
2	2.0	3148.2	-198.1	198.4	-12.48	25.39	4.6	12.435
3	3.0	3115.0	-227.5	196.3	-14.33	24.78	4.6	12.151
4	4.0	3155.7	-275.9	198.8	-17.39	24.61	4.5	11.836
5	5.0	3093.4	-307.5	194.9	-19.37	23.59	4.4	11.454
6	6.0	3132.0	-374.1	197.4	-23.57	23.25	4.4	11.011
7	7.0	2966.8	-380.8	186.9	-23.99	21.50	4.3	10.543
8	8.0	3037.8	-435.7	191.4	-27.46	21.13	4.2	10.057
9	9.0	2999.1	-456.3	189.0	-28.75	19.80	4.1	9.525
10	10.0	3112.6	-505.2	196.1	-31.84	19.15	3.9	8.889
11	11.0	2839.2	-451.2	178.9	-28.43	16.08	3.7	8.181
12	12.0	2942.1	-485.1	185.4	-30.57	15.11	3.5	7.420
13	13.0	2490.6	-379.9	156.9	-23.94	11.59	3.4	6.674
14	14.0	2530.3	-404.1	159.4	-25.46	10.76	3.5	5.962
15	15.0	2788.4	-329.6	175.7	-20.77	8.01	3.4	5.275
16	16.0	3113.6	-348.0	196.2	-21.93	6.88	2.8	4.599
Absolute	4.0			198.8			(T =	21.3 ms)
	10.0				-31.84		(T =	35.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4012.2	3793.1	3574.0	3354.9	3135.8	2916.7	2697.6	2478.5	2259.4	2040.3
RX	4086.6	3890.5	3694.5	3498.4	3323.1	3156.6	2990.2	2823.7	2684.4	2560.8
RU	4065.3	3851.5	3637.8	3424.0	3210.2	2996.4	2782.7	2568.9	2355.1	2141.3

RAU = 1886.2 (kN); RA2 = 2486.6 (kN)

Current CAPWAP Ru = 2464.1 (kN); Corresponding J(RP)= 0.71; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.68	20.50	3001.0	3202.2	3202.2	13.234	0.549	0.600	26.2	3783.3

PILE PROFILE AND PILE MODEL

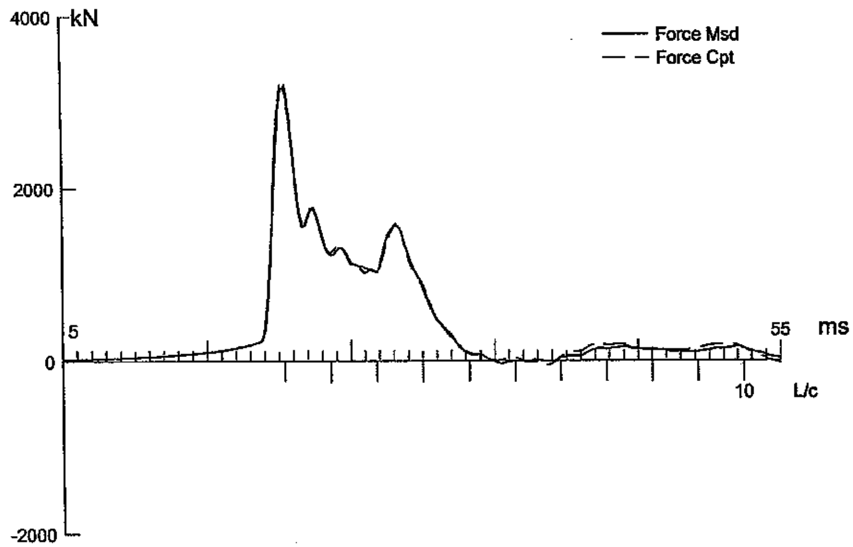
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206874.8	77.300	1.220
16.00	158.70	206874.8	77.300	1.220

Toe Area 0.096 m^2

Top Segment Length 1.00 m, Top Impedance 640.86 kN/m/s

Analysis: 13-Jan-2009

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 6.2 ms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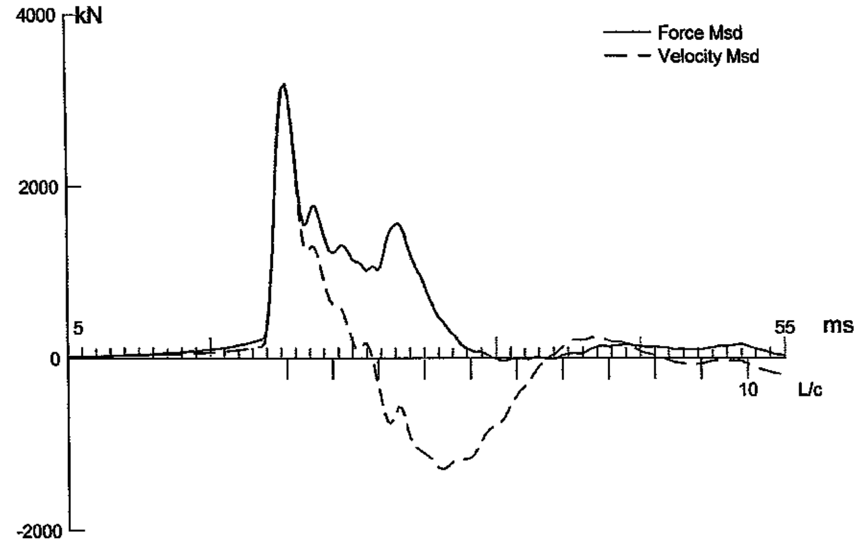
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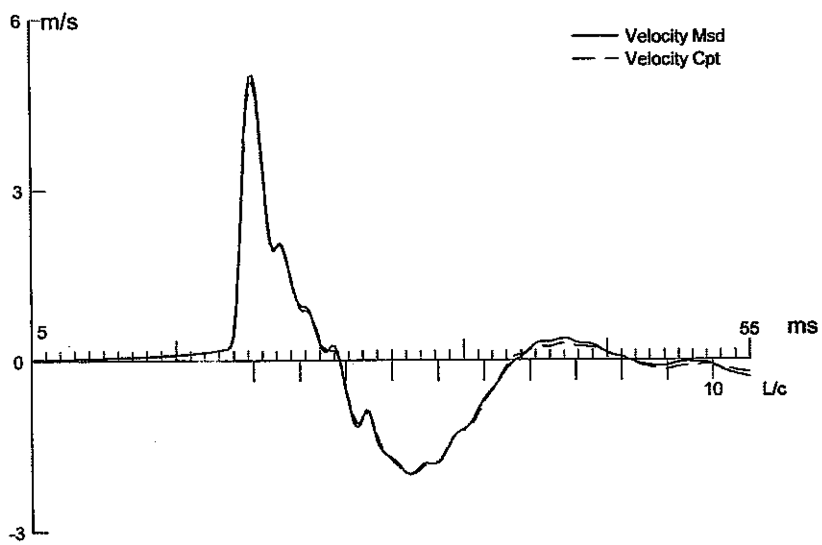
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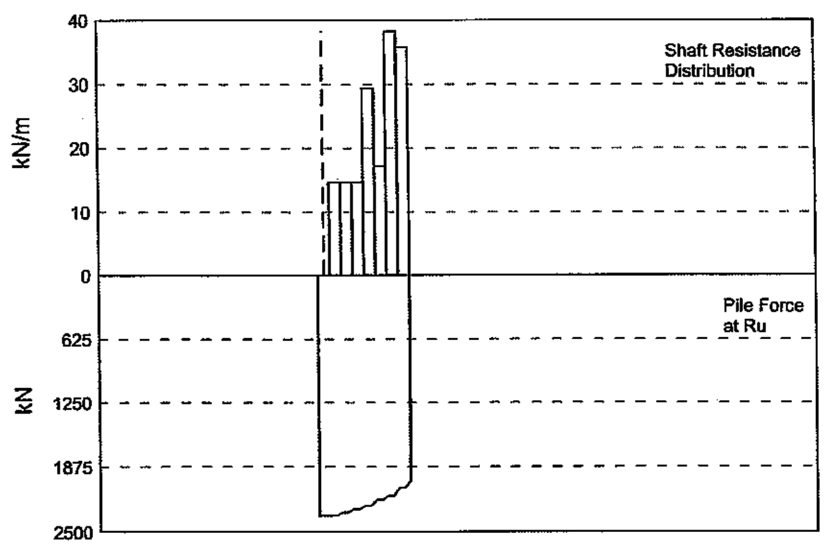
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10
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625
1250
1875
2500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

[redacted]ile: ET# 112, [redacted]
October 24, 2008; Blow: 22
[redacted]

Test: 24-Oct-2008 12:39:
CAPWAP(R) 2006-2
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2342.3; along Shaft 337.3; at Toe 2005.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2342.3					
1	2.0	0.7	0.0	2342.3	0.0	0.00	0.00	0.000	7.245
2	4.1	2.7	30.1	2312.2	30.1	14.68	12.04	1.101	7.244
3	6.1	4.8	30.1	2282.1	60.2	14.68	12.04	1.101	7.244
4	8.2	6.8	30.1	2252.0	90.3	14.68	12.04	1.101	7.244
5	10.2	8.8	60.2	2191.8	150.5	29.37	24.07	1.101	7.244
6	12.3	10.9	35.1	2156.7	185.6	17.12	14.03	1.101	7.244
7	14.4	13.0	78.4	2078.3	264.0	38.24	31.35	1.101	7.077
8	16.4	15.0	73.3	2005.0	337.3	35.76	29.31	1.101	5.688
Avg. Shaft			42.2			22.49	18.43	1.101	6.867
Toe			2005.0				20863.68	0.238	4.990

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.579	0.745
Unloading Quake	(% of loading quake)	40	70
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	3	
Soil Plug Weight	(kN)		0.07

CAPWAP match quality	= 1.38	(Wave Up Match) ; RSA = 0	
Observed: final set	= 1.000 mm;	blow count	= 1000 b/m
Computed: final set	= 0.100 mm;	blow count	= 9999 b/m
max. Top Comp. Stress	= 206.7 MPa	(T= 20.8 ms, max= 1.038 x Top)	
max. Comp. Stress	= 214.6 MPa	(Z= 4.1 m, T= 21.4 ms)	
max. Tens. Stress	= -26.58 MPa	(Z= 14.4 m, T= 36.2 ms)	
max. Energy (EMX)	= 28.87 kJ;	max. Measured Top Displ. (DMX)=14.68 mm	

Analysis: 13-Jan-2009

Test: 24-Oct-2008 12:39:
CAPWAP(R) 2006-2
OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3280.8	-78.9	206.7	-4.97	28.87	4.9	13.925
2	2.0	3316.8	-133.2	209.0	-8.39	28.64	4.8	13.553
3	3.1	3360.2	-185.7	211.7	-11.70	28.39	4.8	13.170
4	4.1	3405.0	-217.6	214.6	-13.71	28.15	4.7	12.774
5	5.1	3275.1	-254.9	206.4	-16.06	26.48	4.6	12.356
6	6.1	3319.4	-319.0	209.2	-20.10	26.11	4.6	11.880
7	7.2	3202.5	-360.8	201.8	-22.73	24.39	4.5	11.356
8	8.2	3272.1	-397.5	206.2	-25.05	23.88	4.4	10.796
9	9.2	3180.2	-388.7	200.4	-24.49	22.28	4.3	10.253
10	10.2	3235.0	-391.7	203.8	-24.68	21.78	4.2	9.685
11	11.3	2986.5	-365.2	188.2	-23.01	19.22	4.1	9.084
12	12.3	3066.4	-413.7	193.2	-26.07	18.52	4.0	8.432
13	13.3	2962.0	-419.3	186.6	-26.42	16.82	3.9	7.755
14	14.4	2986.3	-421.8	188.2	-26.58	16.08	3.8	7.079
15	15.4	2732.6	-360.7	172.2	-22.73	13.65	3.7	6.432
16	16.4	3027.3	-359.1	190.8	-22.63	12.73	3.4	5.782
Absolute	4.1			214.6			(T =	21.4 ms)
	14.4				-26.58		(T =	36.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3933.6	3671.1	3408.5	3145.9	2883.3	2620.7	2358.2	2095.6	1833.0	1570.4
RX	3933.6	3671.1	3408.5	3167.2	2969.9	2784.4	2649.5	2521.3	2431.8	2353.0
RU	3957.4	3697.2	3437.0	3176.8	2916.6	2656.4	2396.2	2136.0	1875.8	1615.6

RAU = 1979.0 (kN); RA2 = 2429.0 (kN)

Current CAPWAP Ru = 2342.3 (kN); Corresponding J(RP)= 0.61; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.15	20.61	3298.7	3260.7	3260.7	14.678	-0.881	1.000	29.7	3784.6

PILE PROFILE AND PILE MODEL

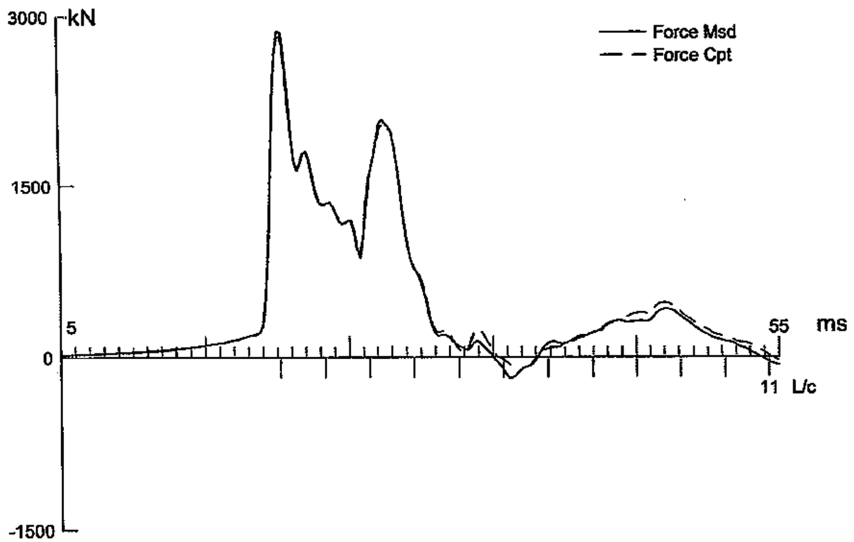
Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206874.8	77.300	1.220
16.40	158.70	206874.8	77.300	1.220

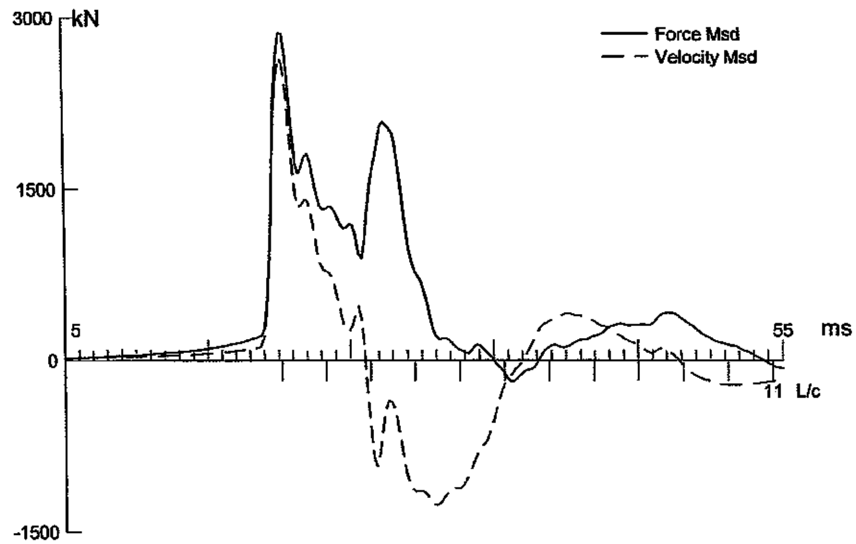
Toe Area 0.096 m²

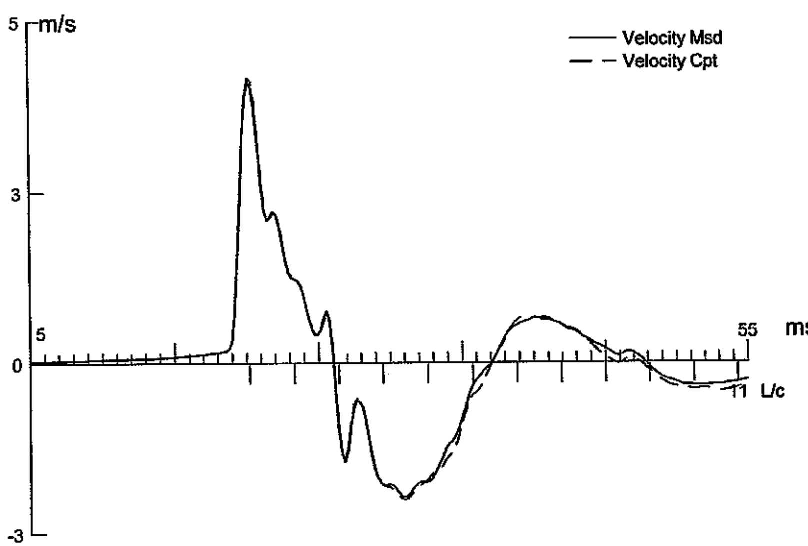
Top Segment Length 1.02 m, Top Impedance 640.86 kN/m/s

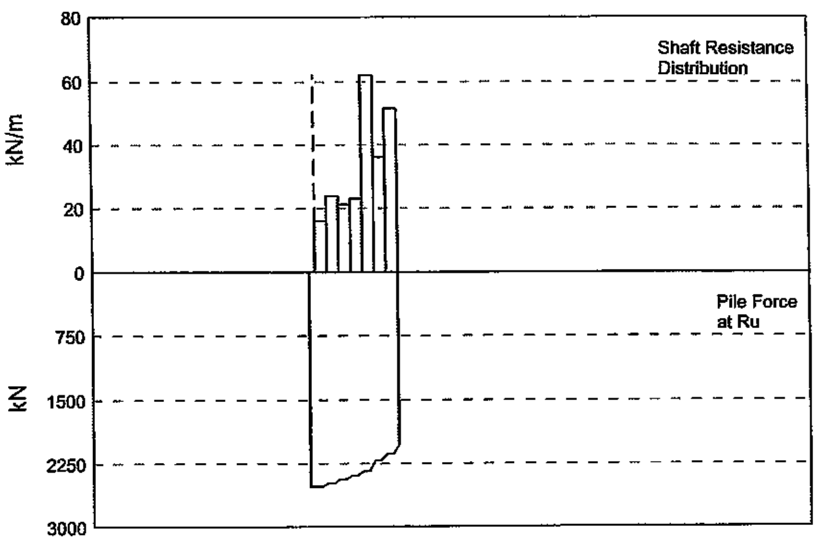
Analysis: 13-Jan-2009

Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 5123.0 m/s, 2L/c 6.4 ms


3000 kN
1500
0
-1500
5
55 ms
11 L/c
Force Msd
Force Cpt


3000 kN
1500
0
-1500
5
55 ms
11 L/c
Force Msd
Velocity Msd


5 m/s
3
0
-3
5
55 ms
11 L/c
Velocity Msd
Velocity Cpt


80
60
40
20
0
750
1500
2250
3000
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]

[redacted]Pile: ET# 320,[redacted]
November 14, 2008; Blow: 5
[redacted]

Test: 14-Nov-2008 11:45:
CAPWAP(R) 2006-2
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2519.6; along Shaft 494.3; at Toe 2025.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2519.6				
1	3.2	1.7	33.8	2485.8	33.8	20.34	16.67	0.682
2	5.3	3.8	50.5	2435.3	84.3	23.88	19.57	0.682
3	7.4	5.9	44.7	2390.6	129.0	21.14	17.33	0.682
4	9.5	8.0	48.4	2342.2	177.4	22.89	18.76	0.682
5	11.6	10.1	131.3	2210.9	308.7	62.09	50.89	0.682
6	13.7	12.2	76.3	2134.6	385.0	36.08	29.57	0.682
7	15.9	14.3	109.3	2025.3	494.3	51.69	42.37	0.682
Avg. Shaft			70.6			34.45	28.23	0.682
Toe			2025.3				21074.92	0.794

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.038	3.826
Case Damping Factor		0.526	2.511
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	100	109
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			0.342

CAPWAP match quality = 1.74 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.816 mm; blow count = 1226 b/m

max. Top Comp. Stress = 179.0 MPa (T= 20.7 ms, max= 1.130 x Top)
max. Comp. Stress = 202.3 MPa (Z= 15.9 m, T= 23.7 ms)
max. Tens. Stress = -21.47 MPa (Z= 11.6 m, T= 35.5 ms)
max. Energy (EMX) = 23.44 kJ; max. Measured Top Displ. (DMX)=13.35 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.1	2840.1	-153.6	179.0	-9.68	23.44	4.2	12.933
2	2.1	2873.8	-208.1	181.1	-13.11	23.08	4.1	12.473
3	3.2	2904.9	-246.4	183.0	-15.53	22.72	4.1	11.986
4	4.2	2825.8	-232.4	178.1	-14.65	21.42	4.0	11.524
5	5.3	2861.7	-267.3	180.3	-16.84	20.99	3.9	11.012
6	6.3	2721.8	-257.4	171.5	-16.22	19.16	3.9	10.417
7	7.4	2753.0	-297.6	173.5	-18.76	18.53	3.8	9.787
8	8.5	2648.8	-268.5	166.9	-16.92	16.89	3.7	9.157
9	9.5	2703.5	-318.6	170.4	-20.07	16.18	3.6	8.493
10	10.6	2653.9	-309.6	167.2	-19.51	14.37	3.5	7.745
11	11.6	2708.5	-340.8	170.7	-21.47	13.29	3.3	6.913
12	12.7	2590.8	-198.1	163.3	-12.48	10.32	3.4	6.109
13	13.7	2511.0	-236.1	158.2	-14.88	9.47	3.6	5.368
14	14.8	2884.2	-175.7	181.7	-11.07	7.76	3.4	4.627
15	15.9	3210.4	-198.6	202.3	-12.51	7.01	2.5	3.889
Absolute	15.9			202.3			(T =	23.7 ms)
	11.6				-21.47		(T =	35.5 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3892.7	3722.3	3551.9	3381.4	3211.0	3040.6	2870.2	2699.8	2529.4	2359.0
RX	4056.9	3902.5	3763.2	3631.6	3499.9	3368.3	3236.6	3106.3	3013.2	2920.0
RU	3908.4	3739.5	3570.7	3401.9	3233.0	3064.2	2895.3	2726.5	2557.6	2388.8

RAU = 1562.3 (kN); RA2 = 2624.2 (kN)

Current CAPWAP Ru = 2519.6 (kN); RMX requires J > 0.9;

Check with PDA-W; RA2 may be a better Case Method

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
4.20	20.44	2686.8	2910.0	2927.7	13.349	0.927	1.000	24.2	3369.7

PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206632.6	77.300	1.220
15.86	158.70	206632.6	77.300	1.220

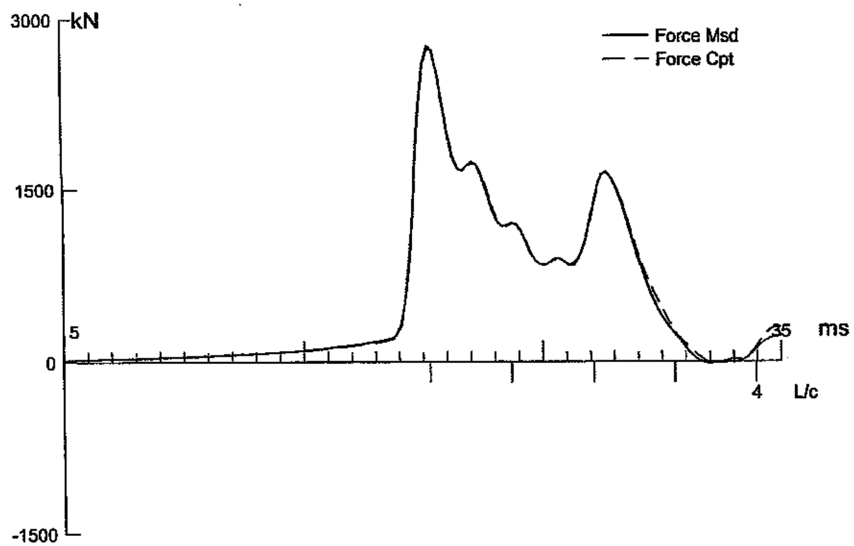
Toe Area 0.096 m^2

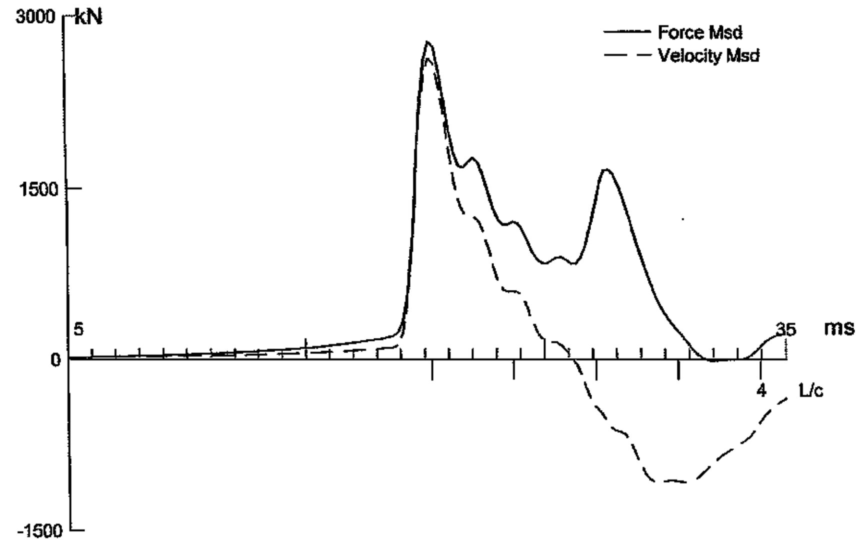
Top Segment Length 1.06 m, Top Impedance 640.48 kN/m/s

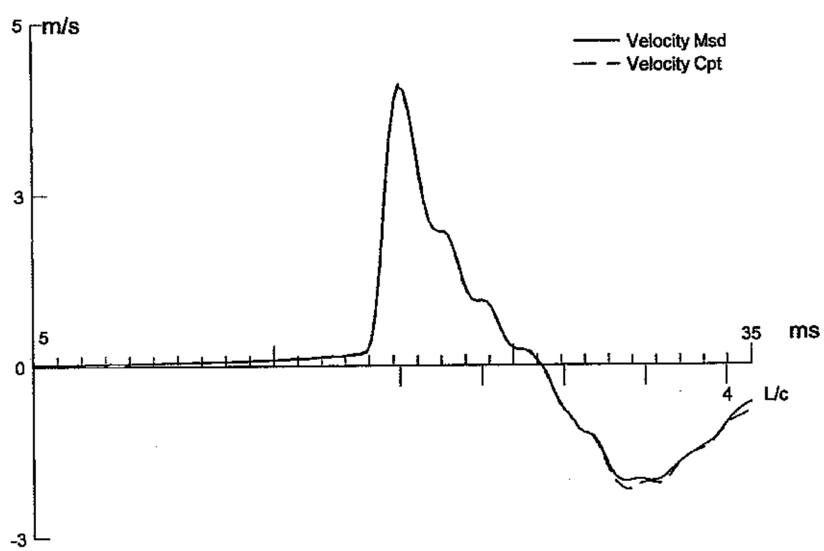
Pile Damping 1.0 %, Time Incr 0.207 ms, Wave Speed 5120.0 m/s, 2L/c 6.2 ms

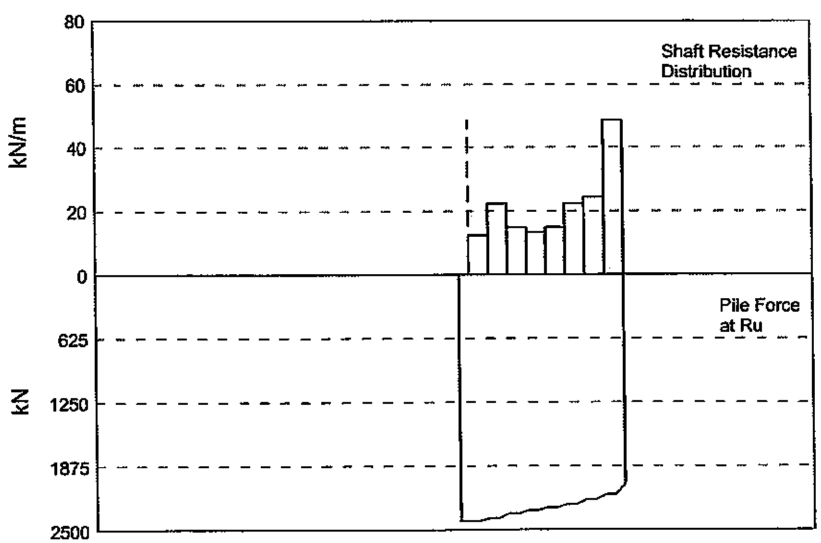
[redacted]#ET404; November 5, 2008; Blow: 6 (Test: 05-Nov-2008 16:13:)

13-Jan-2009
CAPWAP(R) 2006-2









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[redacted]Pile #ET404 Test: 05-Nov-2008 16:13:
November 5, 2008; Blow: 6 CAPWAP(R) 2006-2
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2410.4; along Shaft 355.4; at Toe 2055.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2410.4					
1	3.1	1.7	25.4	2385.0	25.4	14.97	12.27	1.333	2.941
2	5.1	3.8	45.8	2339.2	71.2	22.26	18.24	1.333	2.940
3	7.2	5.8	30.5	2308.7	101.7	14.82	12.15	1.333	2.940
4	9.3	7.9	27.4	2281.3	129.1	13.32	10.91	1.333	2.940
5	11.3	9.9	30.5	2250.8	159.6	14.82	12.15	1.333	2.940
6	13.4	12.0	45.8	2205.0	205.4	22.26	18.24	1.333	2.940
7	15.4	14.0	50.0	2155.0	255.4	24.30	19.92	1.333	2.940
8	17.5	16.1	100.0	2055.0	355.4	48.60	39.84	1.333	2.031
Avg. Shaft			44.4			22.07	18.09	1.333	2.684
Toe			2055.0				21383.98	0.424	2.927

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.740	1.360
Unloading Quake	(% of loading quake)	41	30
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			1.003
Soil Support Dashpot		0.680	0.000
Soil Support Weight	(kN)	12.80	0.00

CAPWAP match quality = 2.38 (Wave Up Match) ; RSA = 0
Observed: final set = 2.000 mm; blow count = 500 b/m
Computed: final set = 1.942 mm; blow count = 515 b/m

max. Top Comp. Stress = 174.2 MPa (T= 20.7 ms, max= 1.073 x Top)
max. Comp. Stress = 186.9 MPa (Z= 17.5 m, T= 24.3 ms)
max. Tens. Stress = -23.98 MPa (Z= 13.4 m, T= 34.8 ms)
max. Energy (EMX) = 21.34 kJ; max. Measured Top Displ. (DMX)=12.29 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2764.9	-100.2	174.2	-6.31	21.34	4.1	12.010
2	2.1	2803.4	-100.6	176.6	-6.34	21.18	4.0	11.694
3	3.1	2853.9	-105.8	179.8	-6.67	21.01	3.9	11.365
4	4.1	2757.3	-135.6	173.7	-8.54	19.79	3.9	11.049
5	5.1	2808.3	-179.1	177.0	-11.28	19.60	3.8	10.699
6	6.2	2585.8	-186.4	162.9	-11.74	17.66	3.7	10.307
7	7.2	2624.2	-238.2	165.4	-15.01	17.31	3.6	9.852
8	8.2	2488.3	-290.2	156.8	-18.29	15.85	3.6	9.338
9	9.3	2525.6	-354.5	159.1	-22.34	15.42	3.5	8.821
10	10.3	2415.4	-360.5	152.2	-22.72	14.13	3.5	8.299
11	11.3	2462.0	-376.5	155.1	-23.72	13.59	3.4	7.708
12	12.3	2357.8	-358.5	148.6	-22.59	12.09	3.3	7.050
13	13.4	2415.1	-380.5	152.2	-23.98	11.25	3.2	6.327
14	14.4	2421.5	-341.8	152.6	-21.54	9.43	3.1	5.585
15	15.4	2471.9	-353.8	155.8	-22.29	8.64	2.9	4.874
16	16.5	2679.3	-304.3	168.8	-19.18	7.19	2.5	4.183
17	17.5	2966.5	-308.8	186.9	-19.46	6.49	2.0	3.501
Absolute	17.5			186.9			(T =	24.3 ms)
	13.4				-23.98		(T =	34.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3712.7	3533.7	3354.7	3175.7	2996.7	2817.7	2638.7	2459.7	2280.7	2101.7
RX	3712.7	3533.7	3363.8	3206.1	3048.4	2890.8	2737.1	2612.9	2488.7	2396.1
RU	3785.4	3613.6	3441.9	3270.1	3098.4	2926.6	2754.9	2583.1	2411.4	2239.6

RAU = 490.0 (kN); RA2 = 2191.6 (kN)

Current CAPWAP Ru = 2410.4 (kN); Corresponding J(RP)= 0.73; J(RX) = 0.88

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.18	20.50	2678.5	2824.3	2830.2	12.292	2.705	2.000	22.0	3076.0

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	158.70	206632.6	77.300	1.220
17.49	158.70	206632.6	77.300	1.220

Toe Area 0.096 m^2

Top Segment Length 1.03 m, Top Impedance 640.48 kN/m/s

Pile Damping 1.0 %, Time Incr 0.201 ms, Wave Speed 5120.0 m/s, 2L/c 6.8 ms

Site 10

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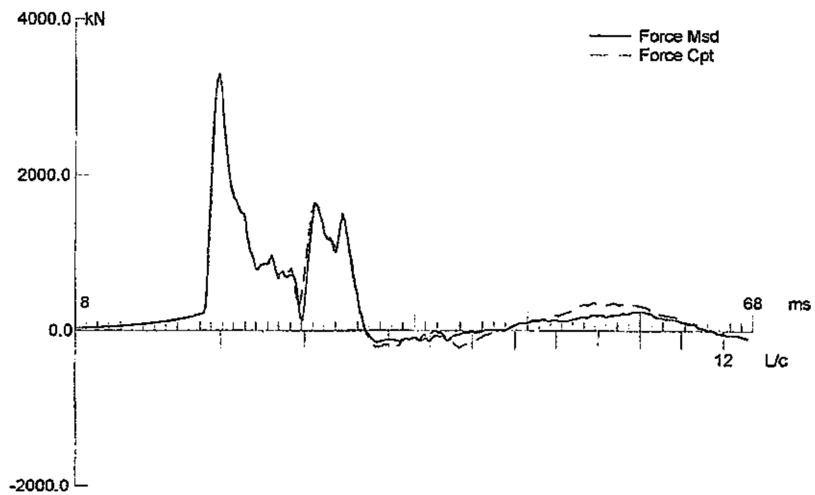
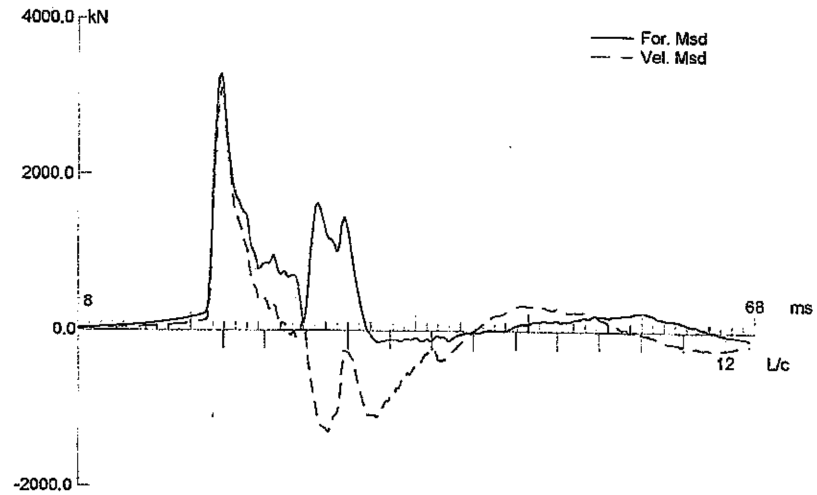
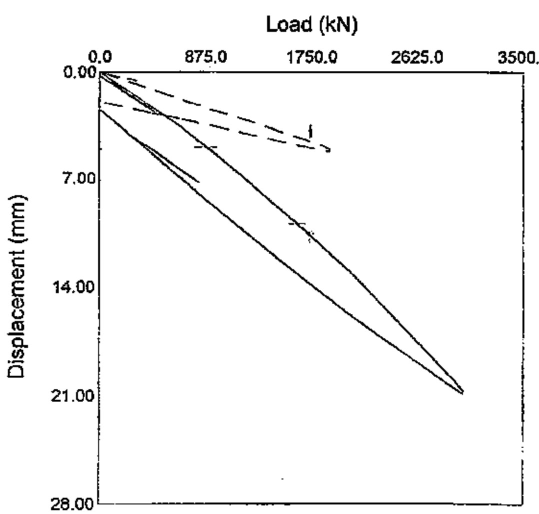
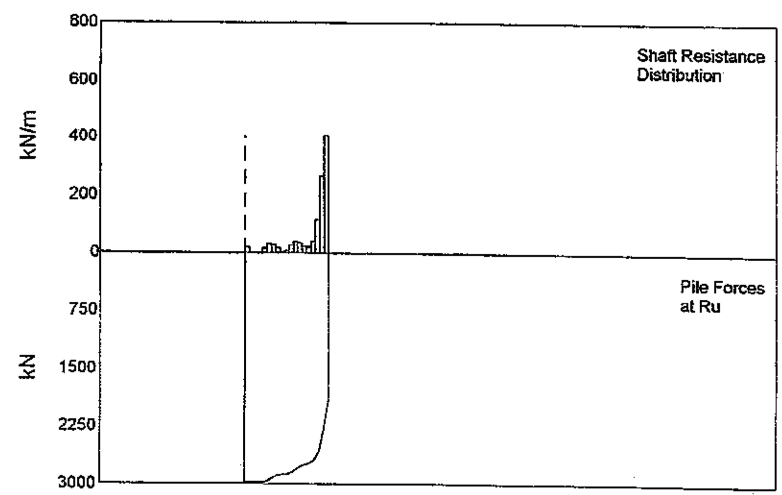
15-Aug-2006
CAPWAP® Ver. 2000-1









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CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 3000.9; along Shaft 1100.3; at Toe 1900.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				3000.9					
1	1.0	1.0	20.5	2980.4	20.5	20.50	17.09	0.300	2.500
2	2.0	2.0	0.0	2980.4	20.5	0.00	0.00	0.000	2.500
3	3.0	3.0	0.0	2980.4	20.5	0.00	0.00	0.000	2.500
4	4.0	4.0	0.0	2980.4	20.5	0.00	0.00	0.000	2.500
5	5.0	5.0	16.9	2963.5	37.4	16.90	14.09	0.300	2.500
6	6.0	6.0	31.6	2931.9	69.0	31.61	26.34	0.300	2.500
7	7.0	7.0	30.7	2901.2	99.7	30.71	25.59	0.300	2.500
8	8.0	8.0	17.8	2883.4	117.5	17.80	14.84	0.300	2.500
9	9.0	9.0	4.9	2878.5	122.4	4.90	4.08	0.300	2.500
10	10.0	10.0	8.9	2869.6	131.3	8.90	7.42	0.300	2.500
11	11.0	11.0	27.6	2842.0	158.9	27.61	23.00	0.300	2.500
12	12.0	12.0	40.0	2802.0	198.9	40.01	33.34	0.300	2.500
13	13.0	13.0	35.1	2766.9	234.0	35.11	29.26	0.300	2.500
14	14.0	14.0	22.7	2744.2	256.8	22.70	18.92	0.300	2.500
15	15.0	15.0	20.0	2724.2	276.8	20.00	16.67	0.300	2.500
16	16.0	16.0	40.0	2684.2	316.8	40.01	33.34	0.300	2.500
17	17.0	17.0	114.8	2569.3	431.6	114.82	95.69	0.300	2.500
18	18.0	18.0	263.9	2305.5	695.4	263.86	219.88	0.300	2.500
19	19.0	19.0	404.9	1900.6	1100.3	404.89	337.40	0.300	2.500
Avg. Skin			57.9			57.91	48.26	0.300	2.500
Toe			1900.6				113808.44	0.200	5.000

Soil Model Parameters/Extensions		Skin	Toe	
Case Damping Factor		0.489	0.564	Smith Type
Unloading Quake	(% of loading quake)	98	67	
Reloading Level	(% of Ru)	100	100	
Unloading Level	(% of Ru)	26		
Soil Plug Weight	(kN)		0.57	

CAPWAP match quality: 3.51 (Wave Up Match)
Observed: final set = 2.137 mm; blow count = 468 b/m
Computed: final set = 1.204 mm; blow count = 830 b/m

[redacted]

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3298.9	-264.7	197.539	-15.852	24.56	4.6	12.509
2	2.0	3253.4	-303.2	194.816	-18.153	24.09	4.6	12.337
3	3.0	3261.9	-344.4	195.321	-20.622	24.02	4.6	12.147
4	4.0	3281.3	-344.1	196.485	-20.604	23.95	4.6	11.957
5	5.0	3310.3	-385.7	198.224	-23.098	23.84	4.5	11.722
6	6.0	3299.8	-391.1	197.592	-23.417	23.42	4.5	11.497
7	7.0	3246.9	-355.9	194.427	-21.311	22.69	4.4	11.202
8	8.0	3188.9	-354.7	190.953	-21.241	21.93	4.4	10.865
9	9.0	3163.9	-387.9	189.455	-23.228	21.45	4.4	10.558
10	10.0	3180.7	-414.8	190.460	-24.836	21.17	4.3	10.232
11	11.0	3197.3	-416.1	191.457	-24.915	20.86	4.3	9.922
12	12.0	3170.8	-457.0	189.870	-27.366	20.18	4.2	9.559
13	13.0	3110.5	-407.7	186.255	-24.413	19.26	4.2	9.138
14	14.0	3062.6	-460.4	183.389	-27.567	18.32	4.1	8.628
15	15.0	3066.7	-479.7	183.636	-28.723	17.36	4.0	8.028
16	16.0	3132.6	-486.8	187.583	-29.148	16.33	3.9	7.370
17	17.0	2966.1	-503.1	177.609	-30.126	15.01	4.1	6.667
18	18.0	2496.9	-500.9	149.515	-29.995	12.92	4.4	5.953
19	19.0	2583.8	-427.2	154.719	-25.580	7.48	4.0	5.287
Absolute	5.0			198.224			(T =	21.9 ms)
	17.0				-30.126		(T =	38.8 ms)

[redacted]

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RS1	3196.9	2867.9	2538.9	2209.8	1880.8	1551.8	1222.8	893.8	564.7	235.7
RMX	3268.0	3062.5	3030.1	2998.0	2966.0	2934.0	2901.9	2877.1	2857.7	2838.3
RSU	3339.3	3024.5	2709.7	2394.9	2080.1	1765.3	1450.5	1135.7	820.9	506.1

RAU= 2142.9 (kN); RA2= 2947.9 (kN)

Current CAPWAP Ru= 3000.9 (kN); Corresponding J(Rs)= 0.06; J(Rx)=0.29

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	RLT
m/s	m/s	kN	kN	kN	mm	mm	kJ	kN
4.68	0.00	3158.0	3329.2	3329.2	12.521	2.139	24.7	3371.3

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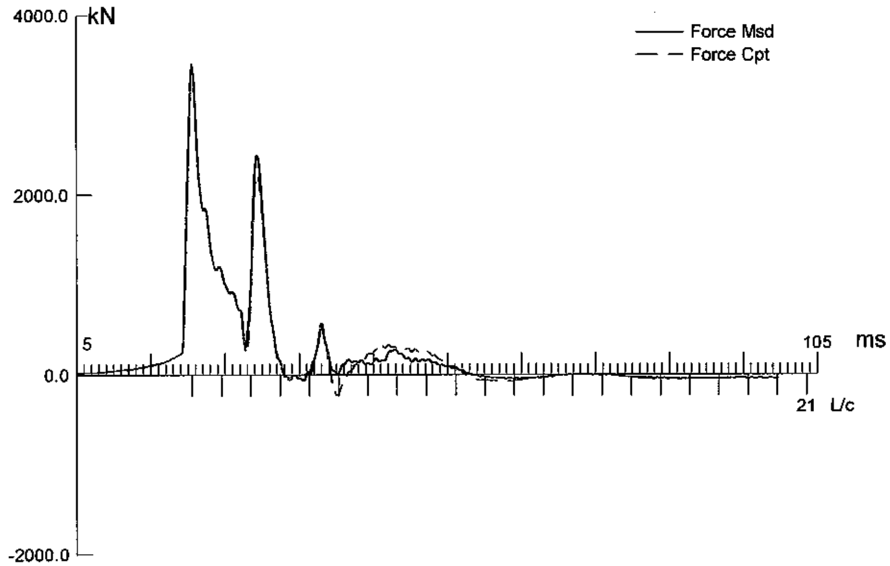
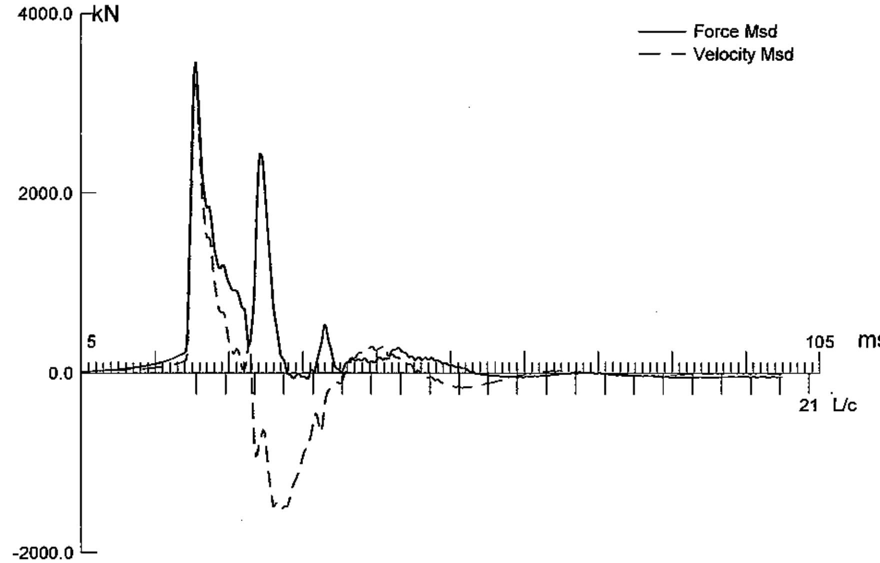
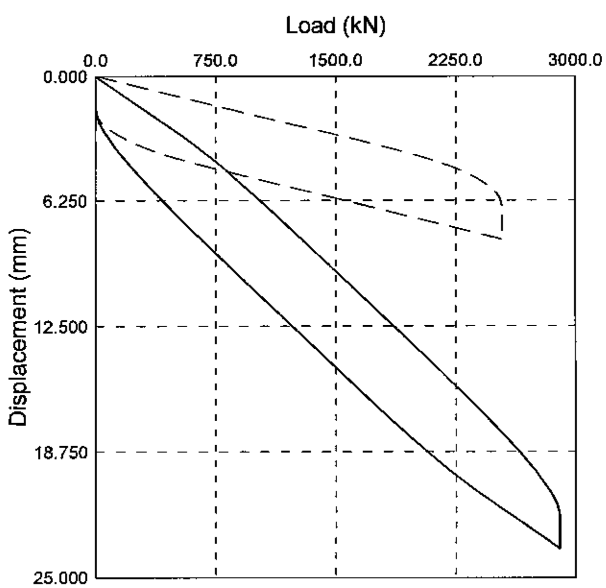
Site 11

[redacted]-1; [redacted]: 508 (Test: 10-Sep-2007 12:48:)

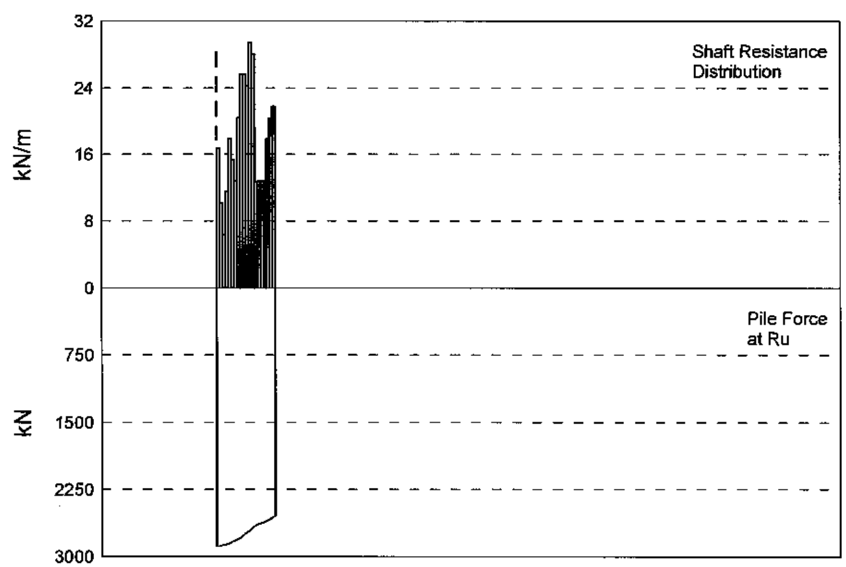
12-Sep-2007
CAPWAP(R) 2006







Ru =	2900.7 kN
Rs =	359.9 kN
Rb =	2540.8 kN
Dy =	21.9 mm
Dx =	23.6 mm



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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2900.7; along Shaft 359.9; at Toe 2540.8 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
2	2.0	2.0	10.3	2873.5	27.2	10.17	8.34	0.500	2.500
3	3.0	3.0	6.5	2867.0	33.7	6.42	5.26	0.500	2.500
4	4.0	4.1	11.7	2855.3	45.4	11.56	9.47	0.500	2.500
5	5.1	5.1	18.1	2837.2	63.5	17.88	14.65	0.500	2.500
6	6.1	6.1	15.5	2821.7	79.0	15.31	12.55	0.500	2.500
7	7.1	7.1	12.9	2808.8	91.9	12.74	10.44	0.500	2.500
8	8.1	8.1	20.7	2788.1	112.6	20.44	16.76	0.500	2.500
9	9.1	9.1	26.0	2762.1	138.6	25.68	21.05	0.500	2.500
10	10.1	10.1	26.0	2736.1	164.6	25.68	21.05	0.500	2.500
11	11.1	11.1	24.6	2711.5	189.2	24.30	19.91	0.500	2.500
12	12.2	12.2	29.8	2681.7	219.0	29.43	24.12	0.500	2.500
13	13.2	13.2	28.4	2653.3	247.4	28.05	22.99	0.500	2.500
14	14.2	14.2	12.9	2640.4	260.3	12.74	10.44	0.500	2.500
15	15.2	15.2	12.9	2627.5	273.2	12.74	10.44	0.500	2.500
16	16.2	16.2	12.9	2614.6	286.1	12.74	10.44	0.500	2.500
17	17.2	17.2	12.9	2601.7	299.0	12.74	10.44	0.500	2.500
18	18.2	18.2	18.1	2583.6	317.1	17.88	14.65	0.500	2.500
19	19.2	19.2	20.7	2562.9	337.8	20.44	16.76	0.500	2.500
20	20.2	20.2	22.1	2540.8	359.9	21.83	17.89	0.500	1.569
Avg. Shaft			18.0			17.77	14.57	0.500	2.443
Toe			2540.8				145521.19	0.500	5.000

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.255	1.802
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	10	
Soil Plug Weight	(kN)		0.19

CAPWAP match quality = 2.89 (Wave Up Match) ; RSA = 0
Observed: final set = 1.700 mm; blow count = 588 b/m
Computed: final set = 0.440 mm; blow count = 2274 b/m

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3476.0	-229.3	199.1	-13.13	28.11	4.6	13.896
2	2.0	3434.7	-234.9	196.7	-13.45	27.56	4.6	13.716
3	3.0	3420.3	-217.2	195.9	-12.44	27.18	4.6	13.525
4	4.0	3422.4	-212.5	196.0	-12.17	26.92	4.5	13.313
5	5.1	3408.6	-241.2	195.2	-13.82	26.47	4.5	13.043
6	6.1	3374.1	-326.4	193.2	-18.69	25.80	4.5	12.739
7	7.1	3353.3	-364.0	192.1	-20.85	25.23	4.4	12.436
8	8.1	3348.1	-374.8	191.8	-21.47	24.72	4.4	12.133
9	9.1	3319.7	-439.4	190.1	-25.16	23.97	4.3	11.792
10	10.1	3276.4	-526.7	187.7	-30.16	23.04	4.2	11.385
11	11.1	3233.1	-572.0	185.2	-32.76	22.05	4.2	10.927
12	12.2	3192.1	-577.1	182.8	-33.05	21.12	4.1	10.477
13	13.2	3128.6	-622.2	179.2	-35.64	20.09	4.1	10.007
14	14.2	3062.1	-641.8	175.4	-36.76	18.96	4.1	9.493
15	15.2	3043.2	-681.2	174.3	-39.01	18.03	4.0	8.870
16	16.2	3028.1	-714.6	173.4	-40.93	16.88	4.0	8.160
17	17.2	2987.1	-751.1	171.1	-43.02	15.58	4.1	7.397
18	18.2	2813.4	-768.3	161.1	-44.00	14.28	4.6	6.618
19	19.2	3257.8	-768.3	186.6	-44.01	13.00	4.4	5.860
20	20.2	3748.3	-772.7	214.7	-44.26	13.15	3.5	5.122
Absolute	20.2			214.7			(T =	25.1 ms)
	20.2				-44.26		(T =	37.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4459.6	4206.8	3953.9	3701.1	3448.2	3195.4	2942.6	2689.7	2436.9	2184.0
RX	4539.1	4379.4	4219.7	4059.9	3900.2	3740.5	3627.3	3515.4	3403.6	3291.7
RU	4476.6	4225.4	3974.3	3723.2	3472.0	3220.9	2969.7	2718.6	2467.5	2216.3

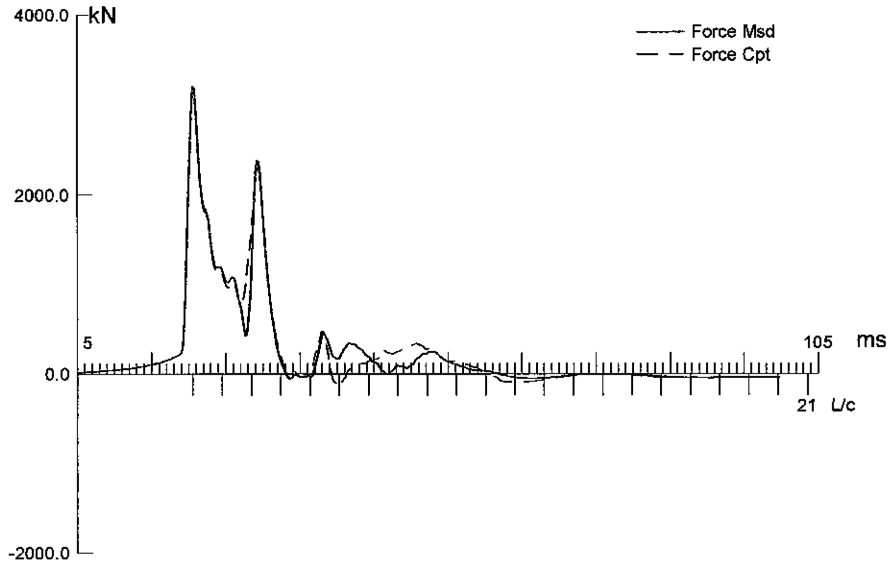
RAU = 1436.3 (kN); RA2 = 2916.1 (kN)

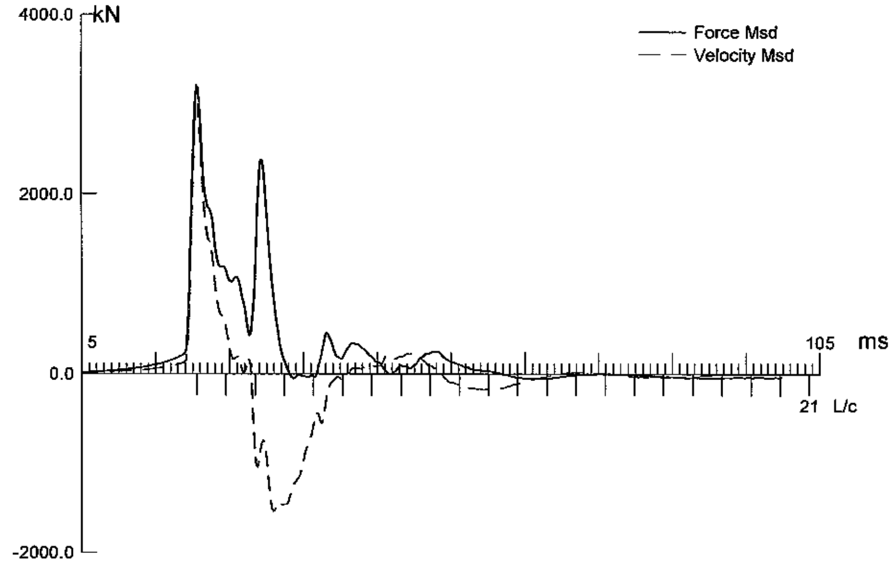
Current CAPWAP Ru = 2900.7 (kN); RMX requires J > 0.9;

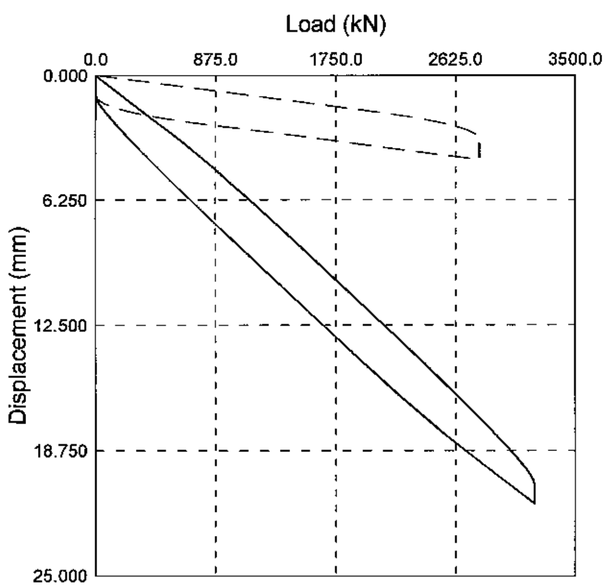
Check with PDA-W; RA2 may be a better Case Method

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	m/s	kN	kN	kN	mm	mm	mm	kJ	kN
4.84	0.00	3413.4	3574.6	3574.6	14.012	1.705	1.700	28.5	4065.2

Peak Velocity Time = 20.75 ms.

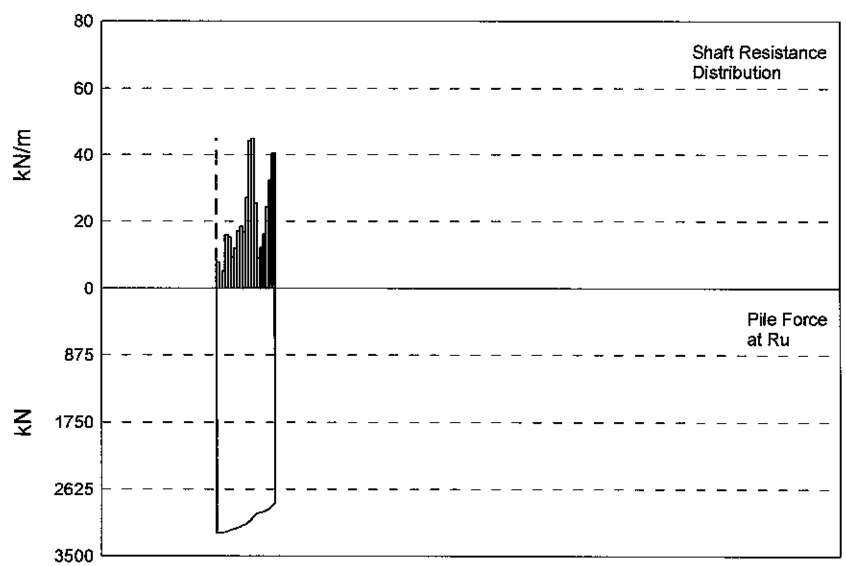






Pile Top
Bottom

Ru =	3200.7 kN
Rs =	400.0 kN
Rb =	2800.7 kN
Dy =	20.4 mm
Dx =	21.4 mm



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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3200.7; along Shaft 400.0; at Toe 2800.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
2	2.0	2.2	0.0	3192.8	7.9	0.00	0.00	0.000	2.500
3	3.0	3.2	5.3	3187.5	13.2	5.23	4.36	0.500	2.500
4	4.0	4.2	16.2	3171.3	29.4	16.00	13.33	0.500	2.500
5	5.1	5.2	15.5	3155.8	44.9	15.31	12.76	0.500	2.500
6	6.1	6.2	9.6	3146.2	54.5	9.48	7.90	0.500	2.500
7	7.1	7.2	12.1	3134.1	66.6	11.95	9.96	0.500	2.500
8	8.1	8.3	17.4	3116.7	84.0	17.19	14.32	0.500	2.500
9	9.1	9.3	18.7	3098.0	102.7	18.47	15.39	0.500	2.500
10	10.1	10.3	17.0	3081.0	119.7	16.79	13.99	0.500	2.500
11	11.1	11.3	27.5	3053.5	147.2	27.16	22.63	0.500	2.500
12	12.2	12.3	44.9	3008.6	192.1	44.35	36.95	0.500	2.500
13	13.2	13.3	45.5	2963.1	237.6	44.94	37.45	0.500	2.500
14	14.2	14.3	25.8	2937.3	263.4	25.48	21.23	0.500	2.500
15	15.2	15.3	9.2	2928.1	272.6	9.09	7.57	0.500	2.500
16	16.2	16.3	12.3	2915.8	284.9	12.15	10.12	0.500	2.500
17	17.2	17.4	16.5	2899.3	301.4	16.30	13.58	0.500	2.500
18	18.2	18.4	24.7	2874.6	326.1	24.40	20.33	0.500	2.500
19	19.2	19.4	32.8	2841.8	358.9	32.40	27.00	0.500	1.719
20	20.2	20.4	41.1	2800.7	400.0	40.59	33.83	0.500	1.015
Avg. Shaft			20.0			19.61	16.34	0.500	2.283
Toe			2800.7				160406.64	0.500	2.500

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.284	1.986
Unloading Quake	(% of loading quake)	30	51
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	1	

CAPWAP match quality = 5.44 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.237 mm; blow count = 4215 b/m

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3213.6	-135.2	184.1	-7.74	26.38	4.3	13.298
2	2.0	3200.5	-143.0	183.3	-8.19	26.04	4.3	13.034
3	3.0	3219.2	-196.4	184.4	-11.25	25.86	4.3	12.728
4	4.0	3223.3	-257.8	184.6	-14.77	25.50	4.2	12.379
5	5.1	3191.8	-307.4	182.8	-17.61	24.87	4.2	12.036
6	6.1	3166.9	-346.4	181.4	-19.84	24.28	4.2	11.699
7	7.1	3165.5	-385.9	181.3	-22.10	23.81	4.1	11.345
8	8.1	3159.6	-459.2	181.0	-26.30	23.23	4.1	10.927
9	9.1	3145.1	-515.8	180.1	-29.54	22.44	4.0	10.471
10	10.1	3139.0	-554.1	179.8	-31.74	21.62	4.0	9.989
11	11.1	3144.6	-601.6	180.1	-34.46	20.85	3.9	9.497
12	12.2	3113.3	-664.9	178.3	-38.08	19.82	3.8	8.969
13	13.2	3019.3	-691.4	172.9	-39.60	18.38	3.7	8.366
14	14.2	2915.7	-706.1	167.0	-40.44	16.79	3.7	7.700
15	15.2	2870.0	-717.9	164.4	-41.11	15.36	3.7	6.953
16	16.2	2970.2	-747.8	170.1	-42.83	14.12	3.6	6.155
17	17.2	3071.3	-768.3	175.9	-44.00	12.76	3.5	5.332
18	18.2	3479.6	-780.2	199.3	-44.69	11.24	3.0	4.458
19	19.2	3853.5	-788.1	220.7	-45.14	9.61	2.4	3.557
20	20.2	4074.4	-786.4	233.4	-45.04	9.18	1.8	2.710
Absolute	20.2			233.4			(T =	24.9 ms)
	19.2				-45.14		(T =	37.2 ms)

[redacted] Pile: [redacted]-5
Blow: 636

Test: 10-Sep-2007 13:22:
CAPWAP(R) 2006
OP: IW

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4563.4	4381.2	4199.1	4016.9	3834.7	3652.6	3470.4	3288.3	3106.1	2923.9
RX	4563.4	4404.0	4262.4	4120.7	3979.0	3837.4	3695.7	3585.9	3478.1	3370.3
RU	4682.7	4512.5	4342.3	4172.1	4001.9	3831.6	3661.4	3491.2	3321.0	3150.8

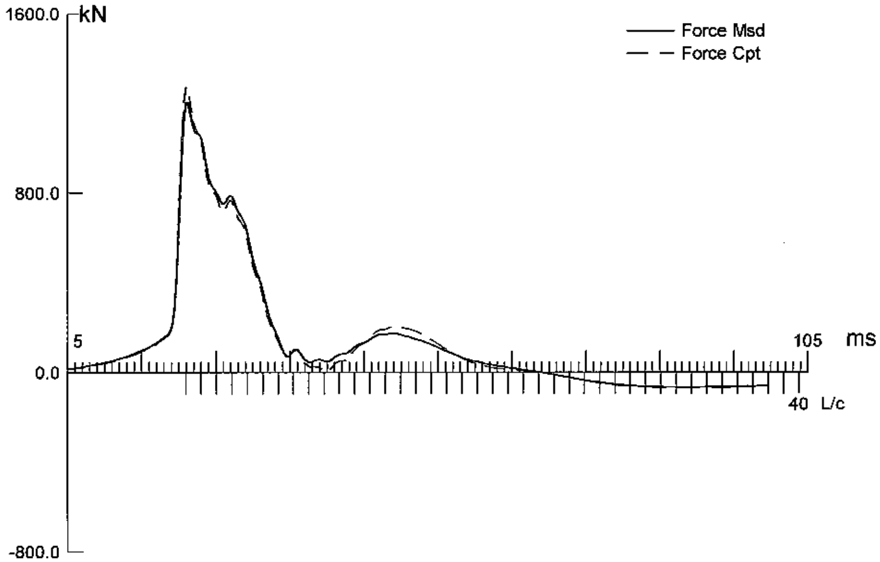
RAU = 2133.7 (kN); RA2 = 2955.6 (kN)

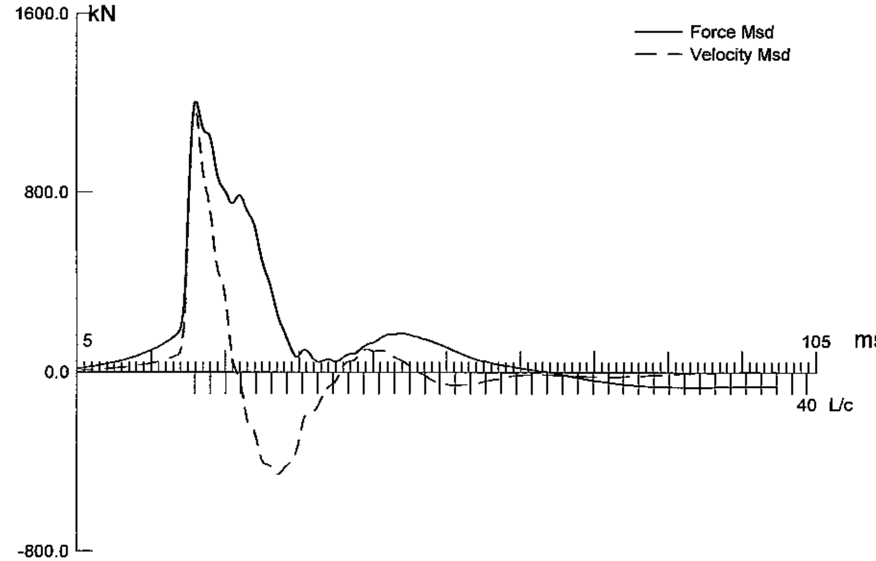
Current CAPWAP Ru = 3200.7 (kN); Corresponding J(RP)= 0.75;

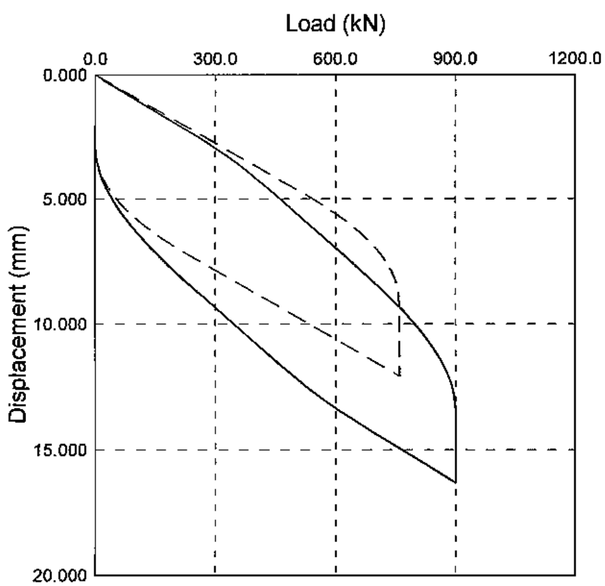
RMX requires higher damping; see PDA-W

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	m/s	kN	kN	kN	mm	mm	mm	kJ	kN
4.44	0.00	3127.8	3257.1	3257.1	13.344	1.205	1.000	26.6	3985.1

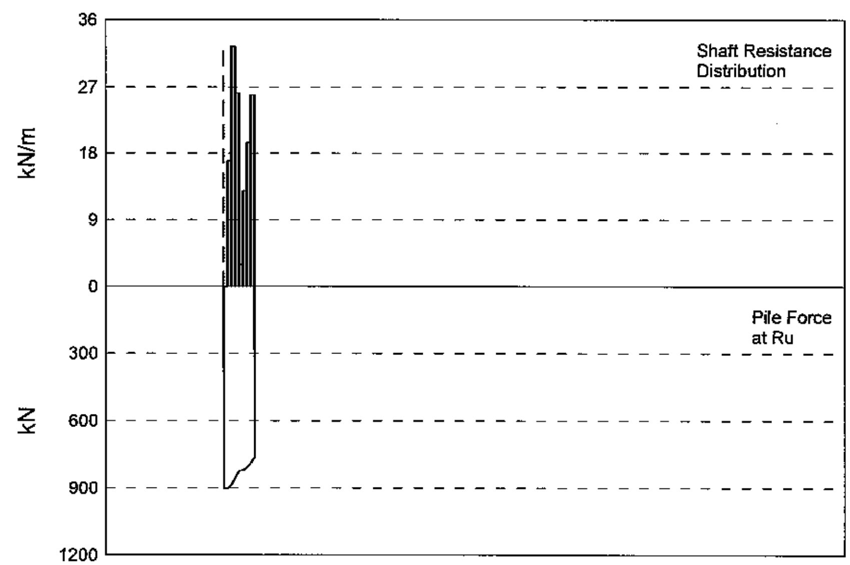
Peak Velocity Time = 20.75 ms.







Ru =	900.9	kN
Rs =	140.3	kN
Rb =	760.6	kN
Dy =	13.3	mm
Dx =	16.3	mm



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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 900.9; along Shaft 140.3; at Toe 760.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
2	2.0	1.8	17.4	883.5	17.4	16.98	14.27	0.500
3	3.1	2.8	33.3	850.2	50.7	32.49	27.30	0.500
4	4.1	3.8	26.9	823.3	77.6	26.24	22.05	0.500
5	5.1	4.8	3.0	820.3	80.6	2.93	2.46	0.500
6	6.1	5.8	13.2	807.1	93.8	12.88	10.82	0.500
7	7.2	6.9	19.9	787.2	113.7	19.41	16.31	0.500
8	8.2	7.9	26.6	760.6	140.3	25.95	21.81	0.500
Avg. Shaft			17.5			17.76	14.92	0.500
Toe			760.6				23453.59	0.500

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.500	7.000
Case Damping Factor		0.169	0.915
Unloading Quake	(% of loading quake)	40	58
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	71	
Soil Plug Weight	(kN)		0.23

CAPWAP match quality	=	2.92	(Wave Up Match) ; RSA = 0		
Observed: final set	=	3.000 mm;	blow count	=	333 b/m
Computed: final set	=	1.510 mm;	blow count	=	662 b/m

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	1290.5	-70.7	12.5	-0.69	10.50	2.7	11.351
2	2.0	1314.6	-71.0	12.8	-0.69	10.30	2.6	10.852
3	3.1	1295.2	-58.8	12.6	-0.57	9.79	2.6	10.344
4	4.1	1234.3	-35.5	12.0	-0.35	9.02	2.5	9.838
5	5.1	1198.9	-16.7	11.7	-0.16	8.38	2.5	9.324
6	6.1	1218.0	-14.6	11.8	-0.14	8.12	2.4	8.807
7	7.2	1210.0	-8.4	11.8	-0.08	7.73	2.4	8.302
8	8.2	1209.7	-10.6	11.8	-0.10	7.26	2.3	7.810
Absolute	2.0			12.8			(T =	22.0 ms)
	2.0				-0.69		(T =	89.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1464.1	1365.9	1267.8	1169.7	1071.6	973.5	875.4	777.3	679.2	581.0
RX	1471.5	1377.3	1283.2	1190.4	1129.3	1080.4	1034.7	1001.1	977.7	958.8
RU	1464.1	1365.9	1267.8	1169.7	1071.6	973.5	875.4	777.3	679.2	581.0

RAU = 876.1 (kN); RA2 = 1211.4 (kN)

Current CAPWAP Ru = 900.9 (kN); Corresponding J(RP)= 0.57;

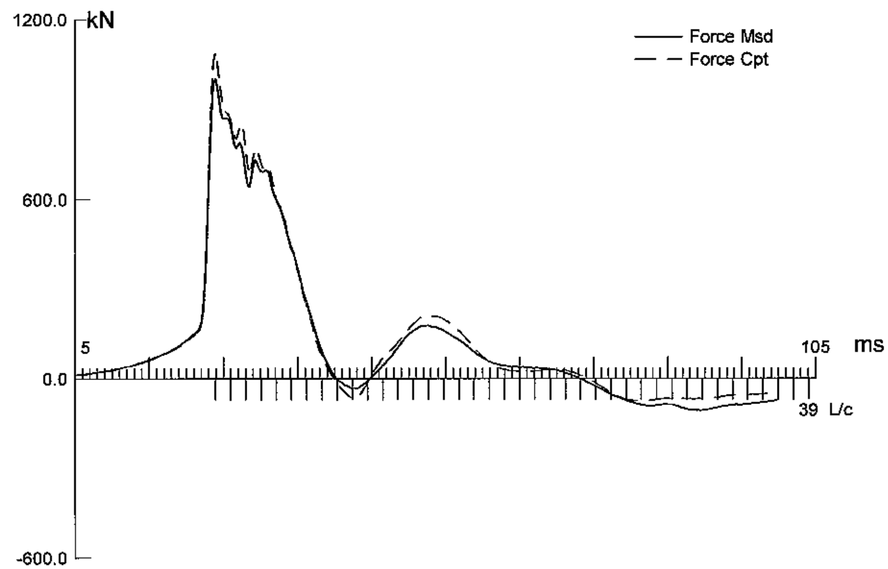
RMX requires higher damping; see PDA-W

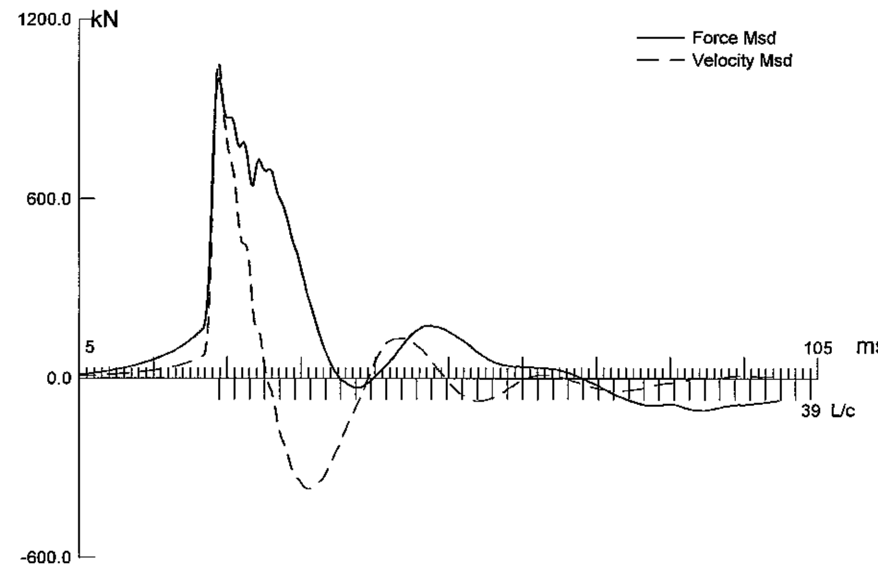
VMX m/s	VFN m/s	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
2.95	0.00	1225.5	1219.7	1219.7	11.678	3.105	3.000	10.7	1827.2

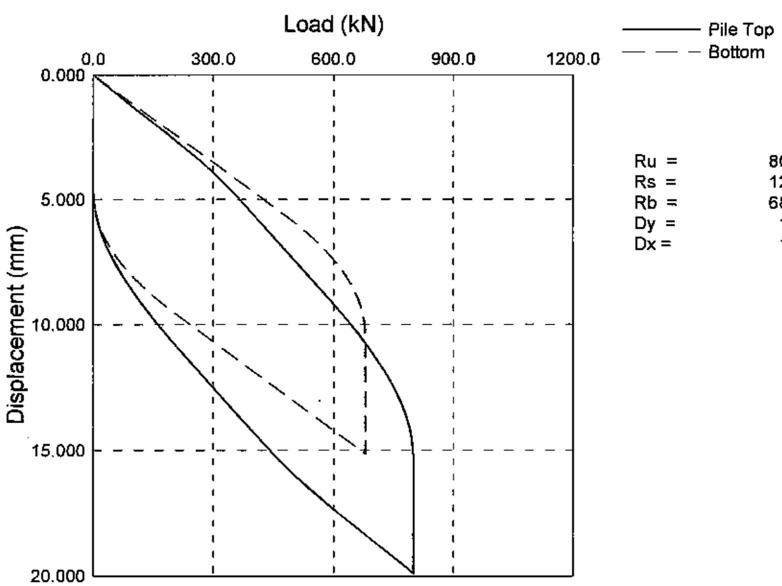
Peak Velocity Time = 21.22 ms.

[redacted] Pile: [redacted]48; [redacted]low: 23 (Test: 10-Sep-2007 14:44:)

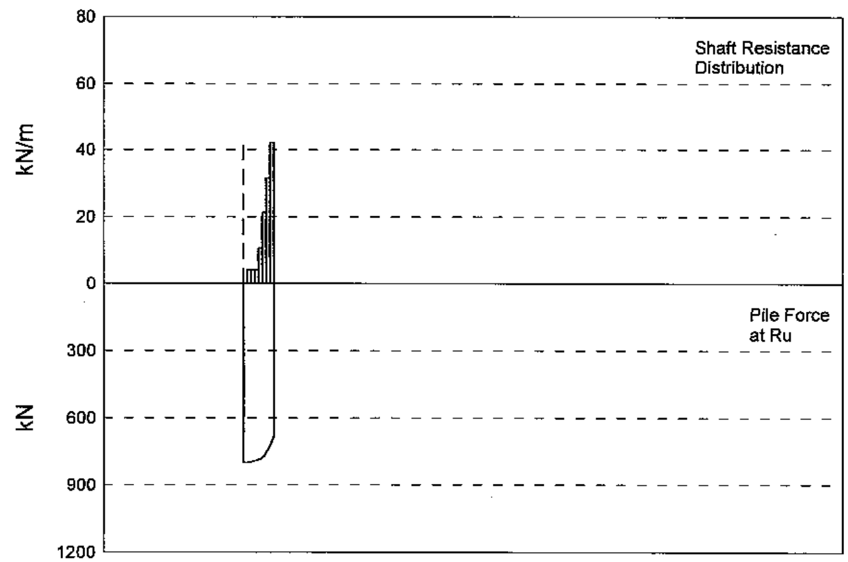
12-Sep-2007
CAPWAP(R) 2006







Ru =	800.6	kN
Rs =	120.0	kN
Rb =	680.6	kN
Dy =	15.2	mm
Dx =	19.9	mm



[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 800.6; along Shaft 120.0; at Toe 680.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
2	2.0	1.8	4.2	796.4	4.2	4.13	4.03	0.600
3	3.1	2.8	4.2	792.2	8.4	4.13	4.03	0.600
4	4.1	3.8	4.2	788.0	12.6	4.13	4.03	0.600
5	5.1	4.8	10.8	777.2	23.4	10.61	10.36	0.600
6	6.1	5.9	21.6	755.6	45.0	21.23	20.71	0.600
7	7.1	6.9	32.1	723.5	77.1	31.55	30.78	0.600
8	8.1	7.9	42.9	680.6	120.0	42.16	41.13	0.600
Avg. Shaft			15.0			15.19	14.82	0.600
Toe			680.6				20986.74	0.500

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.500	8.000
Case Damping Factor		0.213	1.008
Unloading Quake	(% of loading quake)	30	35
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	20	
Soil Plug Weight	(kN)		0.47

CAPWAP match quality = 4.21 (Wave Up Match) ; RSA = 0
Observed: final set = 4.730 mm; blow count = 211 b/m
Computed: final set = 1.269 mm; blow count = 788 b/m

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	1096.3	-83.2	13.1	-1.00	9.99	2.9	12.595
2	2.0	1104.6	-94.6	13.2	-1.13	9.77	2.8	12.013
3	3.1	1104.0	-104.0	13.2	-1.24	9.44	2.8	11.411
4	4.1	1109.9	-112.7	13.3	-1.35	9.10	2.7	10.796
5	5.1	1141.4	-123.2	13.7	-1.47	8.79	2.6	10.195
6	6.1	1177.4	-126.0	14.1	-1.51	8.36	2.4	9.600
7	7.1	1162.6	-120.9	13.9	-1.45	7.78	2.3	9.003
8	8.1	1084.1	-111.0	13.0	-1.33	6.74	2.3	8.445
Absolute	6.1			14.1			(T =	25.7 ms)
	6.1				-1.51		(T =	41.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1205.1	1114.3	1023.5	932.7	841.9	751.1	660.3	569.5	478.7	387.9
RX	1205.1	1114.3	1023.5	972.9	925.5	887.7	853.9	834.2	815.6	801.1
RU	1205.1	1114.3	1023.5	932.7	841.9	751.1	660.3	569.5	478.7	387.9

RAU = 757.8 (kN); RA2 = 985.4 (kN)

Current CAPWAP Ru = 800.6 (kN); Corresponding J(RP)= 0.45

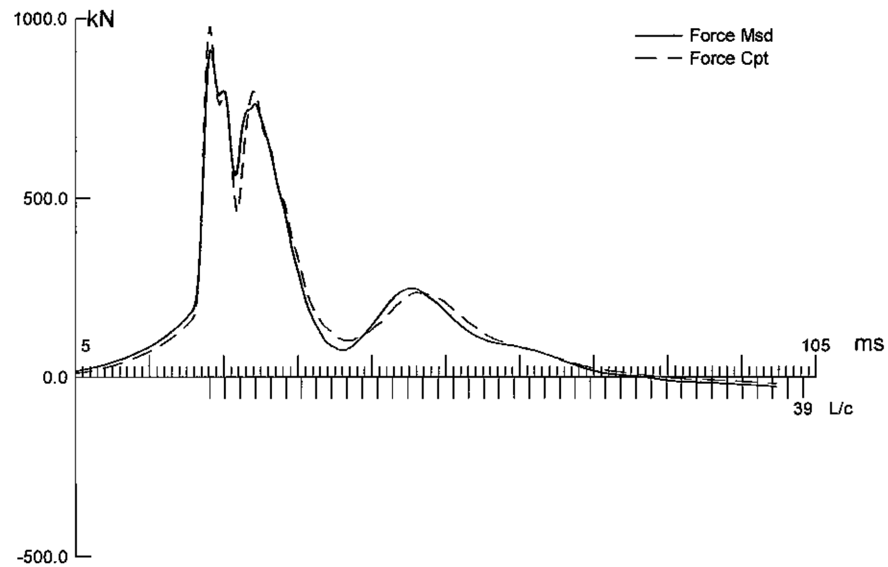
J(RX) = 0.9 matches RX9 within 5%

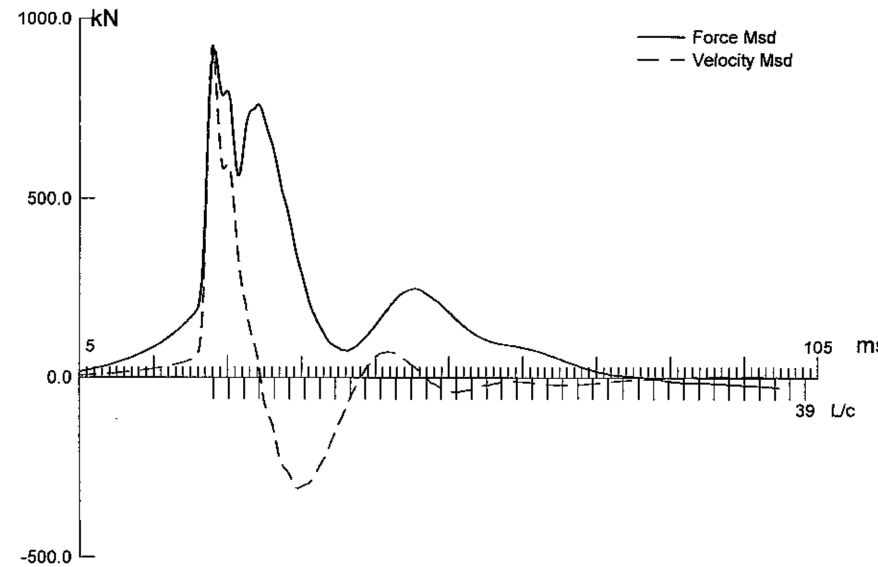
VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	m/s	kN	kN	kN	mm	mm	mm	kJ	kN
3.22	0.00	1087.0	1026.0	1026.0	13.897	5.069	4.730	10.5	1514.7

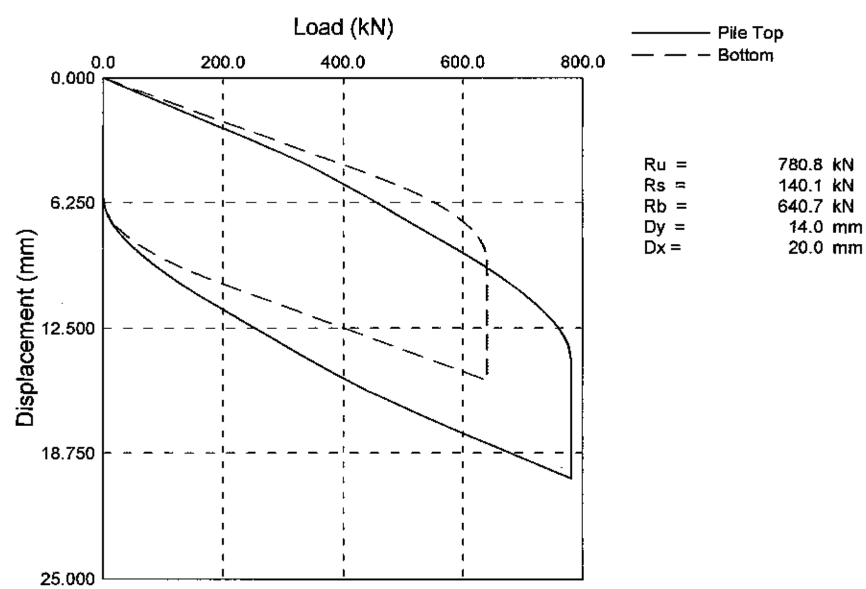
Peak Velocity Time = 24.15 ms.

[redacted]; Pile: [redacted]42; [redacted] Blow: 15 (Test: 10-Sep-2007 15:31:)

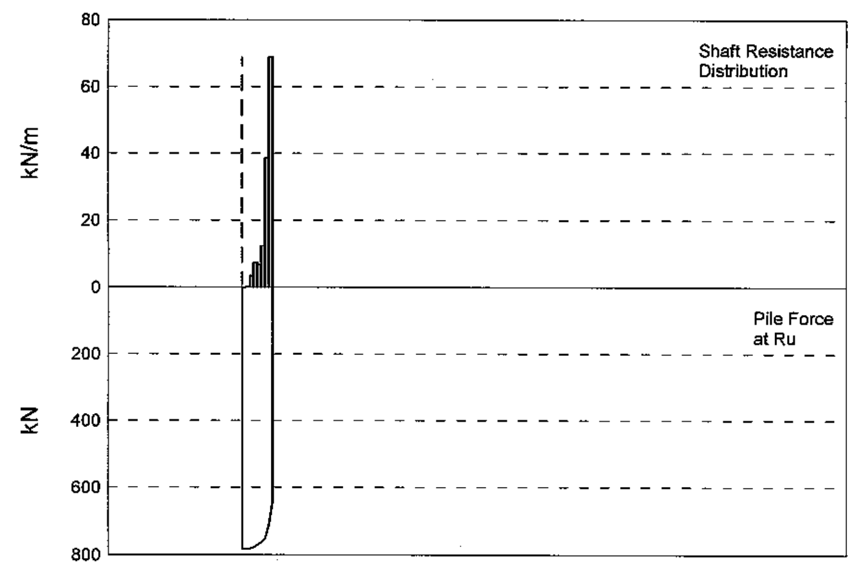
12-Sep-2007
CAPWAP(R) 2006







Ru =	780.8	kN
Rs =	140.1	kN
Rb =	640.7	kN
Dy =	14.0	mm
Dx =	20.0	mm



[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 780.8; along Shaft 140.1; at Toe 640.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
2	2.0	1.8	0.3	780.5	0.3	0.29	0.03	0.500
3	3.1	2.8	3.5	777.0	3.8	3.44	0.34	0.500
4	4.1	3.8	7.5	769.5	11.3	7.37	0.74	0.500
5	5.1	4.8	6.8	762.7	18.1	6.68	0.67	0.500
6	6.1	5.9	12.6	750.1	30.7	12.38	1.24	0.500
7	7.1	6.9	39.2	710.9	69.9	38.53	3.85	0.500
8	8.1	7.9	70.2	640.7	140.1	68.99	6.90	0.500
Avg. Shaft			17.5			17.73	1.77	0.500
Toe			640.7				19756.40	0.500

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.500	7.000
Case Damping Factor		0.218	0.997
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	67
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	7	
Soil Plug Weight	(kN)		0.41

CAPWAP match quality	=	5.08	(Wave Up Match) ; RSA = 0			
Observed: final set	=	6.000 mm;	blow count	=	167 b/m	
Computed: final set	=	1.964 mm;	blow count	=	509 b/m	

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	982.9	-18.3	12.4	-0.23	8.18	2.7	12.516
2	2.0	990.0	-17.9	12.4	-0.22	7.94	2.6	11.884
3	3.1	998.5	-17.3	12.5	-0.22	7.68	2.6	11.251
4	4.1	1002.2	-16.8	12.6	-0.21	7.36	2.6	10.608
5	5.1	992.3	-16.3	12.5	-0.20	6.97	2.5	9.969
6	6.1	992.5	-15.6	12.5	-0.20	6.62	2.6	9.345
7	7.1	1022.8	-14.0	12.9	-0.18	6.20	2.8	8.732
8	8.1	994.7	-10.3	12.5	-0.13	4.82	2.6	8.156
Absolute	7.1			12.9			(T =	27.2 ms)
	1.0				-0.23		(T =	100.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1143.0	1069.5	995.9	922.4	848.9	775.4	701.8	628.3	554.8	481.2
RX	1157.0	1093.3	1029.5	975.8	945.5	915.1	886.0	859.9	837.0	816.7
RU	1143.0	1069.5	995.9	922.4	848.9	775.4	701.8	628.3	554.8	481.2

RAU = 660.4 (kN); RA2 = 941.1 (kN)

Current CAPWAP Ru = 780.8 (kN); Corresponding J(RP)= 0.49

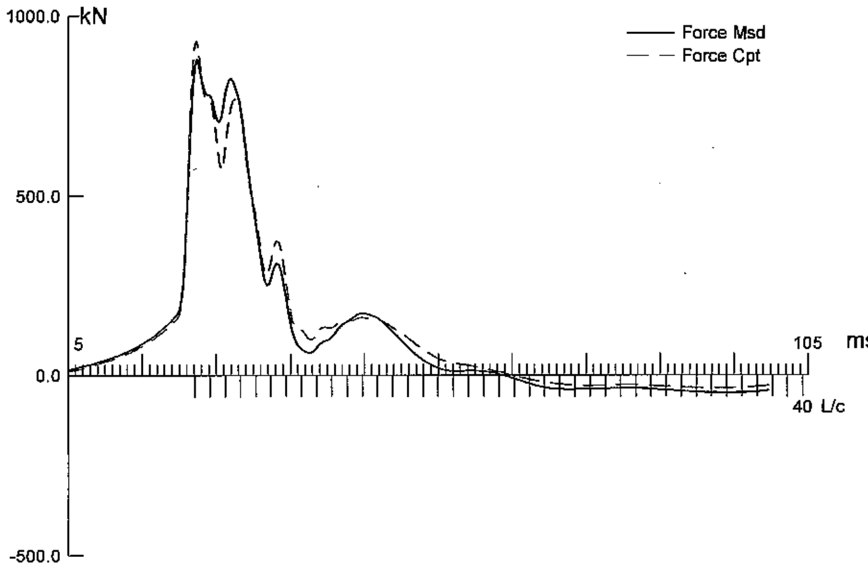
J(RX) = 0.9 matches RX9 within 5%

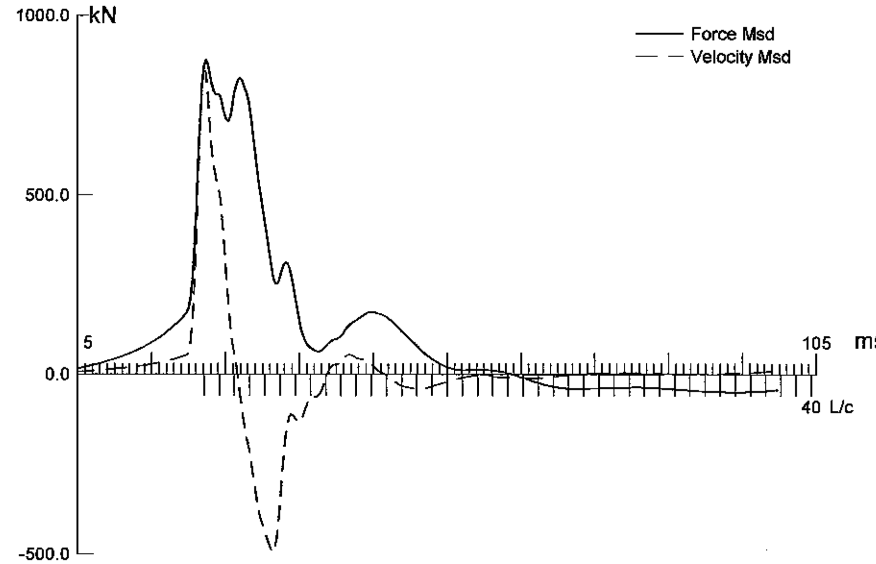
VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	m/s	kN	kN	kN	mm	mm	mm	kJ	kN
2.99	0.00	961.4	916.9	920.8	12.131	5.923	6.000	8.4	1377.8

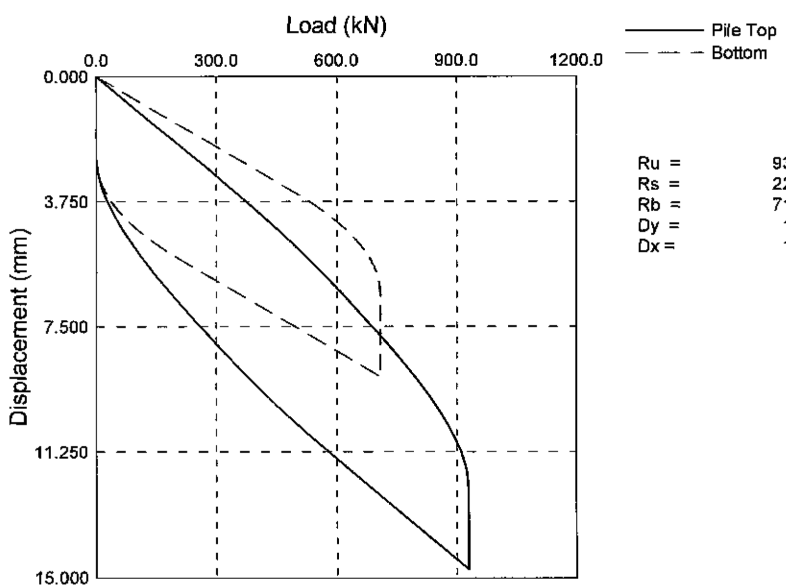
Peak Velocity Time = 23.38 ms.

[redacted] Pile [redacted]-47 [redacted]low: 14 (Test: 10-Sep-2007 15:16:)

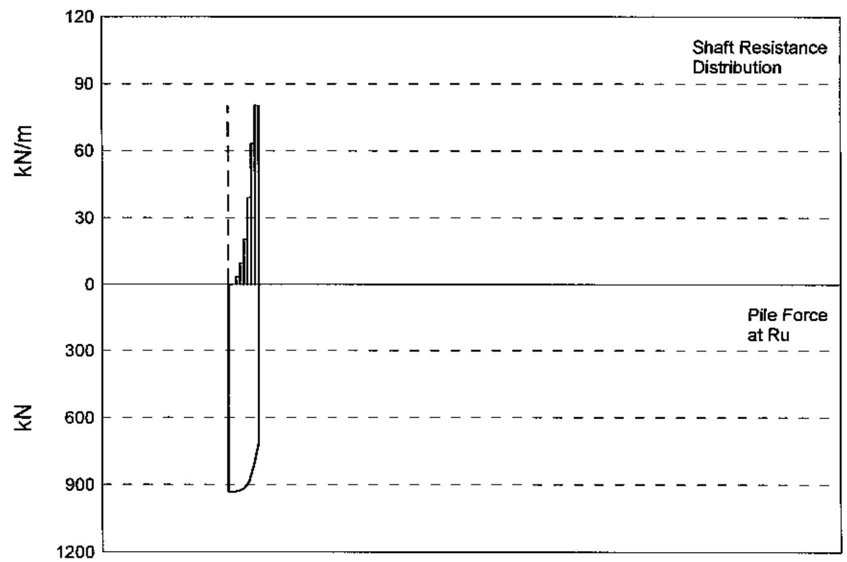
12-Sep-2007
CAPWAP(R) 2006







Ru = 930.1 kN
Rs = 220.0 kN
Rb = 710.1 kN
Dy = 12.3 mm
Dx = 14.7 mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 930.1; along Shaft 220.0; at Toe 710.1 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
2	2.0	1.8	0.2	929.9	0.2	0.20	0.20	0.500
3	3.1	2.8	3.6	926.3	3.8	3.54	3.54	0.500
4	4.1	3.8	9.8	916.5	13.6	9.63	9.63	0.500
5	5.1	4.8	20.7	895.8	34.3	20.34	20.34	0.500
6	6.1	5.9	39.7	856.1	74.0	39.02	39.02	0.500
7	7.1	6.9	64.2	791.9	138.2	63.10	63.10	0.500
8	8.1	7.9	81.8	710.1	220.0	80.39	80.39	0.500
Avg. Shaft			27.5			27.85	27.85	0.500
Toe			710.1				21896.39	0.500

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.500	5.000
Case Damping Factor		0.342	1.105
Damping Type			Smith
Unloading Quake	(% of loading quake)	97	97
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	10	
Soil Plug Weight	(kN)		0.44

CAPWAP match quality	=	6.24	(Wave Up Match) ; RSA = 0		
Observed: final set	=	2.500 mm;	blow count	=	400 b/m
Computed: final set	=	0.988 mm;	blow count	=	1012 b/m

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	937.5	-35.8	11.8	-0.45	6.89	2.4	10.288
2	2.0	949.0	-35.8	11.9	-0.45	6.61	2.4	9.611
3	3.1	966.8	-35.8	12.1	-0.45	6.31	2.4	8.921
4	4.1	984.5	-35.4	12.4	-0.45	5.95	2.3	8.214
5	5.1	978.6	-34.3	12.3	-0.43	5.50	2.2	7.485
6	6.1	1019.6	-32.0	12.8	-0.40	4.93	2.2	6.739
7	7.1	1011.7	-27.5	12.7	-0.35	4.23	2.2	6.033
8	8.1	960.4	-20.5	12.1	-0.26	3.10	2.0	5.397
Absolute	6.1			12.8			(T =	26.2 ms)
	3.1				-0.45		(T =	93.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1258.5	1209.2	1159.9	1110.6	1061.2	1011.9	962.6	913.3	864.0	814.6
RX	1271.4	1224.8	1183.1	1142.3	1101.4	1062.1	1043.4	1024.8	1006.2	987.5
RU	1258.5	1209.2	1159.9	1110.6	1061.2	1011.9	962.6	913.3	864.0	814.6

RAU = 913.1 (kN); RA2 = 1155.3 (kN)

Current CAPWAP Ru = 930.1 (kN); Corresponding J(RP)= 0.67;

RMX requires higher damping; see PDA-W

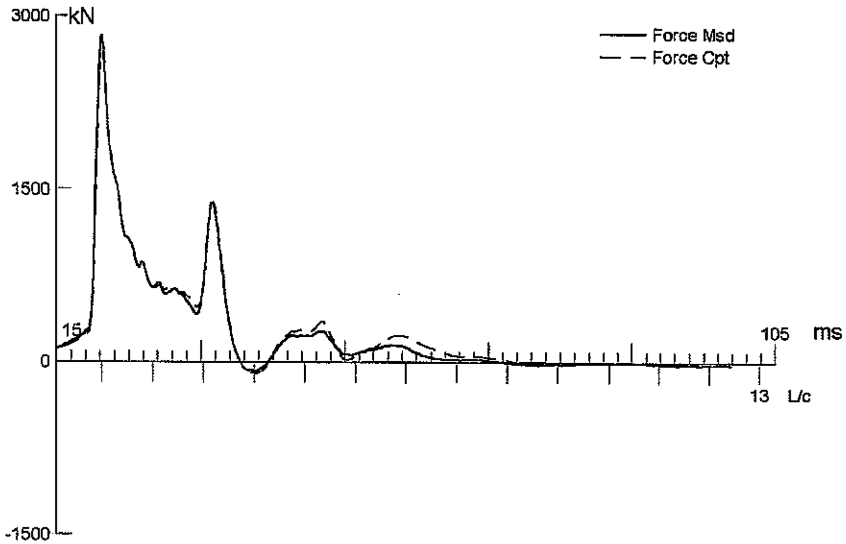
VMX m/s	VFN m/s	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
2.70	0.00	868.7	883.0	887.4	9.942	2.492	2.500	6.8	1360.8

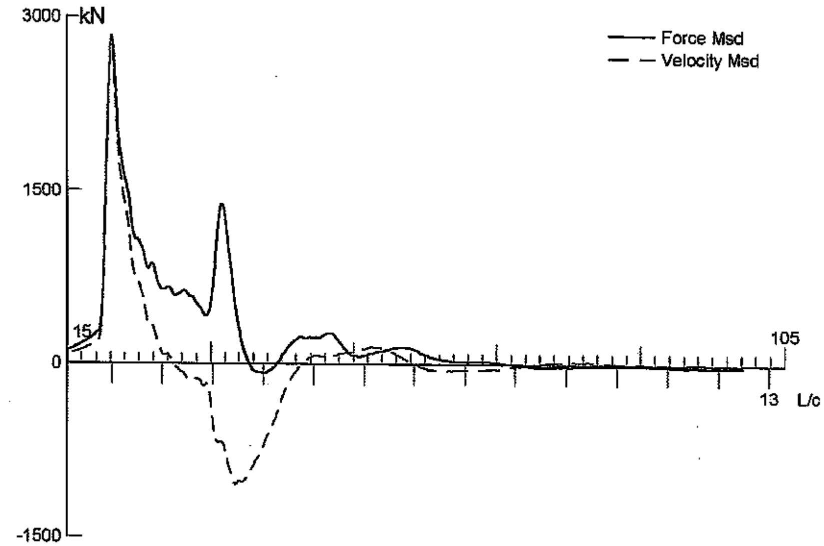
Peak Velocity Time = 22.35 ms.

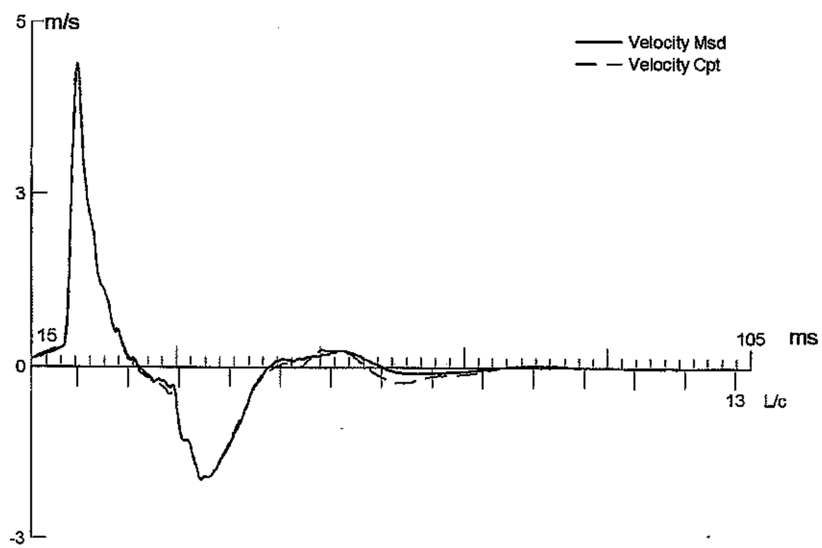
Site 12

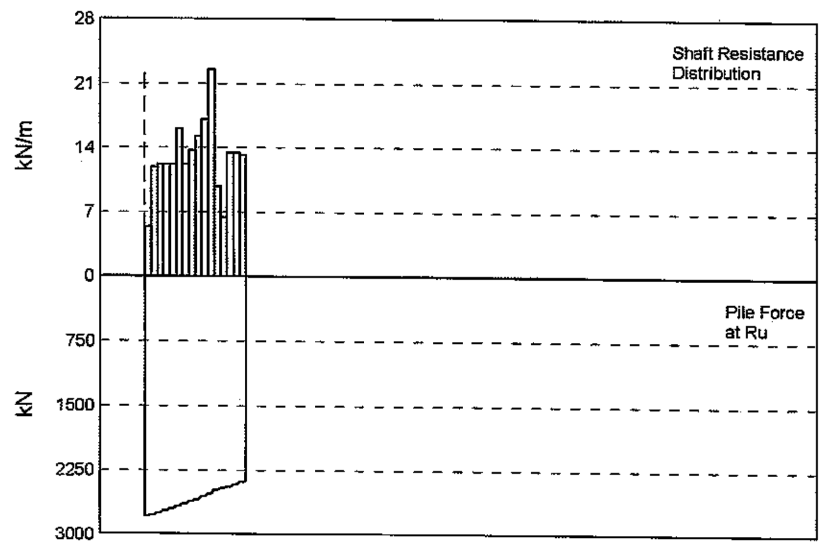
[redacted] Blow: 1324 (Test: 19-Aug-2008 17:53:)

13-Jan-2009
CAPWAP(R) 2006-2









[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2782.7; along Shaft 419.4; at Toe 2363.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2782.7					
1	2.0	1.2	10.9	2771.8	10.9	9.23	7.56	1.055	6.209
2	4.1	3.2	24.0	2747.8	34.9	11.82	9.68	1.055	6.209
3	6.1	5.2	24.7	2723.1	59.6	12.16	9.97	1.055	6.209
4	8.1	7.3	24.6	2698.5	84.2	12.11	9.93	1.055	6.209
5	10.2	9.3	24.7	2673.8	108.9	12.16	9.97	1.055	6.209
6	12.2	11.3	32.6	2641.2	141.5	16.05	13.16	1.055	6.209
7	14.2	13.4	24.7	2616.5	166.2	12.16	9.97	1.055	6.209
8	16.3	15.4	27.6	2588.9	193.8	13.59	11.14	1.055	6.209
9	18.3	17.4	31.0	2557.9	224.8	15.26	12.51	1.055	6.209
10	20.3	19.5	34.8	2523.1	259.6	17.13	14.04	1.055	6.209
11	22.3	21.5	45.7	2477.4	305.3	22.50	18.44	1.055	6.209
12	24.4	23.5	19.7	2457.7	325.0	9.70	7.95	1.055	6.209
13	26.4	25.6	13.0	2444.7	338.0	6.40	5.25	1.055	6.209
14	28.4	27.6	27.3	2417.4	365.3	13.44	11.02	1.055	6.209
15	30.5	29.6	27.3	2390.1	392.6	13.44	11.02	1.055	5.923
16	32.5	31.6	26.8	2363.3	419.4	13.19	10.81	1.055	4.465
Avg. Shaft			26.2			13.25	10.86	1.055	6.079
Toe			2363.3				25405.00	0.207	4.510

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.690	0.763
Damping Type		Smith	
Unloading Quake	(% of loading quake)	31	79
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	78	
Resistance Gap (included in Toe Quake) (mm)			0.342

CAPWAP match quality = 1.83 (Wave Up Match) ; RSA = 0
Observed: final set = 1.700 mm; blow count = 588 b/m
Computed: final set = 1.578 mm; blow count = 634 b/m

max. Top Comp. Stress = 180.1 MPa (T= 21.0 ms, max= 1.006 x Top)
max. Comp. Stress = 181.1 MPa (Z= 4.1 m, T= 21.6 ms)
max. Tens. Stress = -11.85 MPa (Z= 4.1 m, T= 39.8 ms)
max. Energy (EMX) = 24.05 kJ; max. Measured Top Displ. (DMX)=14.67 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2857.9	-133.8	180.1	-8.43	24.05	4.4	14.580
2	2.0	2874.0	-162.9	181.1	-10.26	23.97	4.3	14.368
4	4.1	2874.4	-188.1	181.1	-11.85	23.39	4.3	13.927
6	6.1	2827.0	-179.6	178.1	-11.32	22.37	4.2	13.491
8	8.1	2778.4	-164.3	175.1	-10.35	21.38	4.1	13.058
10	10.2	2733.7	-145.3	172.3	-9.15	20.44	4.0	12.634
12	12.2	2708.0	-115.1	170.6	-7.25	19.55	3.9	12.221
14	14.2	2650.5	-79.9	167.0	-5.04	18.49	3.9	11.822
16	16.3	2622.1	-60.9	165.2	-3.84	17.71	3.8	11.433
18	18.3	2589.9	-41.4	163.2	-2.61	16.84	3.7	10.959
20	20.3	2556.2	-17.9	161.1	-1.13	15.88	3.6	10.410
22	22.3	2520.2	0.0	158.8	0.00	14.84	3.5	9.798
23	23.4	2416.1	-9.0	152.2	-0.57	13.77	3.5	9.440
24	24.4	2436.2	-47.2	153.5	-2.97	13.54	3.4	9.055
25	25.4	2400.7	-69.1	151.3	-4.35	12.95	3.4	8.661
26	26.4	2419.6	-96.8	152.5	-6.10	12.64	3.4	8.243
27	27.4	2408.1	-106.7	151.7	-6.72	12.07	3.4	7.748
28	28.4	2434.2	-132.7	153.4	-8.36	11.55	3.3	7.184
29	29.5	2374.6	-128.1	149.6	-8.07	10.63	3.3	6.579
30	30.5	2360.2	-143.1	148.7	-9.02	9.96	3.3	5.928
31	31.5	2401.3	-127.2	151.3	-8.02	8.95	3.2	5.245
32	32.5	2769.4	-137.5	174.5	-8.66	8.82	2.9	4.516
Absolute	4.1			181.1			(T =	21.6 ms)
	4.1				-11.85		(T =	39.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3320.8	3083.4	2846.1	2608.7	2371.3	2133.9	1896.6	1659.2	1421.8	1184.5
RX	3425.7	3204.4	2983.1	2790.1	2610.8	2460.5	2362.4	2278.8	2207.3	2139.4
RU	3573.7	3361.6	3149.5	2937.4	2725.4	2513.3	2301.2	2089.1	1877.0	1665.0

RAU = 162.7 (kN); RA2 = 1949.3 (kN)

Current CAPWAP Ru = 2782.7 (kN); Corresponding J(RP)= 0.23; J(RX) = 0.30

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.45	20.82	2850.3	2844.2	2844.2	14.671	1.841	1.700	24.3	2967.5

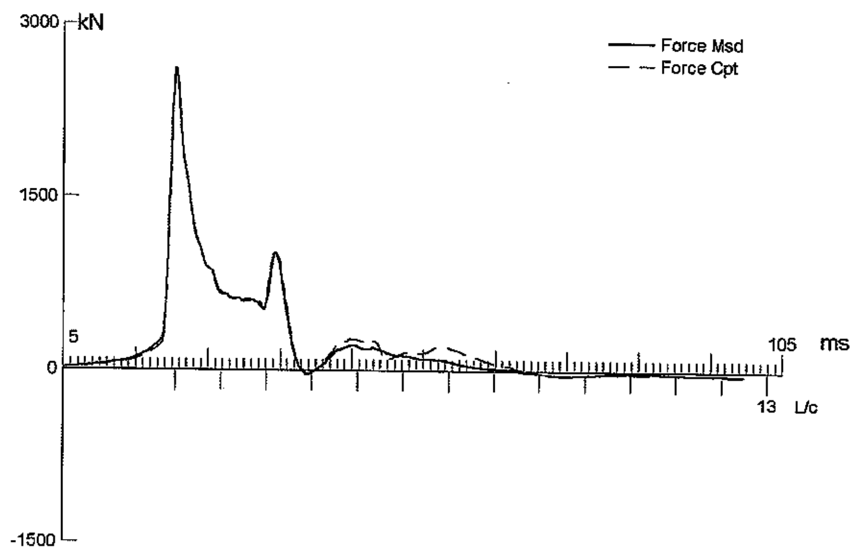
PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206874.8	77.300	1.220
32.50	158.70	206874.8	77.300	1.220

Toe Area 0.093 m²

Top Segment Length 1.02 m, Top Impedance 640.86 kN/m/s

Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 12.7 ms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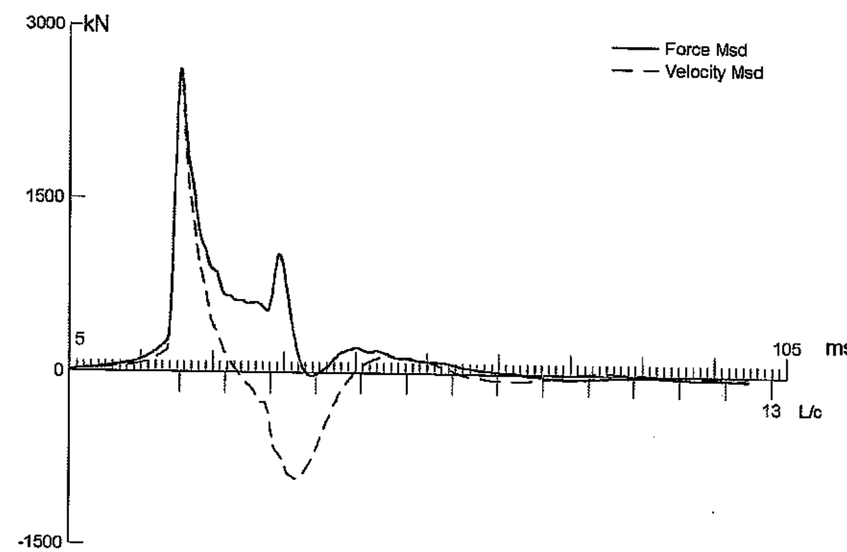
kN
1500
0
-1500
5
105
ms
13
L/c
Force Msd
Force Cpt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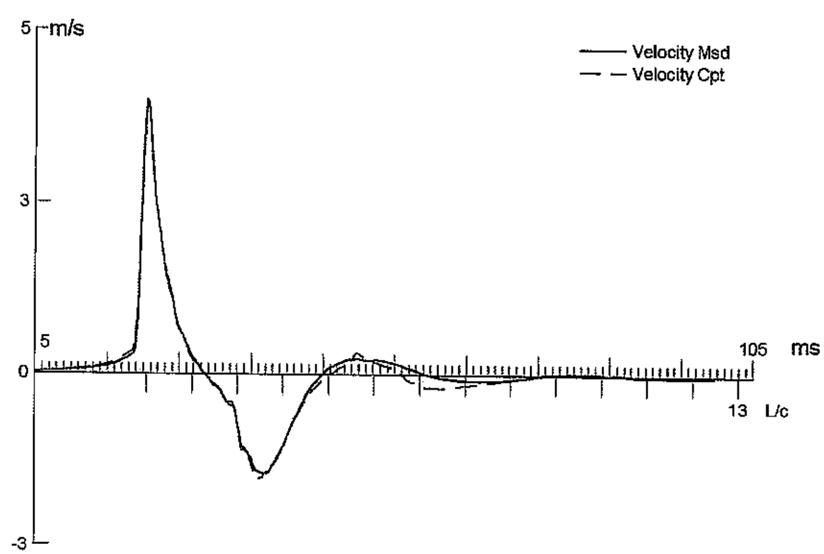
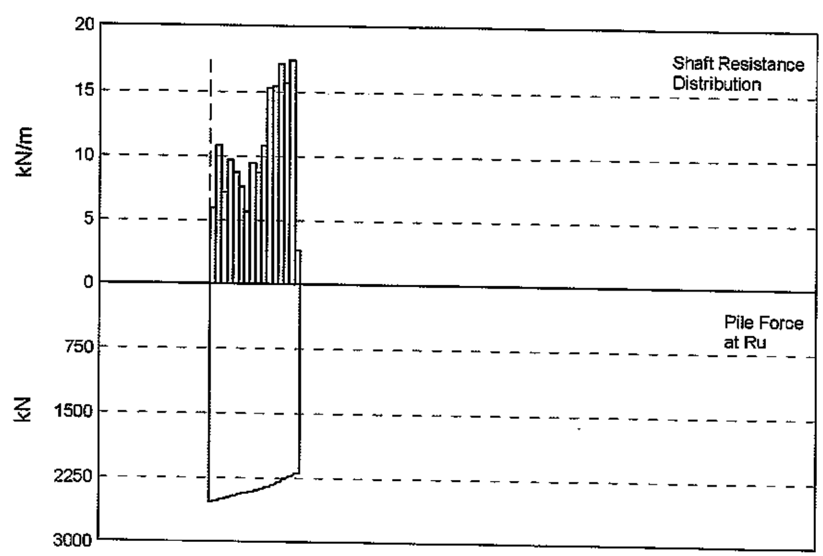
kN
1500
0
-1500
5
105
ms
13
L/c
Force Msd
Velocity Msd


5
m/s
3
0
-3
5
105
ms
13
L/c
Velocity Msd
Velocity Cpt


20
15
10
5
0
kN/m
750
1500
2250
3000
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2525.0; along Shaft 341.3; at Toe 2183.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2525.0					
1	2.0	1.0	12.0	2513.0	12.0	11.64	9.54	1.425	5.000
2	4.1	3.1	21.9	2491.1	33.9	10.78	8.84	1.425	5.000
3	6.1	5.1	14.6	2476.5	48.5	7.19	5.89	1.425	5.000
4	8.1	7.1	19.7	2456.8	68.2	9.70	7.95	1.425	5.000
5	10.2	9.2	17.7	2439.1	85.9	8.71	7.14	1.425	5.000
6	12.2	11.2	15.4	2423.7	101.3	7.58	6.21	1.425	5.000
7	14.2	13.2	11.5	2412.2	112.8	5.66	4.64	1.425	5.000
8	16.3	15.3	19.2	2393.0	132.0	9.45	7.75	1.425	5.000
9	18.3	17.3	17.7	2375.3	149.7	8.71	7.14	1.425	5.000
10	20.3	19.3	21.9	2353.4	171.6	10.78	8.84	1.425	5.000
11	22.3	21.3	31.1	2322.3	202.7	15.31	12.55	1.425	5.000
12	24.4	23.4	31.3	2291.0	234.0	15.41	12.63	1.425	5.000
13	26.4	25.4	34.7	2256.3	268.7	17.08	14.00	1.425	5.000
14	28.4	27.4	31.8	2224.5	300.5	15.66	12.83	1.425	5.000
15	30.5	29.5	35.3	2189.2	335.8	17.38	14.24	1.425	5.000
16	32.5	31.5	5.5	2183.7	341.3	2.71	2.22	1.425	4.360
Avg. Shaft			21.3			10.83	8.88	1.425	4.990
Toe			2183.7				23474.33	0.190	2.952

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.759	0.647
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	30	54
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	51	
Resistance Gap (included in Toe Quake) (mm)			0.481
Soil Plug Weight	(kN)		1.93

CAPWAP match quality	= 2.30	(Wave Up Match) ; RSA = 0	
Observed: final set	= 1.300 mm;	blow count	= 769 b/m
Computed: final set	= 2.300 mm;	blow count	= 435 b/m
max. Top Comp. Stress	= 164.6 MPa	(T= 20.8 ms, max= 1.009 x Top)	
max. Comp. Stress	= 166.2 MPa	(Z= 2.0 m, T= 21.2 ms)	
max. Tens. Stress	= -13.10 MPa	(Z= 26.4 m, T= 44.8 ms)	
max. Energy (EMX)	= 23.48 kJ;	max. Measured Top Displ. (DMX)=14.88 mm	

[redacted]Pile 12 (EOID)
August 19, 2008; Blow: 1432
[redacted]

Test: 19-Aug-2008 16:44:
CAPWAP(R) 2006-2
OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2612.5	-59.8	164.6	-3.77	23.48	4.0	14.911
2	2.0	2637.1	-77.7	166.2	-4.90	23.41	3.9	14.711
4	4.1	2618.3	-104.7	165.0	-6.60	22.66	3.9	14.300
6	6.1	2535.9	-99.6	159.8	-6.27	21.45	3.8	13.877
8	8.1	2502.8	-91.7	157.7	-5.78	20.61	3.7	13.436
10	10.2	2454.4	-83.4	154.7	-5.26	19.58	3.6	12.986
12	12.2	2438.1	-73.3	153.6	-4.62	18.68	3.5	12.522
14	14.2	2407.6	-74.1	151.7	-4.67	17.89	3.4	12.031
16	16.3	2392.5	-83.1	150.8	-5.24	17.25	3.4	11.521
18	18.3	2345.6	-88.6	147.8	-5.58	16.37	3.3	10.985
20	20.3	2320.9	-100.6	146.2	-6.34	15.50	3.2	10.363
22	22.3	2296.7	-140.6	144.7	-8.86	14.51	3.1	9.679
23	23.4	2208.1	-151.0	139.1	-9.51	13.48	3.1	9.298
24	24.4	2244.7	-178.9	141.4	-11.27	13.18	3.0	8.874
25	25.4	2164.9	-182.4	136.4	-11.49	12.17	3.0	8.426
26	26.4	2203.5	-208.0	138.8	-13.10	11.80	2.9	7.958
27	27.4	2121.1	-195.3	133.7	-12.31	10.75	2.9	7.457
28	28.4	2163.3	-207.2	136.3	-13.06	10.29	2.8	6.918
29	29.5	2119.4	-185.4	133.5	-11.68	9.29	2.7	6.353
30	30.5	2127.4	-183.9	134.0	-11.59	8.71	2.7	5.754
31	31.5	2101.4	-148.3	132.4	-9.34	7.73	2.7	5.133
32	32.5	2396.7	-137.1	151.0	-8.64	7.61	2.5	4.486
Absolute	2.0			166.2			(T =	21.2 ms)
	26.4				-13.10		(T =	44.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3159.4	2950.3	2741.1	2532.0	2322.9	2113.7	1904.6	1695.4	1486.3	1277.1
RX	3159.4	2955.7	2763.1	2570.4	2379.6	2221.3	2097.4	1995.7	1906.3	1823.0
RU	3366.6	3178.2	2989.8	2801.4	2613.0	2424.5	2236.1	2047.7	1859.3	1670.8

RAU = 155.4 (kN); RA2 = 1513.9 (kN)

Current CAPWAP Ru = 2525.0 (kN); Corresponding J(RP)= 0.30; J(RX) = 0.32

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.04	20.62	2591.3	2659.6	2659.6	14.876	1.299	1.300	23.7	2925.6

PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206874.8	77.300	1.220
32.50	158.70	206874.8	77.300	1.220

Toe Area 0.093 m^2

Top Segment Length 1.02 m, Top Impedance 640.86 kN/m/s

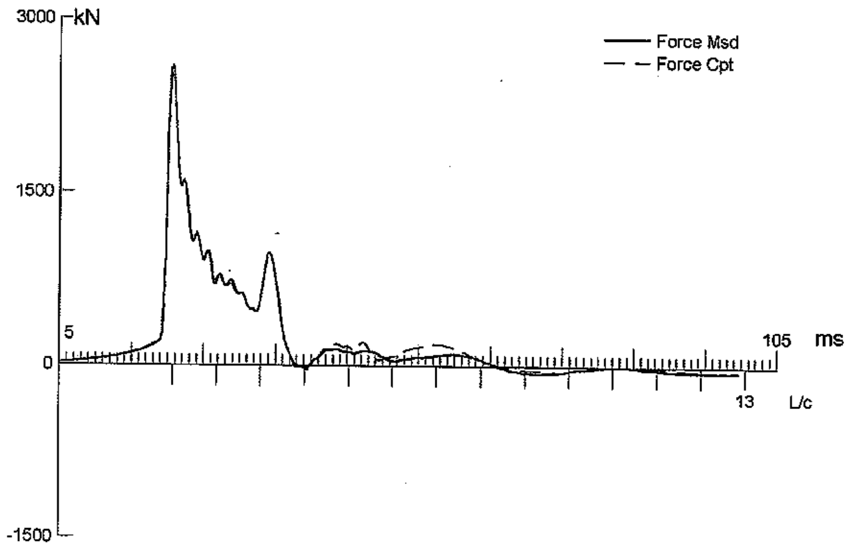
[redacted] 12 (EOID)
August 19, 2008; Blow: 1432
[redacted]

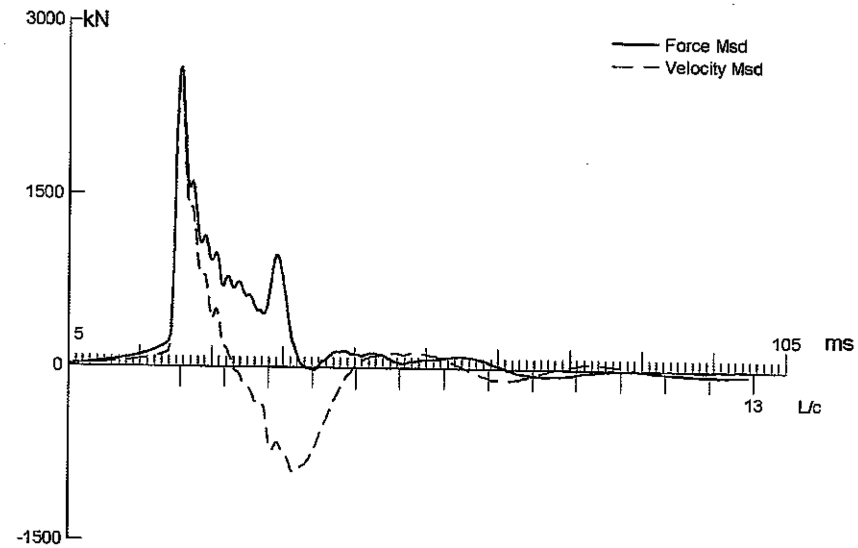
Test: 19-Aug-2008 16:44:
CAPWAP(R) 2006-2
OP: RB

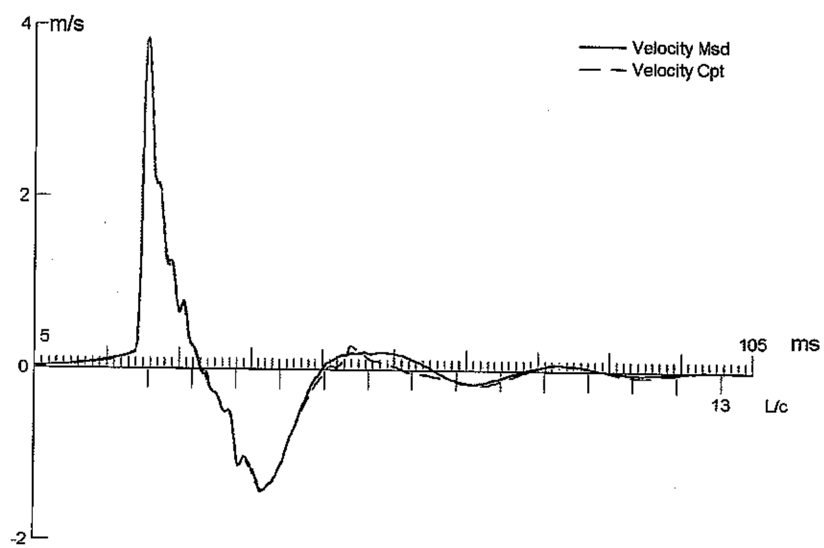
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 12.7 ms

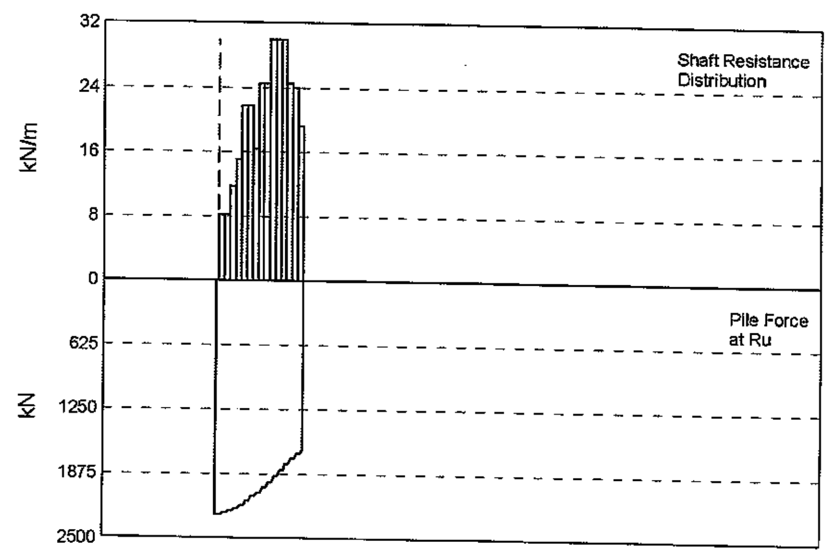
[redacted] Pile 5 (RSTRK); August 28, 2008; Blow: 4 (Test: 28-Aug-2008 11:21:)

13-Jan-2009
CAPWAP(R) 2006-2









[redacted]

[redacted] Pile 5 (RSTRK) Test: 28-Aug-2008 11:21:
August 28, 2008; Blow: 4 CAPWAP(R) 2006-2
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2266.7; along Shaft 629.4; at Toe 1637.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2266.7					
1	3.0	1.5	16.6	2250.1	16.6	10.72	8.79	0.623	1.025
2	5.1	3.6	16.6	2233.5	33.2	8.17	6.70	0.623	1.024
3	7.1	5.6	23.8	2209.7	57.0	11.71	9.60	0.623	1.024
4	9.1	7.6	30.6	2179.1	87.6	15.06	12.34	0.623	1.024
5	11.2	9.7	44.3	2134.8	131.9	21.80	17.87	0.623	1.024
6	13.2	11.7	44.3	2090.5	176.2	21.80	17.87	0.623	1.024
7	15.2	13.7	33.2	2057.3	209.4	16.34	13.39	0.623	1.024
8	17.3	15.8	49.8	2007.5	259.2	24.50	20.09	0.623	1.024
9	19.3	17.8	49.8	1957.7	309.0	24.50	20.09	0.623	1.024
10	21.3	19.8	60.9	1896.8	369.9	29.97	24.56	0.623	1.024
11	23.4	21.9	60.9	1835.9	430.8	29.97	24.56	0.623	1.024
12	25.4	23.9	60.9	1775.0	491.7	29.97	24.56	0.623	1.024
13	27.4	25.9	49.8	1725.2	541.5	24.50	20.09	0.623	1.024
14	29.5	28.0	48.8	1676.4	590.3	24.01	19.68	0.623	1.024
15	31.5	30.0	39.1	1637.3	629.4	19.24	15.77	0.623	0.818
Avg. Shaft			42.0			20.98	17.20	0.623	1.011
Toe			1637.3				17600.64	0.398	3.453

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.612	1.017
Unloading Quake	(% of loading quake)	177	39
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	22	
Resistance Gap (included in Toe Quake) (mm)			0.622
Soil Support Dashpot		0.350	0.000
Soil Support Weight	(kN)	12.40	0.00

CAPWAP match quality = 1.88 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 0.812 mm; blow count = 1232 b/m

max. Top Comp. Stress = 164.0 MPa (T= 21.0 ms, max= 1.010 x Top)
max. Comp. Stress = 165.7 MPa (Z= 3.0 m, T= 21.4 ms)
max. Tens. Stress = -17.61 MPa (Z= 25.4 m, T= 44.2 ms)
max. Energy (EMX) = 21.13 kJ; max. Measured Top Displ. (DMX)=13.82 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2603.4	-56.8	164.0	-3.58	21.13	3.8	13.385
2	2.0	2616.1	-77.2	164.8	-4.86	21.04	3.8	13.161
4	4.1	2589.0	-95.9	163.1	-6.04	20.39	3.8	12.699
6	6.1	2570.8	-111.7	162.0	-7.04	19.75	3.7	12.221
8	8.1	2540.0	-117.3	160.1	-7.39	18.95	3.6	11.737
10	10.2	2504.3	-123.5	157.8	-7.78	18.02	3.5	11.261
12	12.2	2427.9	-139.9	153.0	-8.82	16.86	3.4	10.796
14	14.2	2343.0	-151.6	147.6	-9.55	15.73	3.4	10.312
16	16.3	2308.6	-184.3	145.5	-11.62	14.86	3.3	9.820
18	18.3	2230.2	-190.4	140.5	-12.00	13.61	3.1	9.185
20	20.3	2164.9	-182.9	136.4	-11.53	12.39	3.0	8.494
21	21.3	2196.8	-204.8	138.4	-12.90	12.20	3.0	8.160
22	22.4	2071.7	-226.1	130.5	-14.25	11.05	2.9	7.793
23	23.4	2102.6	-264.7	132.5	-16.68	10.77	2.9	7.377
24	24.4	1978.7	-265.9	124.7	-16.76	9.64	2.8	6.954
25	25.4	2005.6	-279.4	126.4	-17.61	9.37	2.7	6.542
26	26.4	1877.9	-267.6	118.3	-16.86	8.35	2.7	6.119
27	27.4	1902.4	-275.7	119.9	-17.37	8.00	2.7	5.656
28	28.5	1804.0	-259.0	113.7	-16.32	7.07	2.6	5.165
29	29.5	1837.4	-259.7	115.8	-16.36	6.63	2.6	4.637
30	30.5	1921.5	-240.0	121.1	-15.12	5.79	2.4	4.112
31	31.5	2149.0	-240.1	135.4	-15.13	5.64	2.1	3.588
Absolute	3.0			165.7			(T =	21.4 ms)
	25.4				-17.61		(T =	44.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3059.8	2856.9	2654.0	2451.1	2248.3	2045.4	1842.5	1639.6	1436.7	1233.8
RX	3134.7	2940.7	2746.8	2552.8	2368.1	2198.4	2028.8	1887.7	1780.7	1706.2
RU	3254.3	3070.9	2887.5	2704.1	2520.6	2337.2	2153.8	1970.4	1786.9	1603.5

RAU = 284.6 (kN); RA2 = 1447.7 (kN)

Current CAPWAP Ru = 2266.7 (kN); Corresponding J(RP)= 0.39; J(RX) = 0.46

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.87	20.83	2479.6	2609.0	2609.0	13.816	1.008	1.000	21.4	2890.0

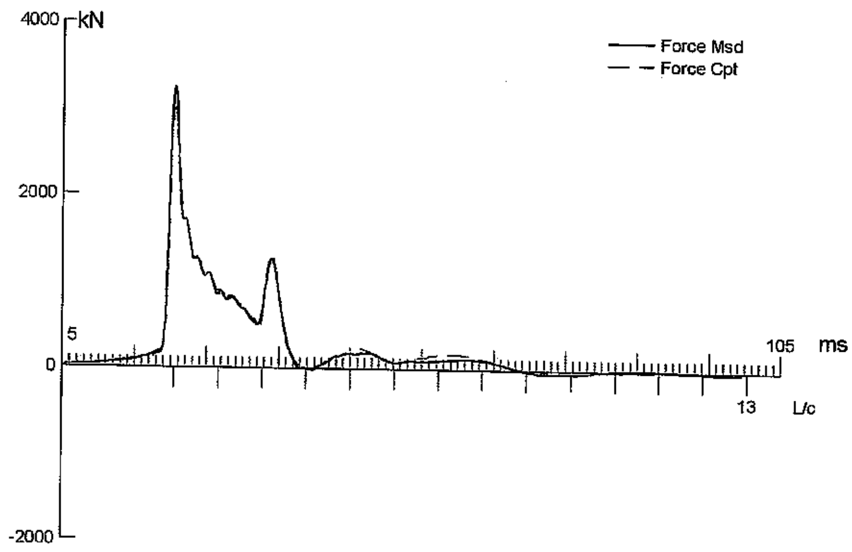
PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206874.8	77.300	1.220
31.50	158.70	206874.8	77.300	1.220

Toe Area 0.093 m²

Top Segment Length 1.02 m, Top Impedance 640.86 kN/m/s

Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 12.3 ms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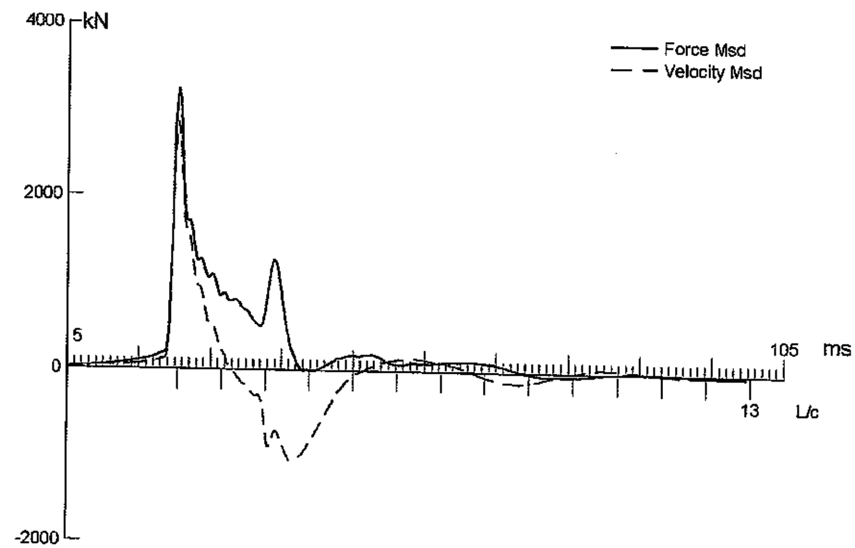
2000
0
-2000
5
105 ms
13 L/c
Force Msd
Force Cpt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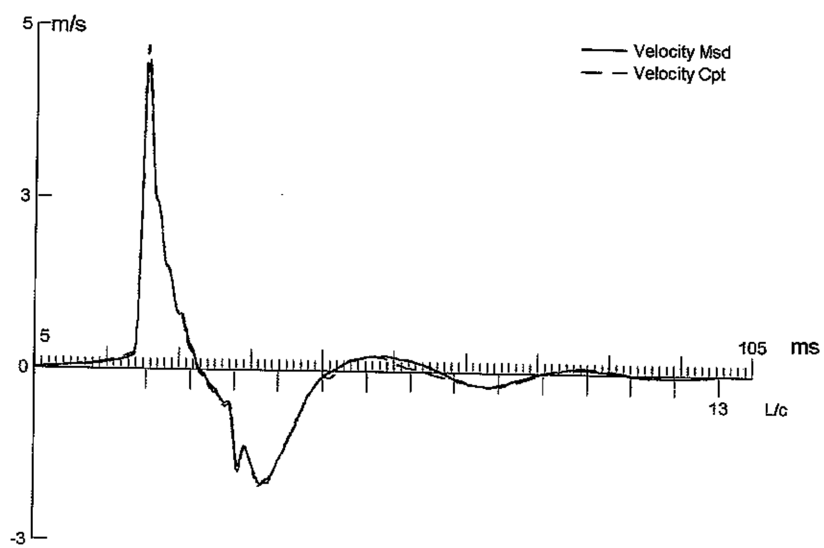
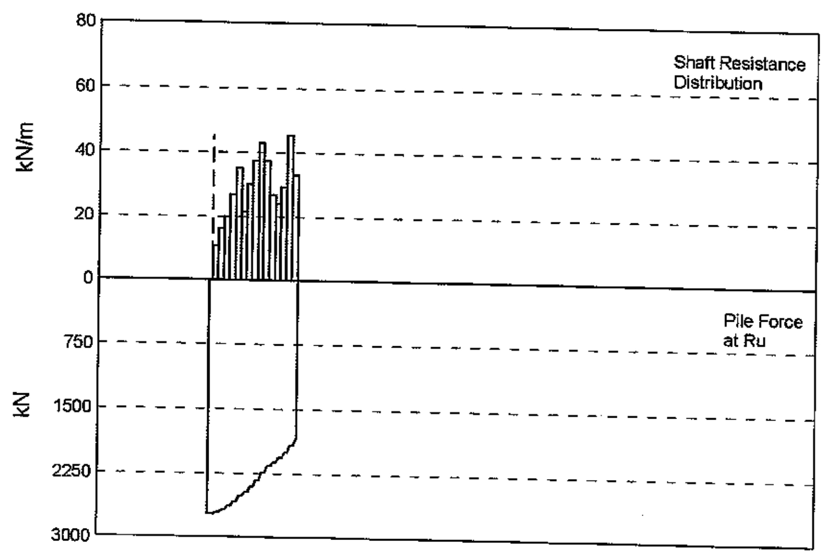
0
-2000
5
105 ms
13 L/c
Force Msd
Velocity Msd


5 m/s
3
0
-3
5
105 ms
13 L/c
Velocity Msd
Velocity Cpt


80
60
40
20
0
750
1500
2250
3000
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted] S_Abut_Pile 5A RS
September 2, 2008; Blow: 76; PILCO 19-42_ FS#4
[redacted]

Test: 02-Sep-2008 11:09:
CAPWAP(R) 2006-2
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2723.5; along Shaft 883.2; at Toe 1840.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2723.5					
1	3.0	1.3	21.7	2701.8	21.7	16.71	13.70	0.648	6.816
2	5.1	3.3	32.6	2669.2	54.3	16.04	13.15	0.648	6.817
3	7.1	5.4	40.1	2629.1	94.4	19.73	16.17	0.648	6.817
4	9.1	7.4	54.3	2574.8	148.7	26.72	21.90	0.648	6.817
5	11.2	9.4	71.1	2503.7	219.8	34.99	28.68	0.648	6.817
6	13.2	11.5	43.2	2460.5	263.0	21.26	17.42	0.648	6.817
7	15.2	13.5	61.1	2399.4	324.1	30.07	24.64	0.648	6.817
8	17.3	15.5	75.7	2323.7	399.8	37.25	30.53	0.648	6.817
9	19.3	17.6	86.9	2236.8	486.7	42.76	35.05	0.648	6.817
10	21.3	19.6	75.7	2161.1	562.4	37.25	30.53	0.648	6.817
11	23.4	21.6	54.3	2106.8	616.7	26.72	21.90	0.648	6.817
12	25.4	23.7	48.6	2058.2	665.3	23.91	19.60	0.648	6.817
13	27.4	25.7	59.4	1998.8	724.7	29.23	23.96	0.648	6.817
14	29.5	27.7	91.9	1906.9	816.6	45.22	37.07	0.648	6.817
15	31.5	29.8	66.6	1840.3	883.2	32.77	26.86	0.648	5.590
Avg. Shaft			58.9			29.69	24.33	0.648	6.724
Toe			1840.3				19782.85	0.645	5.615

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.893	1.852
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	63	115
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	18	
Resistance Gap (included in Toe Quake) (mm)			1.334
Soil Plug Weight	(kN)		1.29

CAPWAP match quality = 1.49 (Wave Up Match) ; RSA = 0
Observed: final set = 0.900 mm; blow count = 1111 b/m
Computed: final set = 0.967 mm; blow count = 1034 b/m

max. Top Comp. Stress = 193.1 MPa (T= 20.8 ms, max= 1.009 x Top)
max. Comp. Stress = 194.9 MPa (Z= 3.0 m, T= 21.2 ms)
max. Tens. Stress = -9.73 MPa (Z= 29.5 m, T= 45.4 ms)
max. Energy (EMX) = 28.14 kJ; max. Measured Top Displ. (DMX)=15.47 mm

Analysis: 13-Jan-2009

[redacted] Pile 5A RS Test: 02-Sep-2008 11:09:
September 2, 2008; Blow: 76; PILCO 19-42_ FS#4 CAPWAP(R) 2006-2
[redacted] OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3064.6	-48.1	193.1	-3.03	28.14	4.7	15.641
2	2.0	3075.1	-74.0	193.8	-4.66	28.02	4.7	15.371
4	4.1	3054.1	-108.4	192.4	-6.83	27.14	4.6	14.818
6	6.1	3018.0	-119.7	190.2	-7.54	26.00	4.6	14.267
8	8.1	2976.7	-126.7	187.6	-7.98	24.76	4.5	13.738
10	10.2	2919.0	-118.1	183.9	-7.44	23.27	4.4	13.237
12	12.2	2823.6	-104.7	177.9	-6.60	21.57	4.3	12.814
14	14.2	2783.3	-110.3	175.4	-6.95	20.51	4.2	12.372
16	16.3	2721.9	-101.5	171.5	-6.40	19.15	4.1	11.884
18	18.3	2643.2	-80.0	166.6	-5.04	17.54	4.0	11.302
20	20.3	2545.7	-62.5	160.4	-3.94	15.81	3.9	10.665
21	21.3	2579.7	-88.3	162.6	-5.56	15.64	3.8	10.352
22	22.4	2456.5	-87.9	154.8	-5.54	14.34	3.8	10.009
23	23.4	2480.3	-110.6	156.3	-6.97	14.09	3.8	9.629
24	24.4	2395.0	-112.5	150.9	-7.09	13.11	3.8	9.242
25	25.4	2415.6	-133.5	152.2	-8.41	12.85	3.7	8.849
26	26.4	2342.8	-133.5	147.6	-8.41	11.97	3.7	8.414
27	27.4	2368.3	-148.8	149.2	-9.38	11.55	3.7	7.919
28	28.5	2274.0	-141.2	143.3	-8.90	10.47	3.7	7.376
29	29.5	2255.4	-154.4	142.1	-9.73	9.97	3.7	6.817
30	30.5	2093.6	-135.1	131.9	-8.51	8.73	3.8	6.258
31	31.5	2636.8	-145.6	166.1	-9.17	8.38	3.4	5.664
Absolute	3.0			194.9			(T =	21.2 ms)
	29.5				-9.73		(T =	45.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3611.3	3362.5	3113.8	2865.1	2616.3	2367.6	2118.9	1870.1	1621.4	1372.7
RX	3741.7	3504.6	3268.6	3061.5	2854.4	2647.3	2469.5	2316.7	2197.4	2109.2
RU	3693.8	3453.4	3212.9	2972.4	2732.0	2491.5	2251.0	2010.5	1770.1	1529.6

RAU = 369.0 (kN); RA2 = 1731.5 (kN)

Current CAPWAP Ru = 2723.5 (kN); Corresponding J(RP)= 0.36; J(RX) = 0.46

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.45	20.63	2854.5	3244.1	3260.6	15.466	0.544	0.900	28.5	3480.7

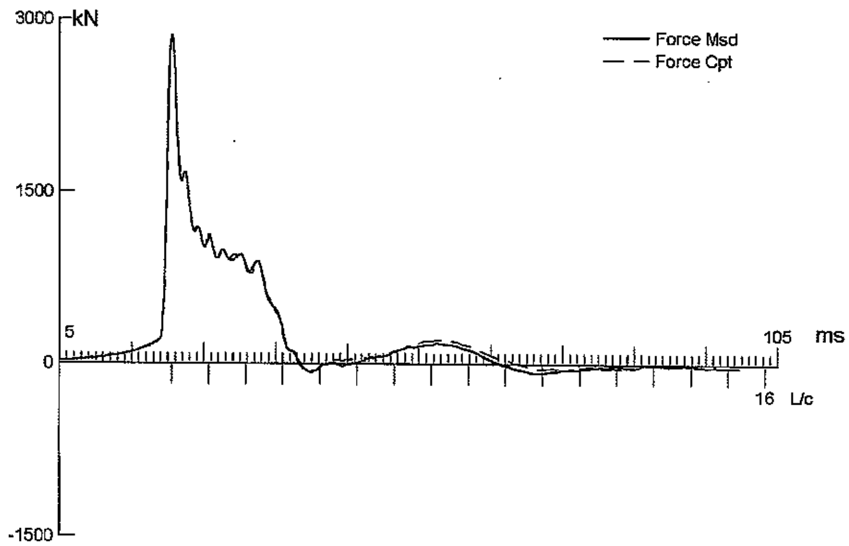
PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206874.8	77.300	1.220
31.50	158.70	206874.8	77.300	1.220

Toe Area 0.093 m^2

Top Segment Length 1.02 m, Top Impedance 640.86 kN/m/s

Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 12.3 ms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1500
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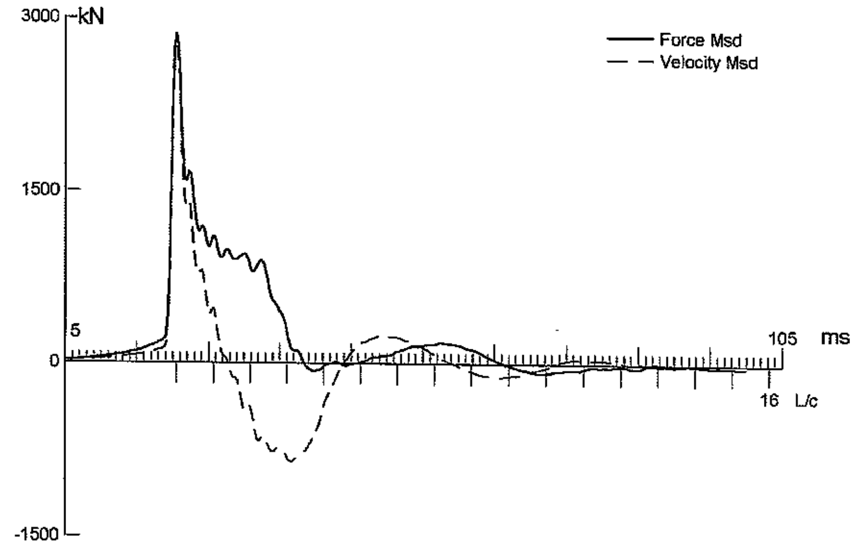
ms
16
L/c
Force Msd
Force Cpt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L/c
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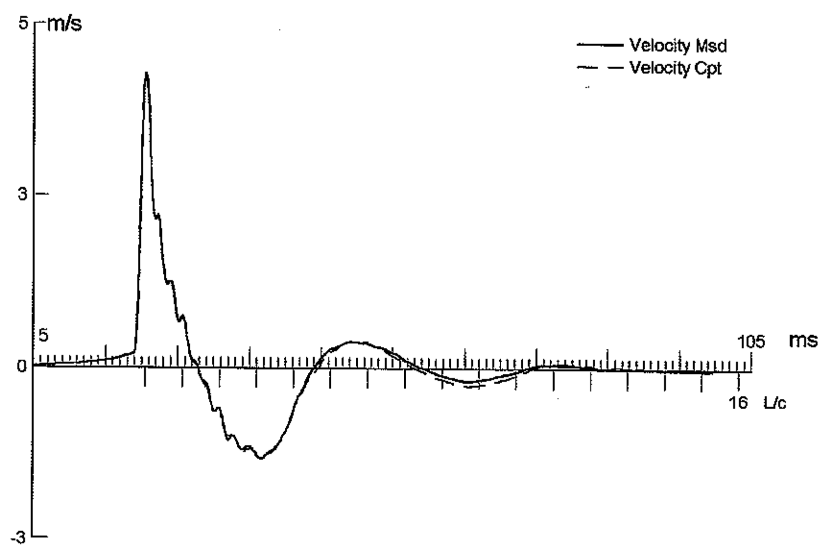
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m/s
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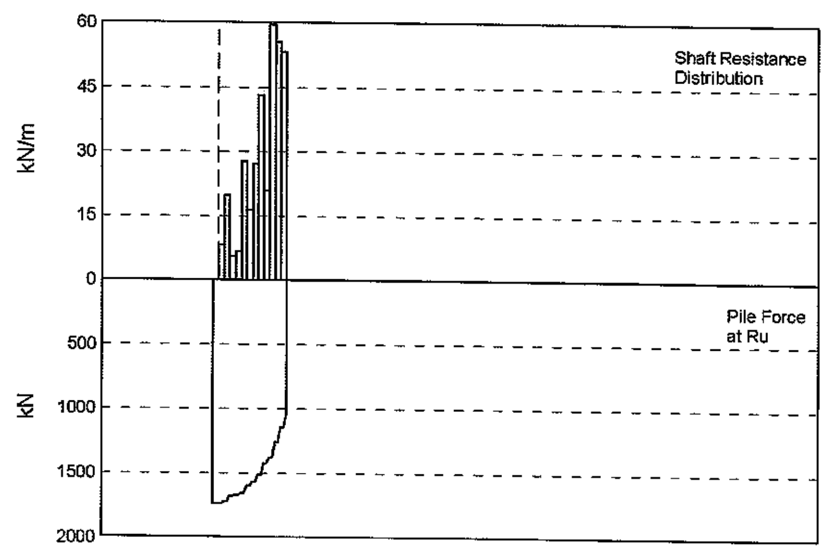
-3
5
105
ms
16
L/c
Velocity Msd
Velocity Cpt


60
45
30
15
0
500
1000
1500
2000
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1737.4; along Shaft 700.2; at Toe 1037.2 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				1737.4				
1	4.1	1.6	16.9	1720.5	16.9	10.72	8.78	1.040
2	6.1	3.6	40.3	1680.2	57.2	19.77	16.20	1.040
3	8.2	5.7	11.2	1669.0	68.4	5.49	4.50	1.040
4	10.2	7.7	13.5	1655.5	81.9	6.62	5.43	1.040
5	12.2	9.7	56.5	1599.0	138.4	27.72	22.72	1.040
6	14.3	11.8	33.0	1566.0	171.4	16.19	13.27	1.040
7	16.3	13.8	55.3	1510.7	226.7	27.13	22.24	1.040
8	18.3	15.8	87.8	1422.9	314.5	43.07	35.30	1.040
9	20.4	17.9	42.5	1380.4	357.0	20.85	17.09	1.040
10	22.4	19.9	121.5	1258.9	478.5	59.60	48.86	1.040
11	24.5	22.0	113.3	1145.6	591.8	55.58	45.56	1.040
12	26.5	24.0	108.4	1037.2	700.2	53.18	43.59	1.040
Avg. Shaft			58.3			29.17	23.91	1.040
Toe			1037.2				11149.69	0.505

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	3.804	4.336
Case Damping Factor		1.136	0.817
Unloading Quake	(% of loading quake)	67	111
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			0.394
Soil Plug Weight	(kN)		0.68

CAPWAP match quality = 1.46 (Wave Up Match) ; RSA = 0
Observed: final set = 1.200 mm; blow count = 833 b/m
Computed: final set = 2.199 mm; blow count = 455 b/m

max. Top Comp. Stress = 179.7 MPa (T= 20.9 ms, max= 1.026 x Top)
max. Comp. Stress = 184.4 MPa (Z= 4.1 m, T= 21.5 ms)
max. Tens. Stress = -13.60 MPa (Z= 12.2 m, T= 42.6 ms)
max. Energy (EMX) = 24.48 kJ; max. Measured Top Displ. (DMX)=14.50 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2851.6	-100.1	179.7	-6.31	24.48	4.3	14.156
2	2.0	2865.3	-126.5	180.5	-7.97	24.31	4.3	13.839
4	4.1	2926.0	-178.7	184.4	-11.26	23.98	4.2	13.200
6	6.1	2910.5	-189.3	183.4	-11.93	22.91	4.0	12.539
8	8.2	2738.9	-175.5	172.6	-11.06	20.93	4.0	11.828
10	10.2	2746.5	-202.5	173.1	-12.76	20.01	3.9	11.037
11	11.2	2737.6	-197.4	172.5	-12.44	19.29	3.8	10.631
12	12.2	2786.1	-215.9	175.6	-13.60	19.05	3.7	10.247
13	13.3	2563.5	-158.2	161.5	-9.97	17.13	3.6	9.867
14	14.3	2611.1	-179.5	164.5	-11.31	16.87	3.6	9.472
15	15.3	2523.3	-152.3	159.0	-9.60	15.71	3.5	9.059
16	16.3	2600.0	-170.5	163.8	-10.74	15.43	3.4	8.641
17	17.3	2438.4	-113.0	153.6	-7.12	13.90	3.2	8.259
18	18.3	2494.5	-127.6	157.2	-8.04	13.63	3.2	7.853
19	19.4	2202.9	-34.8	138.8	-2.19	11.61	3.1	7.445
20	20.4	2290.6	-54.6	144.3	-3.44	11.31	3.0	7.017
21	21.4	2226.9	-20.5	140.3	-1.29	10.29	2.8	6.590
22	22.4	2313.5	-34.0	145.8	-2.14	10.01	2.7	6.171
23	23.4	2027.0	0.0	127.7	0.00	8.13	2.5	5.791
24	24.5	2116.8	0.0	133.4	0.00	7.88	2.3	5.402
25	25.5	1813.6	0.0	114.3	0.00	6.45	2.3	5.039
26	26.5	1818.5	0.0	114.6	0.00	5.56	2.3	4.681
Absolute	4.1			184.4			(T =	21.5 ms)
	12.2				-13.60		(T =	42.6 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3454.5	3232.6	3010.7	2788.8	2566.9	2345.0	2123.2	1901.3	1679.4	1457.5
RX	3454.5	3232.6	3010.7	2788.8	2566.9	2345.0	2123.2	1901.3	1704.1	1628.8
RU	3595.9	3388.2	3180.5	2972.7	2765.0	2557.3	2349.5	2141.8	1934.0	1726.3

RAU = 256.3 (kN); RA2 = 1797.3 (kN)

Current CAPWAP Ru = 1737.4 (kN); Corresponding J(RP)= 0.77; J(RX) = 0.78

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.34	20.69	2779.4	2893.9	2893.9	14.504	1.204	1.200	24.9	3177.5

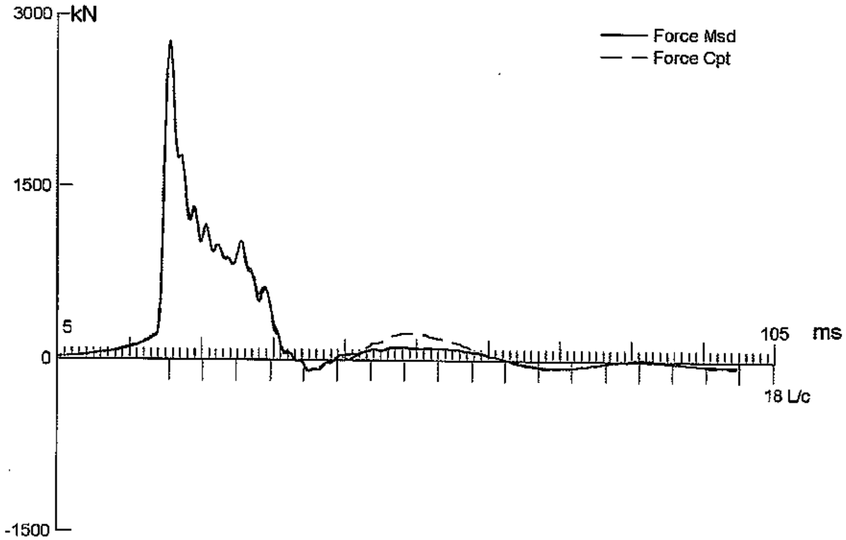
PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206874.8	77.300	1.220
26.50	158.70	206874.8	77.300	1.220

Toe Area 0.093 m^2

Top Segment Length 1.02 m, Top Impedance 640.86 kN/m/s

Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 5123.0 m/s, 2L/c 10.3 ms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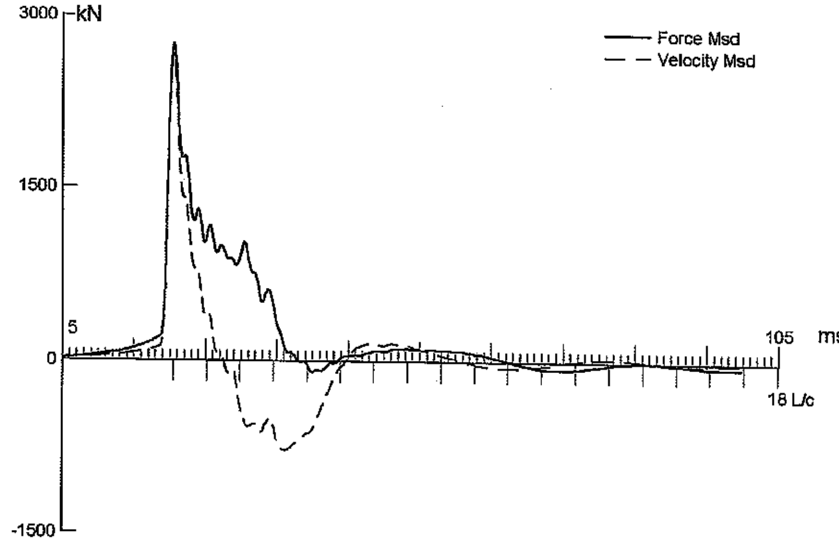
1500
0
-1500
5
105 ms
18 L/c
Force Msd
Force Cpt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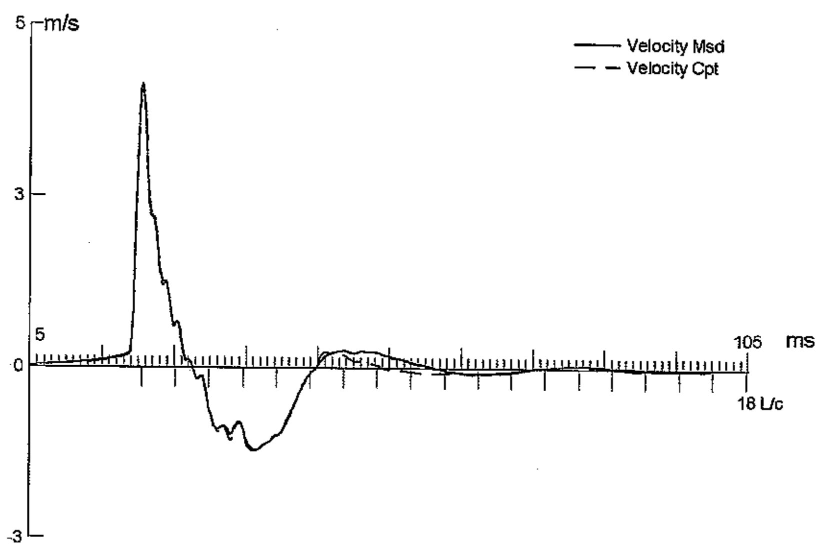
-1500
5
105 ms
18 L/c
Force Msd
Velocity Msd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3
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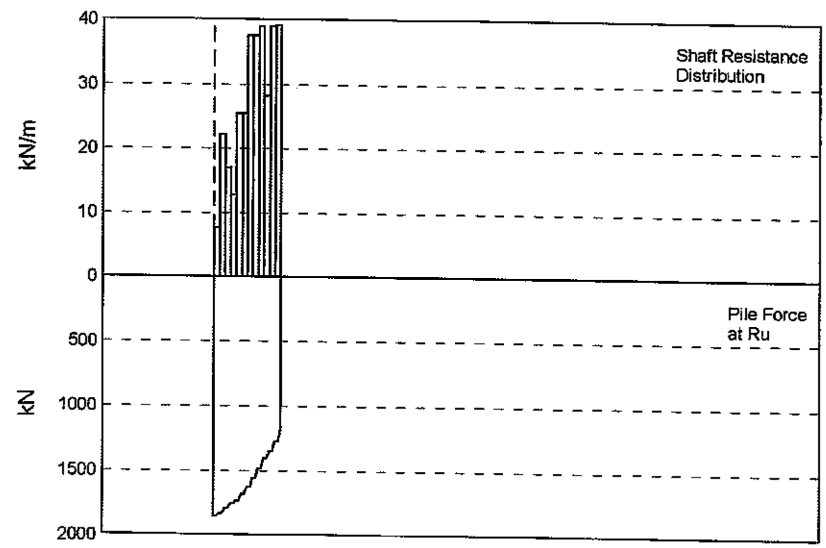
-3
5
105 ms
18 L/c
Velocity Msd
Velocity Cpt


40
30
20
10
0
500
1000
1500
2000
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]-Pile 11 (RSTRK)
August 28, 2008; Blow: 9
[redacted]

Test: 28-Aug-2008 11:55:
CAPWAP(R) 2006-2
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1850.0; along Shaft 661.5; at Toe 1188.5 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				1850.0					
1	2.0	2.0	15.1	1834.9	15.1	7.55	6.19	0.656	1.534
2	4.0	4.0	44.3	1790.6	59.4	22.15	18.16	0.656	1.536
3	6.0	6.0	33.8	1756.8	93.2	16.90	13.85	0.656	1.536
4	8.0	8.0	25.3	1731.5	118.5	12.65	10.37	0.656	1.536
5	10.0	10.0	50.9	1680.6	169.4	25.45	20.86	0.656	1.536
6	12.0	12.0	50.9	1629.7	220.3	25.45	20.86	0.656	1.536
7	14.0	14.0	75.1	1554.6	295.4	37.55	30.78	0.656	1.536
8	16.0	16.0	75.1	1479.5	370.5	37.55	30.78	0.656	1.536
9	18.0	18.0	78.0	1401.5	448.5	39.00	31.97	0.656	1.536
10	20.0	20.0	56.6	1344.9	505.1	28.30	23.20	0.656	1.536
11	22.0	22.0	78.0	1266.9	583.1	39.00	31.97	0.656	1.536
12	24.0	24.0	78.4	1188.5	661.5	39.20	32.13	0.656	1.412
Avg. Shaft			55.1			27.56	22.59	0.656	1.521
Toe			1188.5				12776.14	0.440	3.392

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.677	0.816
Damping Type		Smith	
Unloading Quake	(% of loading quake)	30	49
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	10	
Resistance Gap (included in Toe Quake) (mm)			0.459

CAPWAP match quality	=	2.15	(Wave Up Match) ; RSA = 0
Observed: final set	=	1.400 mm;	blow count = 714 b/m
Computed: final set	=	1.369 mm;	blow count = 730 b/m
max. Top Comp. Stress	=	176.7 MPa	(T= 20.9 ms, max= 1.013 x Top)
max. Comp. Stress	=	178.9 MPa	(Z= 4.0 m, T= 21.5 ms)
max. Tens. Stress	=	-20.74 MPa	(Z= 14.0 m, T= 42.9 ms)
max. Energy (EMX)	=	23.95 kJ;	max. Measured Top Displ. (DMX)=14.14 mm

Analysis: 13-Jan-2009

[redacted] Pile 11 (RSTRK)
August 28, 2008; Blow: 9
[redacted]

Test: 28-Aug-2008 11:55:
CAPWAP (R) 2006-2
OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2804.2	-102.2	176.7	-6.44	23.95	4.1	13.440
2	2.0	2832.5	-125.9	178.5	-7.93	23.80	4.0	13.141
4	4.0	2839.7	-173.1	178.9	-10.90	23.04	3.9	12.556
6	6.0	2732.2	-202.5	172.2	-12.76	21.45	3.8	11.964
7	7.0	2640.9	-216.8	166.4	-13.66	20.38	3.8	11.679
8	8.0	2668.0	-240.6	168.1	-15.16	20.24	3.7	11.384
9	9.0	2628.0	-243.1	165.6	-15.32	19.42	3.7	11.067
10	10.0	2657.0	-255.1	167.4	-16.08	19.21	3.6	10.712
11	11.0	2535.8	-273.1	159.8	-17.21	17.77	3.5	10.330
12	12.0	2570.3	-295.0	162.0	-18.59	17.49	3.5	9.918
13	13.0	2472.8	-302.3	155.8	-19.05	16.17	3.4	9.548
14	14.0	2508.1	-329.2	158.0	-20.74	15.94	3.3	9.172
15	15.0	2336.4	-317.6	147.2	-20.01	14.22	3.2	8.775
16	16.0	2373.6	-318.4	149.6	-20.06	13.90	3.2	8.330
17	17.0	2205.1	-293.9	138.9	-18.52	12.29	3.1	7.886
18	18.0	2240.1	-294.3	141.2	-18.55	11.96	3.0	7.438
19	19.0	2053.6	-267.3	129.4	-16.84	10.49	2.9	7.016
20	20.0	2087.6	-267.0	131.5	-16.82	10.20	2.9	6.595
21	21.0	1954.2	-248.6	123.1	-15.67	9.20	2.8	6.188
22	22.0	1993.9	-249.2	125.6	-15.71	8.92	2.8	5.775
23	23.0	1957.4	-222.0	123.3	-13.99	7.85	2.6	5.385
24	24.0	2145.9	-221.3	135.2	-13.95	7.41	2.3	4.996
Absolute	4.0			178.9			(T =	21.5 ms)
	14.0				-20.74		(T =	42.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3350.8	3145.7	2940.6	2735.4	2530.3	2325.1	2120.0	1914.8	1709.7	1504.6
RX	3350.8	3145.7	2940.6	2739.4	2547.2	2354.9	2162.7	1987.3	1828.0	1688.2
RU	3527.9	3340.5	3153.1	2965.6	2778.2	2590.8	2403.3	2215.9	2028.5	1841.0

RAU = 651.0 (kN); RA2 = 1611.5 (kN)

Current CAPWAP Ru = 1850.0 (kN); Corresponding J(RP)= 0.73; J(RX) = 0.79

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.14	20.69	2650.9	2751.4	2751.4	14.144	3.613	1.400	24.2	3111.1

PILE PROFILE AND PILE MODEL

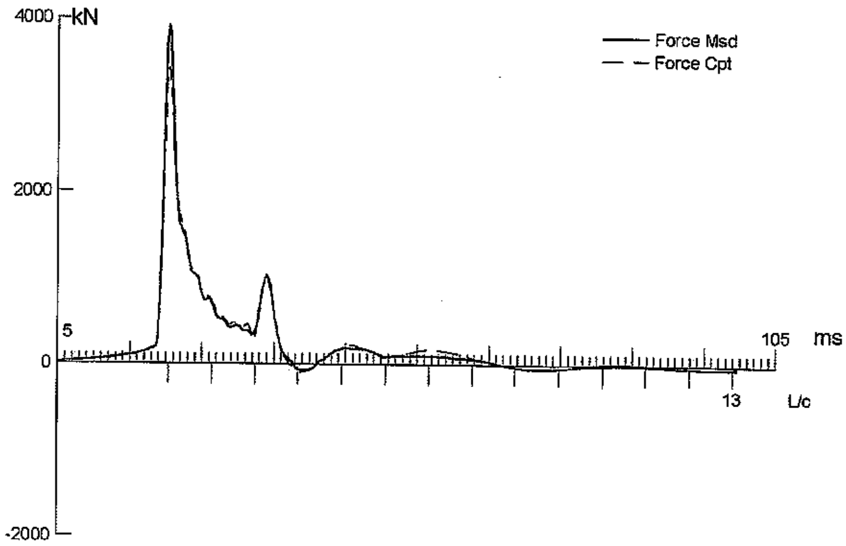
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206874.8	77.300	1.220
24.00	158.70	206874.8	77.300	1.220

Toe Area 0.093 m^2

Top Segment Length 1.00 m, Top Impedance 640.86 kN/m/s

Analysis: 13-Jan-2009

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 9.4 ms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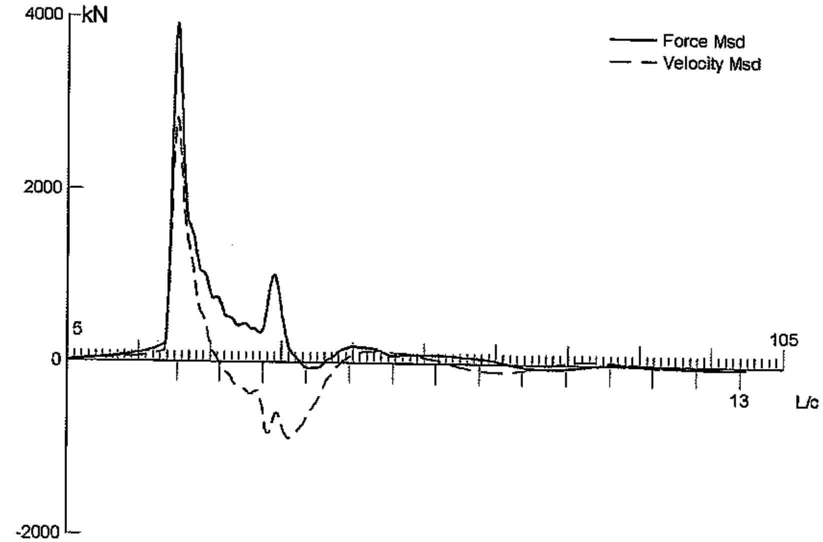
kN
2000
0
-2000
5
105
ms
13
L/c
Force Msd
Force Cpt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2000
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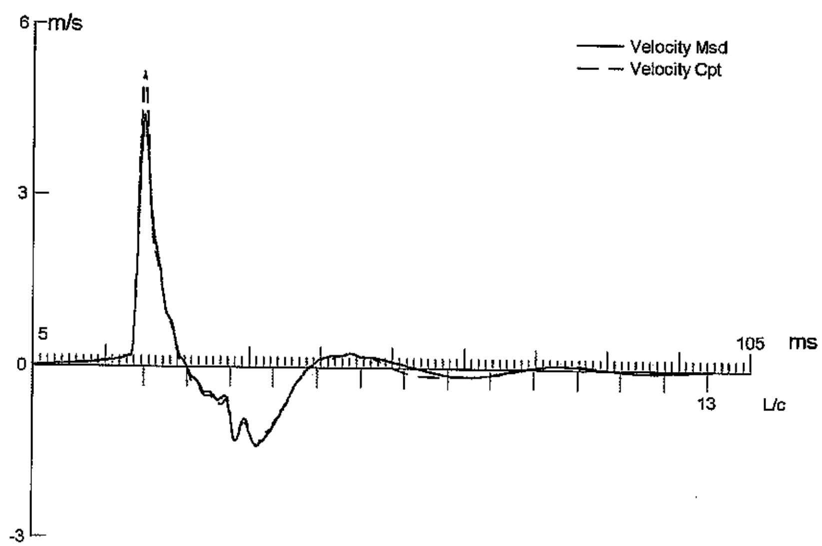
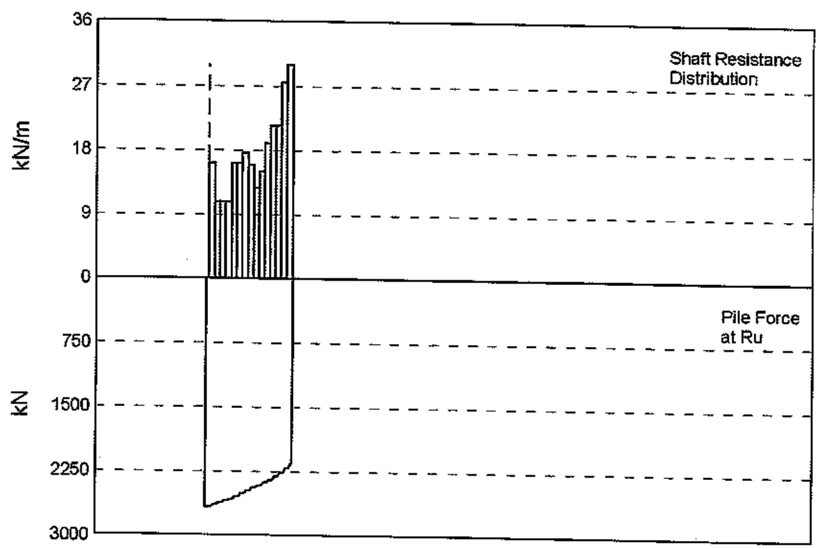
105
ms
13
L/c
Force Msd
Velocity Msd


6
m/s
3
0
-3
5
105
ms
13
L/c
Velocity Msd
Velocity Cpt


36
27
18
9
0
750
1500
2250
3000
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]

[redacted]-Pile 12 B (RSTRK) Test: 02-Sep-2008 12:53:
September 2, 2008; Blow: 42 CAPWAP(R) 2006-2
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2667.9; along Shaft 523.1; at Toe 2144.8 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2667.9				
1	3.0	1.5	32.4	2635.5	32.4	21.60	17.70	1.178
2	5.0	3.5	21.5	2614.0	53.9	10.75	8.81	1.178
3	7.0	5.5	21.5	2592.5	75.4	10.75	8.81	1.178
4	9.0	7.5	21.5	2571.0	96.9	10.75	8.81	1.178
5	11.0	9.5	32.4	2538.6	129.3	16.20	13.28	1.178
6	13.0	11.5	32.4	2506.2	161.7	16.20	13.28	1.178
7	15.0	13.5	35.2	2471.0	196.9	17.60	14.43	1.178
8	17.0	15.5	31.9	2439.1	228.8	15.95	13.07	1.178
9	19.0	17.5	25.4	2413.7	254.2	12.70	10.41	1.178
10	21.0	19.5	29.9	2383.8	284.1	14.95	12.25	1.178
11	23.0	21.5	37.8	2346.0	321.9	18.90	15.49	1.178
12	25.0	23.5	43.1	2302.9	365.0	21.55	17.66	1.178
13	27.0	25.5	43.1	2259.8	408.1	21.55	17.66	1.178
14	29.0	27.5	55.0	2204.8	463.1	27.50	22.54	1.178
15	31.0	29.5	60.0	2144.8	523.1	30.00	24.59	1.178
Avg. Shaft			34.9			17.73	14.53	1.178
Toe			2144.8				23056.17	0.426

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.463	4.732
Case Damping Factor		0.962	1.426
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	81	92
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			0.068
Soil Plug Weight	(kN)		1.81

CAPWAP match quality = 1.59 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 1.927 mm; blow count = 519 b/m

max. Top Comp. Stress = 217.8 MPa (T= 20.7 ms, max= 1.026 x Top)
max. Comp. Stress = 223.6 MPa (Z= 3.0 m, T= 21.1 ms)
max. Tens. Stress = -10.09 MPa (Z= 3.0 m, T= 39.0 ms)
max. Energy (EMX) = 30.33 kJ; max. Measured Top Displ. (DMX)=13.71 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3456.4	-120.1	217.8	-7.57	30.33	5.1	14.210
2	2.0	3515.8	-142.8	221.5	-9.00	30.24	5.0	13.976
4	4.0	3367.6	-120.0	212.2	-7.56	28.24	4.9	13.521
6	6.0	3288.5	-123.8	207.2	-7.80	26.93	4.8	13.078
8	8.0	3212.9	-110.9	202.5	-6.99	25.68	4.7	12.632
10	10.0	3160.3	-104.3	199.1	-6.57	24.48	4.5	12.177
12	12.0	3050.2	-69.2	192.2	-4.36	22.88	4.4	11.721
14	14.0	2948.1	-31.4	185.8	-1.98	21.36	4.2	11.257
16	16.0	2829.3	-5.7	178.3	-0.36	19.81	4.1	10.774
18	18.0	2721.3	0.0	171.5	0.00	18.52	4.0	10.364
20	20.0	2658.5	0.0	167.5	0.00	17.46	3.8	9.875
21	21.0	2700.3	0.0	170.2	0.00	17.30	3.8	9.558
22	22.0	2593.4	0.0	163.4	0.00	16.19	3.7	9.224
23	23.0	2636.2	0.0	166.1	0.00	15.99	3.6	8.873
24	24.0	2501.5	0.0	157.6	0.00	14.72	3.5	8.500
25	25.0	2547.4	0.0	160.5	0.00	14.42	3.4	8.081
26	26.0	2393.1	0.0	150.8	0.00	13.00	3.3	7.609
27	27.0	2464.7	0.0	155.3	0.00	12.56	3.2	7.089
28	28.0	2311.7	0.0	145.7	0.00	11.16	3.1	6.545
29	29.0	2293.8	0.0	144.5	0.00	10.62	3.2	5.968
30	30.0	2172.7	0.0	136.9	0.00	9.09	3.3	5.377
31	31.0	2544.3	0.0	160.3	0.00	8.43	3.0	4.749
Absolute	3.0			223.6			(T =	21.1 ms)
	3.0				-10.09		(T =	39.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3843.9	3554.1	3264.3	2974.5	2684.7	2394.9	2105.1	1815.3	1525.5	1235.7
RX	3846.9	3573.6	3300.3	3027.1	2753.8	2490.4	2255.5	2069.2	1921.2	1800.1
RU	4203.2	3949.3	3695.4	3441.6	3187.7	2933.8	2680.0	2426.1	2172.2	1918.3

RAU = 524.1 (kN); RA2 = 1677.6 (kN)

Current CAPWAP Ru = 2667.9 (kN); Corresponding J(RP)= 0.41; J(RX) = 0.43

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.41	20.50	2825.6	3916.3	3916.3	13.707	2.550	1.000	30.1	4091.8

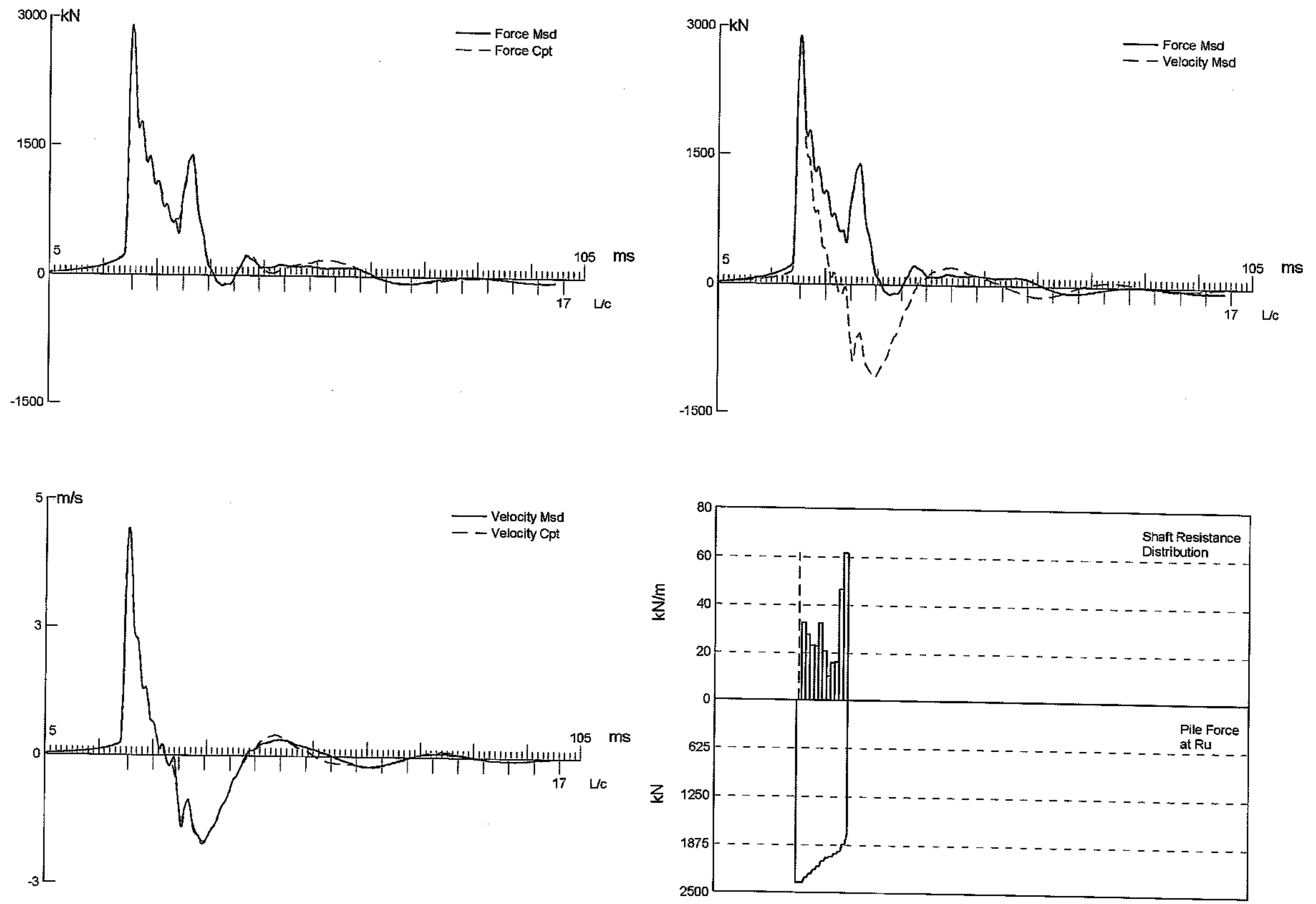
PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206874.8	77.300	1.220
31.00	158.70	206874.8	77.300	1.220

Toe Area 0.093 m²

Top Segment Length 1.00 m, Top Impedance 640.86 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 12.1 ms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1500
0
-1500
5
105 ms
17 L/c
Force Msd
Force Cpt
Force Msd
Velocity Msd
5 m/s
3
-3
Velocity Msd
Velocity Cpt
80
60
40
20
625
1250
1875
2500
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2363.3; along Shaft 626.2; at Toe 1737.1 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2363.3				
1	4.1	2.6	65.8	2297.5	65.8	25.07	20.55	0.673
2	6.1	4.7	55.6	2241.9	121.4	27.42	22.48	0.673
3	8.1	6.7	47.1	2194.8	168.5	23.23	19.04	0.673
4	10.1	8.7	46.1	2148.7	214.6	22.74	18.64	0.673
5	12.2	10.7	65.2	2083.5	279.8	32.16	26.36	0.673
6	14.2	12.8	41.8	2041.7	321.6	20.62	16.90	0.673
7	16.2	14.8	20.1	2021.6	341.7	9.91	8.13	0.673
8	18.2	16.8	32.1	1989.5	373.8	15.83	12.98	0.673
9	20.3	18.8	32.8	1956.7	406.6	16.18	13.26	0.673
10	22.3	20.9	94.3	1862.4	500.9	46.51	38.12	0.673
11	24.3	22.9	125.3	1737.1	626.2	61.80	50.66	0.673
Avg. Shaft			56.9			27.34	22.41	0.673
Toe			1737.1				18673.47	0.411

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	4.659	4.786
Case Damping Factor		0.658	1.114
Damping Type		Smith	
Unloading Quake	(% of loading quake)	64	135
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	68	
Resistance Gap (included in Toe Quake) (mm)			1.308

CAPWAP match quality = 1.79 (Wave Up Match) ; RSA = 0
Observed: final set = 0.750 mm; blow count = 1333 b/m
Computed: final set = 1.475 mm; blow count = 678 b/m

max. Top Comp. Stress = 183.2 MPa (T= 20.8 ms, max= 1.038 x Top)
max. Comp. Stress = 190.2 MPa (Z= 4.1 m, T= 21.4 ms)
max. Tens. Stress = -14.35 MPa (Z= 4.1 m, T= 38.0 ms)
max. Energy (EMX) = 25.64 kJ; max. Measured Top Displ. (DMX)=14.45 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2907.9	-150.9	183.2	-9.51	25.64	4.4	14.190
2	2.0	2924.1	-181.3	184.3	-11.42	25.56	4.4	13.968
4	4.1	3018.8	-227.8	190.2	-14.35	25.37	4.2	13.503
6	6.1	2888.9	-176.3	182.0	-11.11	23.35	4.1	13.000
7	7.1	2747.8	-116.3	173.1	-7.33	21.81	4.1	12.755
8	8.1	2786.9	-131.5	175.6	-8.29	21.71	4.0	12.509
9	9.1	2676.2	-79.5	168.6	-5.01	20.45	4.0	12.244
10	10.1	2716.1	-82.4	171.1	-5.19	20.28	3.9	11.931
11	11.2	2620.3	-46.0	165.1	-2.90	19.04	3.9	11.577
12	12.2	2664.9	-73.4	167.9	-4.62	18.81	3.8	11.207
13	13.2	2530.9	-53.1	159.5	-3.35	17.25	3.8	10.855
14	14.2	2554.2	-91.6	160.9	-5.77	17.01	3.8	10.460
15	15.2	2475.9	-89.3	156.0	-5.63	15.89	3.8	10.024
16	16.2	2488.5	-120.1	156.8	-7.57	15.49	3.7	9.537
17	17.2	2458.0	-130.3	154.9	-8.21	14.73	3.7	9.033
18	18.2	2476.2	-155.3	156.0	-9.79	14.33	3.7	8.537
19	19.3	2424.3	-151.0	152.8	-9.52	13.41	3.7	8.014
20	20.3	2445.8	-170.4	154.1	-10.74	12.89	3.6	7.436
21	21.3	2409.5	-161.0	151.8	-10.14	11.90	3.6	6.840
22	22.3	2498.3	-187.5	157.4	-11.81	11.28	3.5	6.221
23	23.3	2488.9	-130.4	156.8	-8.22	9.90	3.3	5.627
24	24.3	2788.2	-150.9	175.7	-9.51	9.46	2.9	5.039
Absolute	4.1			190.2			(T =	21.4 ms)
	4.1				-14.35		(T =	38.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3684.0	3479.9	3275.9	3071.8	2867.8	2663.8	2459.7	2255.7	2051.6	1847.6
RX	3684.0	3487.7	3304.0	3120.4	2936.8	2753.1	2593.6	2440.0	2291.2	2172.1
RU	3913.7	3732.7	3551.6	3370.5	3189.5	3008.4	2827.3	2646.3	2465.2	2284.1

RAU = 1007.0 (kN); RA2 = 2073.9 (kN)

Current CAPWAP Ru = 2363.3 (kN); Corresponding J(RP)= 0.65; J(RX) = 0.75

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.44	20.58	2843.0	2881.4	2881.4	14.449	1.994	0.750	25.7	3387.2

PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206842.7	77.287	1.220
24.33	158.70	206842.7	77.287	1.220

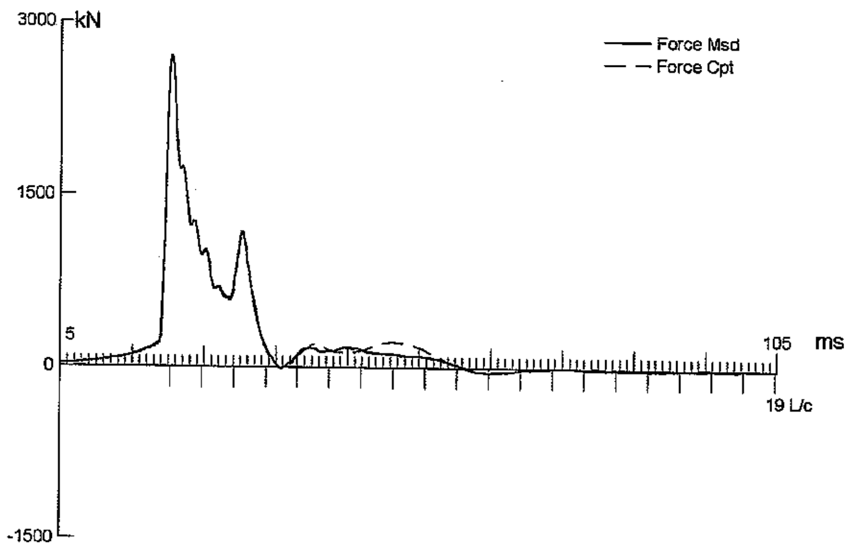
Toe Area 0.093 m^2

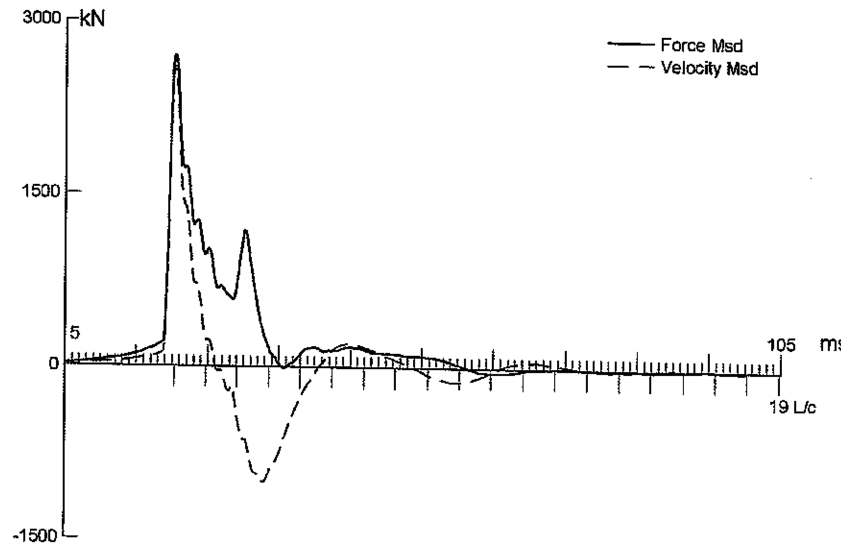
Top Segment Length 1.01 m, Top Impedance 640.75 kN/m/s

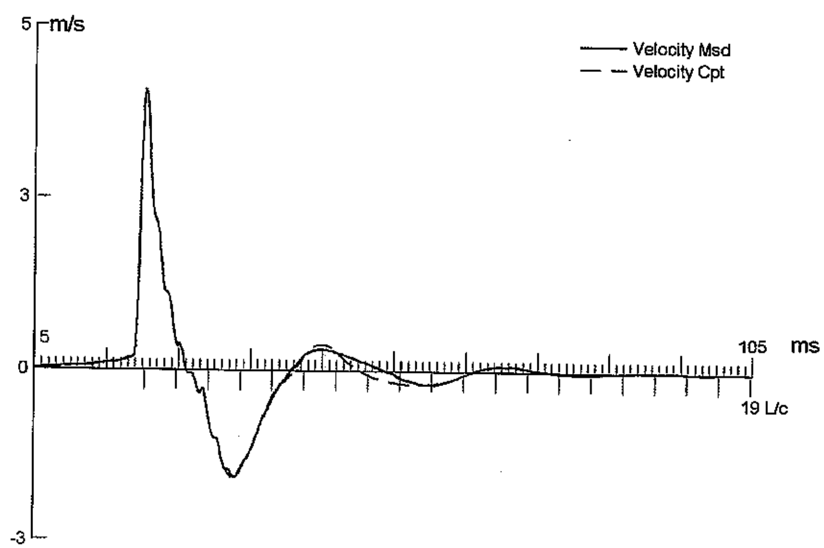
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 9.5 ms

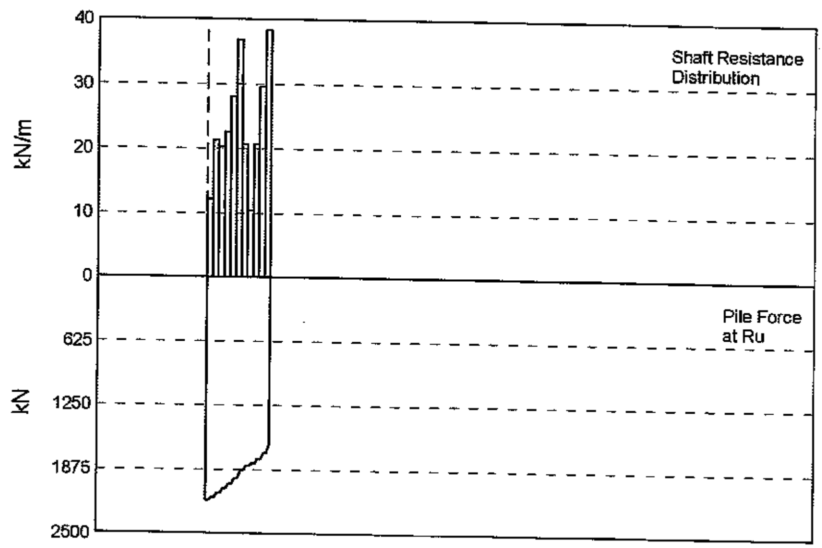
[redacted]-15; September 19, 2008; Blow: 22 (Test: 19-Sep-2008 16:21:)

13-Jan-2009
CAPWAP(R) 2006-2









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CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2170.7; along Shaft 539.7; at Toe 1631.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2170.7				
1	2.1	1.8	25.1	2145.6	25.1	13.85	11.36	0.775
2	4.1	3.9	43.9	2101.7	69.0	21.19	17.37	0.775
3	6.2	6.0	41.8	2059.9	110.8	20.18	16.54	0.775
4	8.3	8.0	46.6	2013.3	157.4	22.49	18.44	0.775
5	10.4	10.1	58.1	1955.2	215.5	28.04	22.99	0.775
6	12.4	12.2	76.5	1878.7	292.0	36.92	30.27	0.775
7	14.5	14.2	42.8	1835.9	334.8	20.66	16.93	0.775
8	16.6	16.3	21.4	1814.5	356.2	10.33	8.47	0.775
9	18.6	18.4	42.8	1771.7	399.0	20.66	16.93	0.775
10	20.7	20.5	61.2	1710.5	460.2	29.54	24.21	0.775
11	22.8	22.5	79.5	1631.0	539.7	38.37	31.45	0.775
Avg. Shaft			49.1			23.95	19.64	0.775
Toe			1631.0				17532.92	0.338

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.981	4.382
Case Damping Factor		0.653	0.860
Damping Type		Smith	
Unloading Quake	(% of loading quake)	78	114
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	80	
Resistance Gap (included in Toe Quake)	(mm)		0.581

CAPWAP match quality	=	1.58	(Wave Up Match) ; RSA = 0
Observed: final set	=	0.750 mm;	blow count = 1333 b/m
Computed: final set	=	1.689 mm;	blow count = 592 b/m
max. Top Comp. Stress	=	171.9 MPa	(T= 20.6 ms, max= 1.012 x Top)
max. Comp. Stress	=	174.0 MPa	(Z= 2.1 m, T= 20.8 ms)
max. Tens. Stress	=	-6.20 MPa	(Z= 4.1 m, T= 36.4 ms)
max. Energy (EMX)	=	22.15 kJ;	max. Measured Top Displ. (DMX)=12.72 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2727.6	-44.6	171.9	-2.81	22.15	4.0	12.563
2	2.1	2761.0	-76.5	174.0	-4.82	22.05	4.0	12.324
3	3.1	2697.6	-68.7	170.0	-4.33	21.21	3.9	12.090
4	4.1	2738.9	-98.4	172.6	-6.20	21.11	3.8	11.852
5	5.2	2602.9	-58.4	164.0	-3.68	19.79	3.8	11.619
6	6.2	2641.7	-77.8	166.5	-4.90	19.70	3.7	11.384
7	7.3	2523.7	-36.4	159.0	-2.29	18.52	3.7	11.148
8	8.3	2567.9	-54.1	161.8	-3.41	18.40	3.6	10.881
9	9.3	2443.5	-4.3	154.0	-0.27	17.13	3.5	10.580
10	10.4	2498.3	-15.1	157.4	-0.95	16.95	3.4	10.253
11	11.4	2345.2	0.0	147.8	0.00	15.52	3.4	9.942
12	12.4	2398.9	0.0	151.2	0.00	15.34	3.3	9.614
13	13.5	2172.2	0.0	136.9	0.00	13.59	3.2	9.257
14	14.5	2206.4	0.0	139.0	0.00	13.32	3.2	8.842
15	15.5	2089.8	0.0	131.7	0.00	12.22	3.1	8.394
16	16.6	2119.7	0.0	133.6	0.00	11.89	3.1	7.938
17	17.6	2085.3	0.0	131.4	0.00	11.20	3.1	7.482
18	18.6	2128.7	0.0	134.1	0.00	10.81	3.0	6.978
19	19.7	2038.9	0.0	128.5	0.00	9.78	2.9	6.441
20	20.7	2065.8	0.0	130.2	0.00	9.28	2.9	5.879
21	21.8	2081.3	0.0	131.1	0.00	8.18	2.9	5.336
22	22.8	2323.6	0.0	146.4	0.00	7.78	2.6	4.792
Absolute	2.1			174.0			(T =	20.8 ms)
	4.1				-6.20		(T =	36.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3246.2	3029.7	2813.1	2596.6	2380.0	2163.4	1946.9	1730.3	1513.7	1297.2
RX	3313.0	3106.9	2900.8	2700.7	2515.7	2330.7	2170.1	2063.2	1980.3	1906.0
RU	3460.3	3265.1	3070.0	2874.8	2679.7	2484.5	2289.3	2094.2	1899.0	1703.9

RAU = 874.1 (kN); RA2 = 2003.0 (kN)

Current CAPWAP Ru = 2170.7 (kN); Corresponding J(RP)= 0.50; J(RX) = 0.60

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.15	20.42	2659.3	2752.5	2752.5	12.716	0.680	0.750	22.6	3358.9

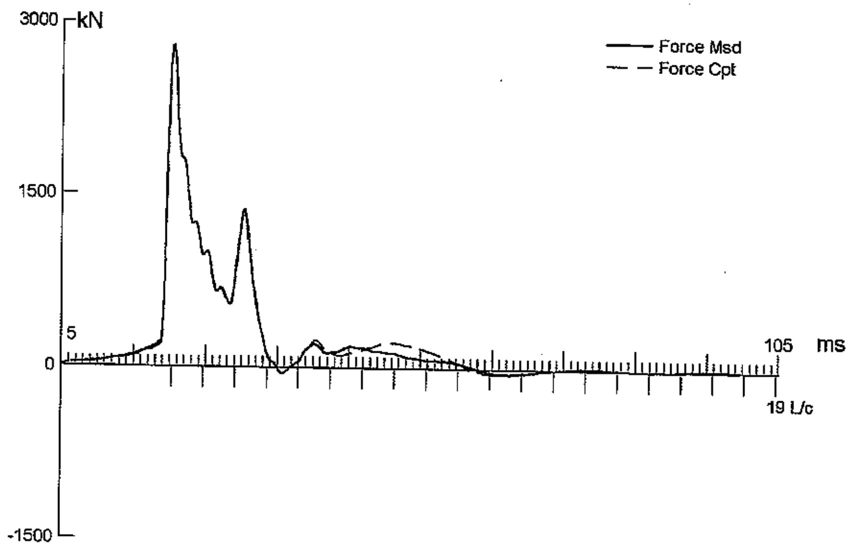
PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206842.7	77.287	1.220
22.79	158.70	206842.7	77.287	1.220

Toe Area 0.093 m^2

Top Segment Length 1.04 m, Top Impedance 640.75 kN/m/s

Pile Damping 1.0 %, Time Incr 0.202 ms, Wave Speed 5123.0 m/s, 2L/c 8.9 ms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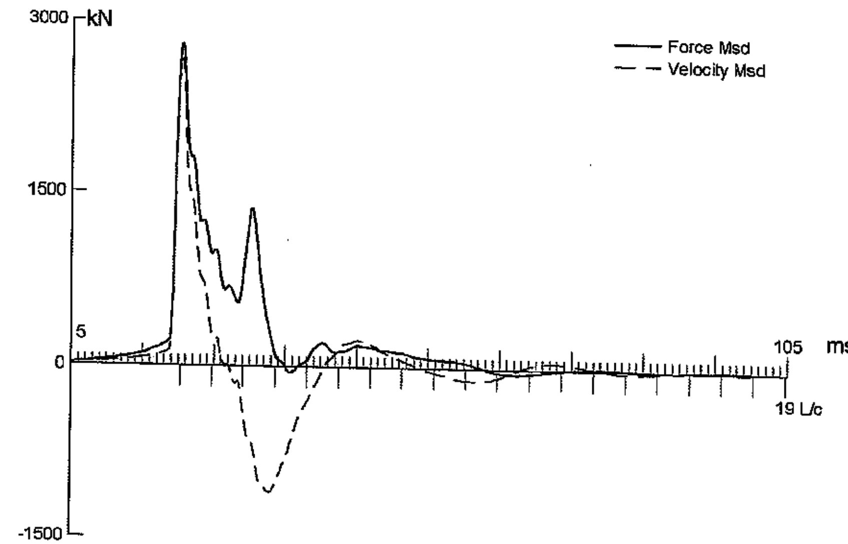
kN
1500
0
-1500
5
105
ms
19 L/c
Force Msd
Force Cpt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kN
1500
0
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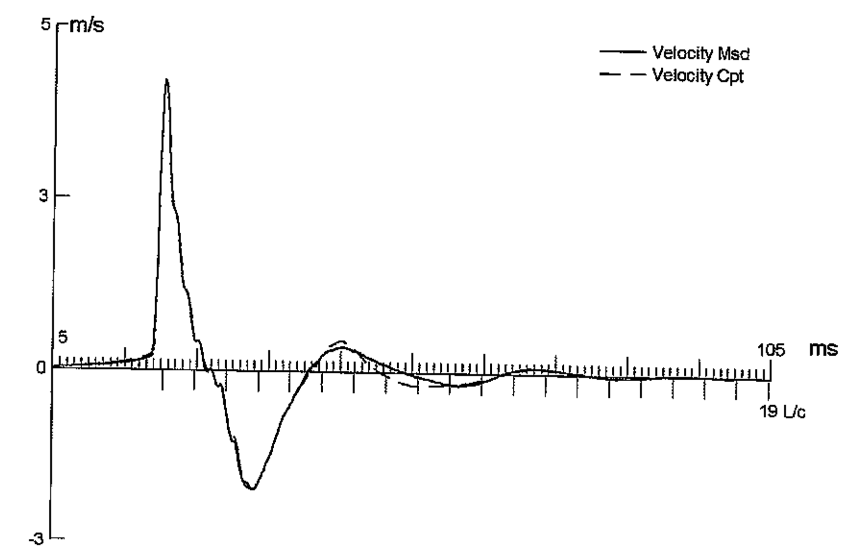
5
105
ms
19 L/c
Force Msd
Velocity Msd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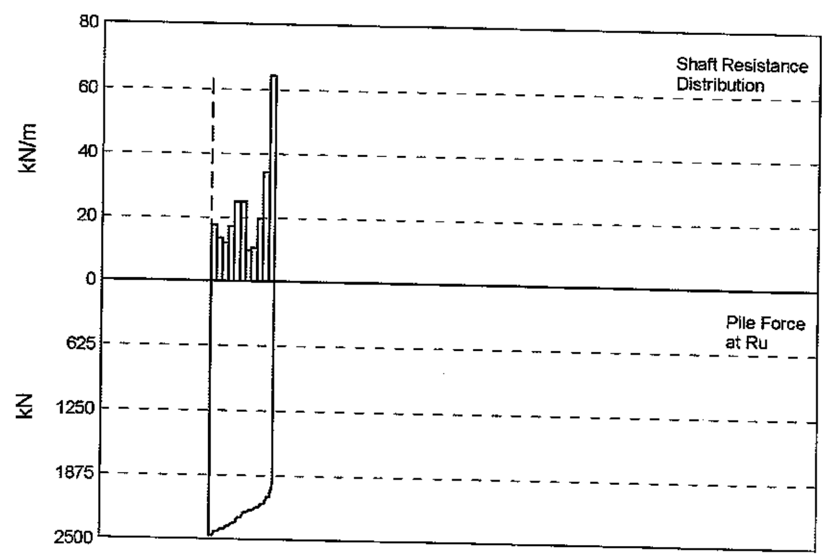
m/s
3
0
-3
5
105
ms
19 L/c
Velocity Msd
Velocity Cpt


80
60
40
20
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2456.9; along Shaft 514.6; at Toe 1942.3 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2456.9					
1	2.1	1.8	36.6	2420.3	36.6	20.20	16.56	1.229	5.047
2	4.1	3.9	27.8	2392.5	64.4	13.42	11.00	1.229	5.048
3	6.2	6.0	24.9	2367.6	89.3	12.02	9.85	1.229	5.048
4	8.3	8.0	35.8	2331.8	125.1	17.28	14.16	1.229	5.048
5	10.4	10.1	51.3	2280.5	176.4	24.76	20.30	1.229	5.048
6	12.4	12.2	51.5	2229.0	227.9	24.86	20.37	1.229	5.048
7	14.5	14.2	19.9	2209.1	247.8	9.61	7.87	1.229	5.048
8	16.6	16.3	22.0	2187.1	269.8	10.62	8.70	1.229	5.048
9	18.6	18.4	40.8	2146.3	310.6	19.69	16.14	1.229	5.048
10	20.7	20.5	70.8	2075.5	381.4	34.17	28.01	1.229	5.048
11	22.8	22.5	133.2	1942.3	514.6	64.29	52.70	1.229	4.961
Avg. Shaft			46.8			22.84	18.72	1.229	5.025
Toe			1942.3				20879.33	0.257	4.920

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.987	0.779
Damping Type		Smith	
Unloading Quake	(% of loading quake)	47	108
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake) (mm)			1.257

CAPWAP match quality = 1.71 (Wave Up Match) ; RSA = 0
Observed: final set = 1.400 mm; blow count = 714 b/m
Computed: final set = 2.108 mm; blow count = 474 b/m

max. Top Comp. Stress = 177.2 MPa (T= 20.8 ms, max= 1.017 x Top)
max. Comp. Stress = 180.2 MPa (Z= 2.1 m, T= 21.0 ms)
max. Tens. Stress = -7.58 MPa (Z= 2.1 m, T= 36.0 ms)
max. Energy (EMX) = 23.93 kJ; max. Measured Top Displ. (DMX)=13.50 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2811.7	-87.4	177.2	-5.51	23.93	4.2	13.372
2	2.1	2860.5	-120.4	180.2	-7.58	23.85	4.1	13.152
3	3.1	2720.3	-67.6	171.4	-4.26	22.31	4.1	12.939
4	4.1	2752.8	-95.9	173.5	-6.04	22.23	4.0	12.721
5	5.2	2658.7	-60.4	167.5	-3.81	21.11	4.0	12.510
6	6.2	2689.6	-80.8	169.5	-5.09	21.04	3.9	12.301
7	7.3	2616.7	-45.5	164.9	-2.87	20.09	3.9	12.073
8	8.3	2658.5	-59.0	167.5	-3.72	19.96	3.8	11.796
9	9.3	2557.5	0.0	161.2	0.00	18.65	3.8	11.473
10	10.4	2609.0	-9.0	164.4	-0.57	18.46	3.7	11.130
11	11.4	2459.7	0.0	155.0	0.00	16.79	3.6	10.806
12	12.4	2502.9	0.0	157.7	0.00	16.58	3.6	10.462
13	13.5	2344.8	0.0	147.8	0.00	14.96	3.6	10.067
14	14.5	2363.9	0.0	149.0	0.00	14.63	3.5	9.620
15	15.5	2312.5	0.0	145.7	0.00	13.80	3.5	9.148
16	16.6	2335.6	-17.4	147.2	-1.10	13.43	3.5	8.666
17	17.6	2288.3	-26.0	144.2	-1.64	12.54	3.4	8.150
18	18.6	2327.6	-58.8	146.7	-3.71	12.01	3.4	7.567
19	19.7	2231.7	-38.3	140.6	-2.42	10.69	3.3	6.949
20	20.7	2226.5	-63.4	140.3	-3.99	10.00	3.3	6.288
21	21.8	2171.1	-3.4	136.8	-0.21	8.45	3.2	5.633
22	22.8	2548.1	-23.3	160.6	-1.47	7.64	2.9	4.971
Absolute	2.1			180.2			(T =	21.0 ms)
	2.1				-7.58		(T =	36.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3350.7	3125.8	2901.0	2676.1	2451.2	2226.4	2001.5	1776.7	1551.8	1327.0
RX	3424.1	3209.8	2995.4	2800.1	2609.9	2447.5	2332.7	2247.4	2173.6	2109.8
RU	3518.8	3310.8	3102.8	2894.7	2686.7	2478.6	2270.6	2062.6	1854.5	1646.5

RAU = 833.8 (kN); RA2 = 2161.3 (kN)

Current CAPWAP Ru = 2456.9 (kN); Corresponding J(RP)= 0.40; J(RX) = 0.49

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.30	20.42	2755.5	2843.7	2843.7	13.498	1.282	1.400	24.4	3280.9

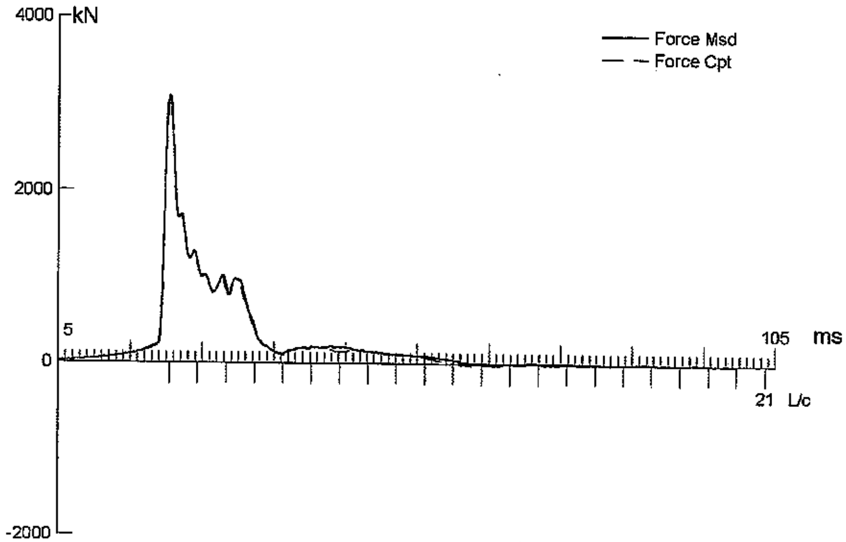
PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206842.7	77.287	1.220
22.79	158.70	206842.7	77.287	1.220

Toe Area 0.093 m^2

Top Segment Length 1.04 m, Top Impedance 640.75 kN/m/s

Pile Damping 1.0 %, Time Incr 0.202 ms, Wave Speed 5123.0 m/s, 2L/c 8.9 ms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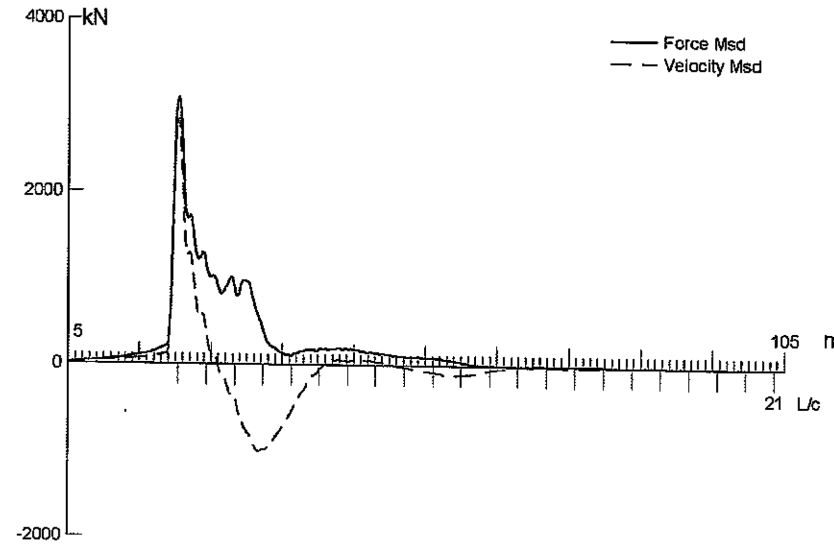
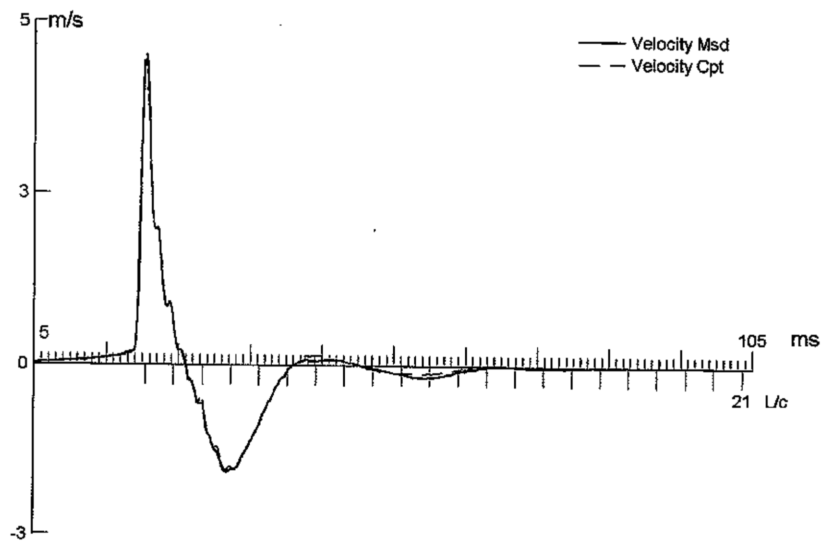
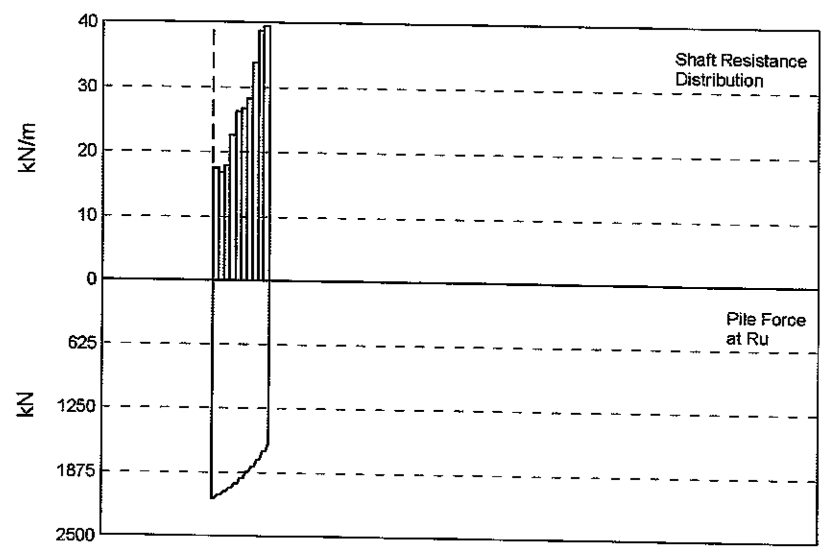
2000
0
-2000
5
105 ms
21 L/c
Force Msd
Force Cpt


4000 kN
2000
0
-2000
5
105 ms
21 L/c
Force Msd
Velocity Msd


5 m/s
3
0
-3
5
105 ms
21 L/c
Velocity Msd
Velocity Cpt


40
30
20
10
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2129.6; along Shaft 543.5; at Toe 1586.1 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2129.6					
1	2.0	1.6	35.4	2094.2	35.4	21.60	17.70	1.165	4.617
2	4.1	3.7	33.8	2060.4	69.2	16.66	13.65	1.165	4.616
3	6.1	5.7	36.2	2024.2	105.4	17.84	14.62	1.165	4.616
4	8.1	7.7	45.6	1978.6	151.0	22.47	18.42	1.165	4.616
5	10.1	9.8	53.3	1925.3	204.3	26.27	21.53	1.165	4.616
6	12.2	11.8	54.4	1870.9	258.7	26.81	21.98	1.165	4.616
7	14.2	13.8	57.4	1813.5	316.1	28.29	23.19	1.165	4.616
8	16.2	15.8	68.6	1744.9	384.7	33.81	27.71	1.165	4.616
9	18.3	17.9	78.6	1666.3	463.3	38.74	31.75	1.165	4.288
10	20.3	19.9	80.2	1586.1	543.5	39.53	32.40	1.165	3.344
Avg. Shaft			54.4			27.31	22.39	1.165	4.381
Toe			1586.1				17050.26	0.356	4.834

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.988	0.881
Damping Type		Smith	
Unloading Quake	(% of loading quake)	100	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	
Resistance Gap (included in Toe Quake) (mm)			0.170
Soil Plug Weight	(kN)		1.01

CAPWAP match quality = 1.22 (Wave Up Match) ; RSA = 0
Observed: final set = 1.300 mm; blow count = 769 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 193.7 MPa (T= 20.8 ms, max= 1.018 x Top)
max. Comp. Stress = 197.1 MPa (Z= 2.0 m, T= 21.0 ms)
max. Tens. Stress = -6.76 MPa (Z= 20.3 m, T= 40.2 ms)
max. Energy (EMX) = 24.00 kJ; max. Measured Top Displ. (DMX)=12.48 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3074.1	-23.0	193.7	-1.45	24.00	4.5	12.388
2	2.0	3128.0	-26.4	197.1	-1.66	23.84	4.4	12.086
3	3.0	2981.3	-25.5	187.9	-1.61	22.38	4.3	11.785
4	4.1	3031.8	-29.1	191.0	-1.83	22.21	4.3	11.463
5	5.1	2903.2	-28.8	182.9	-1.82	20.88	4.2	11.128
6	6.1	2955.6	-30.8	186.2	-1.94	20.69	4.1	10.792
7	7.1	2831.6	-31.7	178.4	-2.00	19.39	4.1	10.451
8	8.1	2890.7	-33.3	182.2	-2.10	19.15	4.0	10.066
9	9.1	2741.8	-33.4	172.8	-2.11	17.67	3.9	9.689
10	10.1	2804.4	-35.4	176.7	-2.23	17.48	3.8	9.351
11	11.2	2638.1	-34.4	166.2	-2.17	16.00	3.8	8.990
12	12.2	2697.9	-43.9	170.0	-2.77	15.72	3.7	8.581
13	13.2	2545.8	-39.6	160.4	-2.50	14.29	3.7	8.144
14	14.2	2602.9	-60.8	164.0	-3.83	13.96	3.6	7.696
15	15.2	2474.4	-62.8	155.9	-3.96	12.70	3.5	7.281
16	16.2	2529.8	-82.9	159.4	-5.22	12.38	3.5	6.837
17	17.2	2409.9	-78.9	151.9	-4.97	11.08	3.4	6.369
18	18.3	2653.3	-96.3	167.2	-6.07	10.65	3.1	5.861
19	19.3	2537.5	-91.9	159.9	-5.79	9.38	2.9	5.340
20	20.3	2400.5	-107.2	151.3	-6.76	8.71	3.1	4.837
Absolute	2.0			197.1			(T =	21.0 ms)
	20.3				-6.76		(T =	40.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3587.4	3350.9	3114.4	2877.9	2641.4	2404.9	2168.4	1931.9	1695.4	1458.9
RX	3587.4	3350.9	3114.4	2877.9	2641.4	2404.9	2168.4	1952.2	1887.4	1822.7
RU	3706.5	3481.9	3257.4	3032.8	2808.2	2583.6	2359.0	2134.4	1909.8	1685.2

RAU = 1307.3 (kN); RA2 = 2139.0 (kN)

Current CAPWAP Ru = 2129.6 (kN); Corresponding J(RP)= 0.62; J(RX) = 0.62

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.46	20.59	2859.0	3093.5	3102.9	12.483	-0.854	1.300	24.3	3521.4

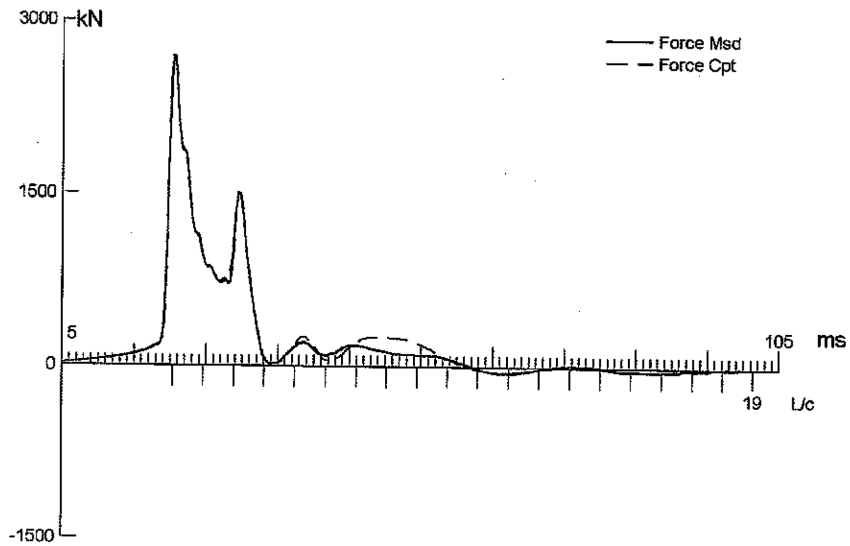
PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206842.7	77.287	1.220
20.29	158.70	206842.7	77.287	1.220

Toe Area 0.093 m^2

Top Segment Length 1.01 m, Top Impedance 640.75 kN/m/s

Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 7.9 ms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kN
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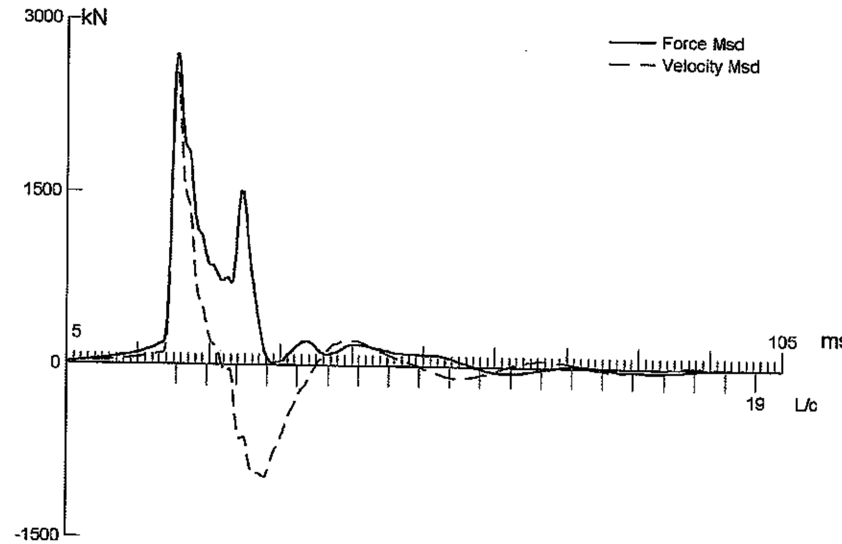
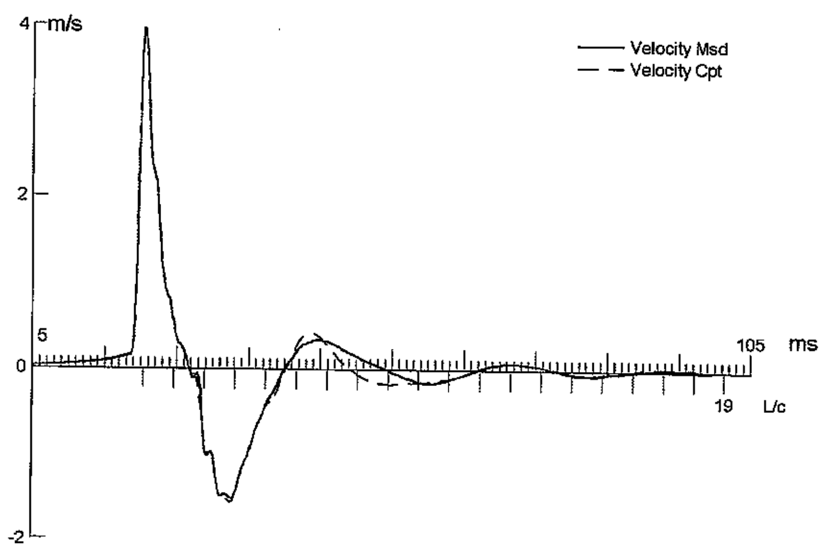
0
-1500
5
105
ms
19
L/c
Force Msd
Force Cpt


3000
kN
1500
0
-1500
5
105
ms
19
L/c
Force Msd
Velocity Msd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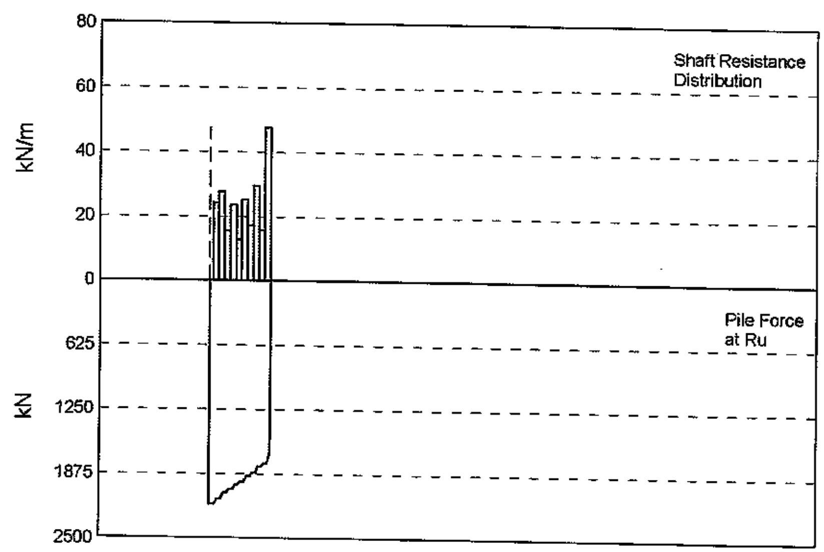
m/s
2
0
-2
5
105
ms
19
L/c
Velocity Msd
Velocity Cpt


80
60
40
20
0
625
1250
1875
2500
kN/m
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]

[redacted]16
September 19, [redacted]ow: 137
[redacted]

Test: 19-Sep-2008 16:59:
CAPWAP(R) 2006-2
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2165.5; along Shaft 494.9; at Toe 1670.6 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2165.5				
1	3.1	2.1	50.0	2115.5	50.0	23.57	19.32	0.761
2	5.2	4.2	56.9	2058.6	106.9	27.34	22.41	0.761
3	7.3	6.3	32.1	2026.5	139.0	15.43	12.64	0.761
4	9.4	8.4	48.9	1977.6	187.9	23.50	19.26	0.761
5	11.4	10.4	26.5	1951.1	214.4	12.73	10.44	0.761
6	13.5	12.5	52.1	1899.0	266.5	25.04	20.52	0.761
7	15.6	14.6	35.8	1863.2	302.3	17.20	14.10	0.761
8	17.7	16.7	61.2	1802.0	363.5	29.41	24.11	0.761
9	19.8	18.8	32.3	1769.7	395.8	15.52	12.72	0.761
10	21.9	20.9	99.1	1670.6	494.9	47.62	39.03	0.761
Avg. Shaft			49.5			23.74	19.46	0.761
Toe			1670.6				17958.61	0.586

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.756	2.738
Case Damping Factor		0.588	1.528
Damping Type		Smith	
Unloading Quake	(% of loading quake)	100	151
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake)	(mm)		0.947

CAPWAP match quality = 2.24 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 1.936 mm; blow count = 517 b/m

max. Top Comp. Stress = 170.6 MPa (T= 20.9 ms, max= 1.038 x Top)
max. Comp. Stress = 177.1 MPa (Z= 21.9 m, T= 25.2 ms)
max. Tens. Stress = -4.80 MPa (Z= 3.1 m, T= 34.3 ms)
max. Energy (EMX) = 22.02 kJ; max. Measured Top Displ. (DMX)=12.55 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2707.2	-50.5	170.6	-3.18	22.02	3.9	12.288
2	2.1	2760.0	-53.9	173.9	-3.40	21.93	3.9	12.043
3	3.1	2796.8	-76.2	176.2	-4.80	21.82	3.8	11.791
4	4.2	2661.4	-20.1	167.7	-1.27	20.23	3.7	11.537
5	5.2	2685.5	-42.9	169.2	-2.70	20.11	3.6	11.259
6	6.2	2509.8	0.0	158.1	0.00	18.40	3.6	10.980
7	7.3	2537.1	0.0	159.9	0.00	18.26	3.5	10.685
8	8.3	2469.1	0.0	155.6	0.00	17.28	3.5	10.390
9	9.4	2488.1	0.0	156.8	0.00	17.12	3.4	10.066
10	10.4	2344.9	0.0	147.8	0.00	15.74	3.4	9.727
11	11.4	2370.0	0.0	149.3	0.00	15.51	3.3	9.337
12	12.5	2329.5	0.0	146.8	0.00	14.63	3.3	8.918
13	13.5	2351.5	0.0	148.2	0.00	14.32	3.2	8.470
14	14.6	2211.0	0.0	139.3	0.00	12.94	3.2	8.003
15	15.6	2239.9	-3.0	141.1	-0.19	12.50	3.1	7.467
16	16.6	2172.8	0.0	136.9	0.00	11.31	3.0	6.869
17	17.7	2219.5	-28.9	139.9	-1.82	10.60	3.0	6.205
18	18.7	2282.6	0.0	143.8	0.00	9.07	2.9	5.532
19	19.8	2322.3	-6.9	146.3	-0.43	8.44	2.7	4.890
20	20.8	2560.9	0.0	161.4	0.00	7.54	2.4	4.276
21	21.9	2810.2	-9.8	177.1	-0.62	7.06	1.9	3.686
Absolute	21.9			177.1			(T =	25.2 ms)
	3.1				-4.80		(T =	34.3 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3466.1	3291.3	3116.4	2941.6	2766.7	2591.9	2417.0	2242.1	2067.3	1892.4
RX	3578.9	3416.0	3253.2	3091.9	2950.1	2808.2	2666.4	2524.6	2392.0	2274.6
RU	3694.6	3542.6	3390.6	3238.6	3086.5	2934.5	2782.5	2630.5	2478.5	2326.5

RAU = 284.6 (kN); RA2 = 1989.1 (kN)

Current CAPWAP Ru = 2165.5 (kN); Corresponding J(RP)= 0.74; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.95	20.51	2527.9	2686.7	2692.8	12.549	3.883	1.000	22.1	3266.9

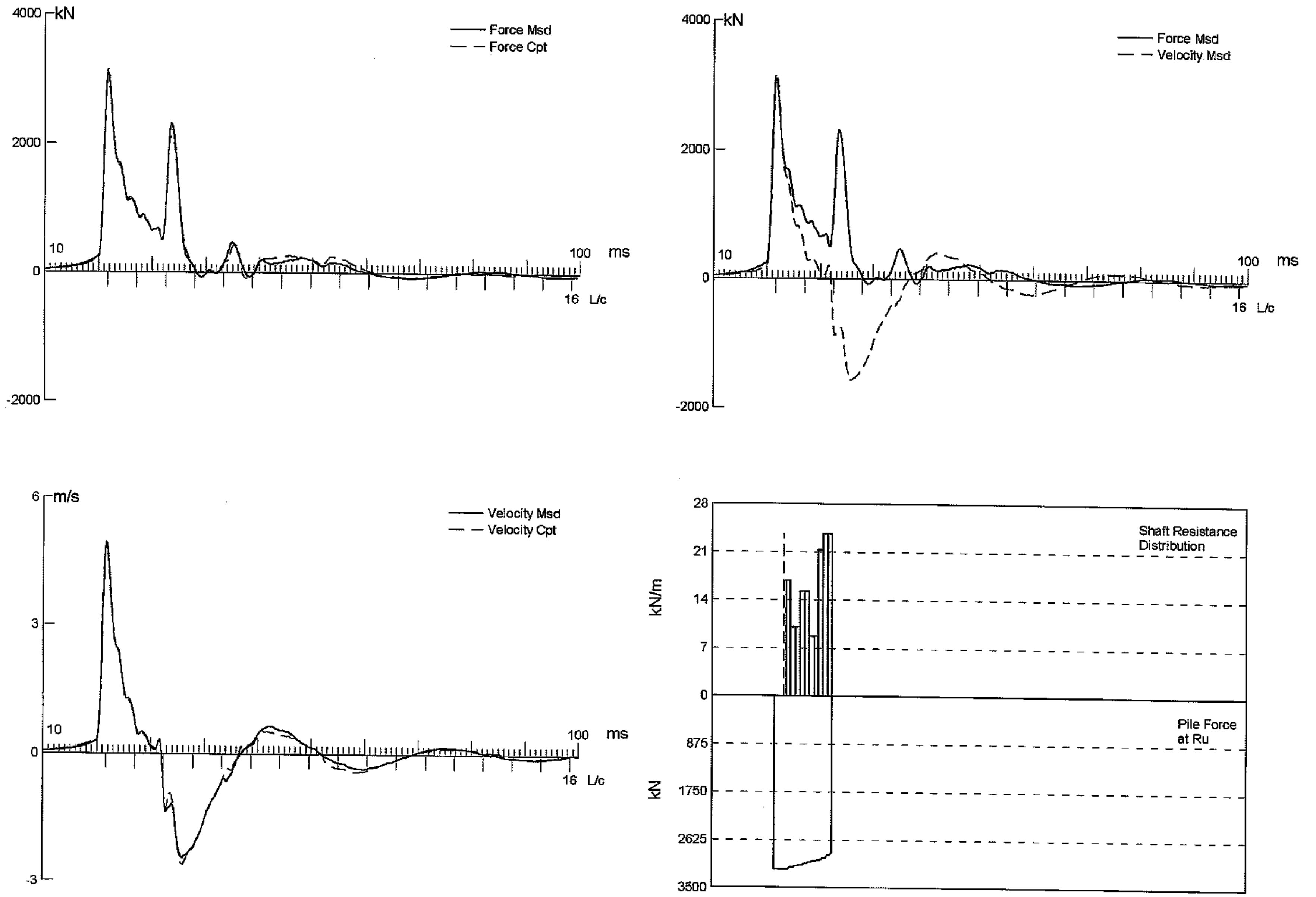
PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206842.7	77.287	1.220
21.85	158.70	206842.7	77.287	1.220

Toe Area 0.093 m²

Top Segment Length 1.04 m, Top Impedance 640.75 kN/m/s

Pile Damping 1.0 %, Time Incr 0.203 ms, Wave Speed 5123.0 m/s, 2L/c 8.5 ms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m/s
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-3
Velocity Cpt
28
21
14
7
kN/m
875
1750
2625
3500
kN
Shaft Resistance
Distribution
Pile Force
at Ru

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3157.1; along Shaft 307.1; at Toe 2850.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				3157.1					
1	7.0	2.9	33.8	3123.3	33.8	11.68	9.58	1.432	4.271
2	9.0	4.9	20.0	3103.3	53.8	10.00	8.19	1.432	4.280
3	11.0	6.9	20.0	3083.3	73.8	10.00	8.19	1.432	4.280
4	13.0	8.9	30.5	3052.8	104.3	15.24	12.50	1.432	4.280
5	15.0	10.9	30.5	3022.3	134.8	15.24	12.50	1.432	4.280
6	17.0	12.9	17.4	3004.9	152.2	8.70	7.13	1.432	4.280
7	19.0	14.9	17.4	2987.5	169.6	8.70	7.13	1.432	4.280
8	21.0	16.9	42.7	2944.8	212.3	21.34	17.49	1.432	4.280
9	23.0	18.9	47.4	2897.4	259.7	23.69	19.42	1.432	4.280
10	25.0	20.9	47.4	2850.0	307.1	23.69	19.42	1.432	3.717
Avg. Shaft			30.7			14.69	12.04	1.432	4.192
Toe			2850.0				29277.61	0.202	5.074

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.686	0.898
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	100	60
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	54	
Resistance Gap (included in Toe Quake) (mm)			2.042
Soil Plug Weight	(kN)		0.39

CAPWAP match quality = 2.27 (Wave Up Match) ; RSA = 0
Observed: final set = 2.080 mm; blow count = 481 b/m
Computed: final set = 1.871 mm; blow count = 534 b/m

max. Top Comp. Stress = 198.2 MPa (T= 20.9 ms, max= 1.114 x Top)
max. Comp. Stress = 220.9 MPa (Z= 25.0 m, T= 26.0 ms)
max. Tens. Stress = -32.34 MPa (Z= 19.0 m, T= 40.0 ms)
max. Energy (EMX) = 31.60 kJ; max. Measured Top Displ. (DMX)=17.75 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3146.2	-117.5	198.2	-7.40	31.60	4.9	17.643
2	2.0	3146.1	-143.0	198.2	-9.01	31.51	4.9	17.408
4	4.0	3150.2	-214.5	198.5	-13.52	31.32	4.9	16.946
6	6.0	3210.2	-301.9	202.3	-19.03	31.13	4.8	16.463
8	8.0	3044.4	-256.7	191.8	-16.18	28.61	4.6	15.857
9	9.0	3086.3	-276.1	194.5	-17.40	28.41	4.6	15.518
10	10.0	2963.4	-293.9	186.7	-18.52	26.97	4.5	15.173
11	11.0	3008.1	-344.8	189.5	-21.72	26.79	4.4	14.845
12	12.0	2900.2	-358.2	182.7	-22.57	25.39	4.4	14.492
13	13.0	2956.4	-373.9	186.3	-23.56	25.12	4.3	14.063
14	14.0	2779.1	-367.9	175.1	-23.18	23.07	4.2	13.582
15	15.0	2826.0	-390.0	178.1	-24.57	22.64	4.2	13.079
16	16.0	2640.4	-409.8	166.4	-25.82	20.67	4.1	12.574
17	17.0	2666.7	-469.3	168.0	-29.57	20.24	4.1	12.072
18	18.0	2568.7	-489.6	161.9	-30.85	18.86	4.1	11.502
19	19.0	2603.7	-513.3	164.1	-32.34	18.15	4.0	10.824
20	20.0	2663.5	-502.2	167.8	-31.64	16.45	3.9	10.082
21	21.0	2760.5	-509.2	173.9	-32.09	15.41	3.8	9.280
22	22.0	2755.2	-448.9	173.6	-28.29	12.85	3.9	8.465
23	23.0	2925.0	-450.8	184.3	-28.40	11.62	4.1	7.607
24	24.0	3167.9	-406.3	199.6	-25.60	9.10	3.9	6.726
25	25.0	3506.1	-405.5	220.9	-25.55	8.50	3.1	5.847
Absolute	25.0			220.9			(T =	26.0 ms)
	19.0				-32.34		(T =	40.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4199.9	3978.3	3756.8	3535.3	3313.7	3092.2	2870.7	2649.2	2427.6	2206.1
RX	4358.9	4174.5	4030.6	3887.4	3745.7	3640.2	3534.7	3429.1	3323.6	3220.1
RU	4221.5	4002.1	3782.8	3563.4	3344.0	3124.7	2905.3	2685.9	2466.6	2247.2

RAU = 1438.1 (kN); RA2 = 2401.9 (kN)

Current CAPWAP Ru = 3157.1 (kN); Corresponding J(RP)= 0.47; matches RX9 within 5%

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
5.07	20.70	3251.3	3163.9	3163.9	17.746	2.086	2.080	32.0	3227.3

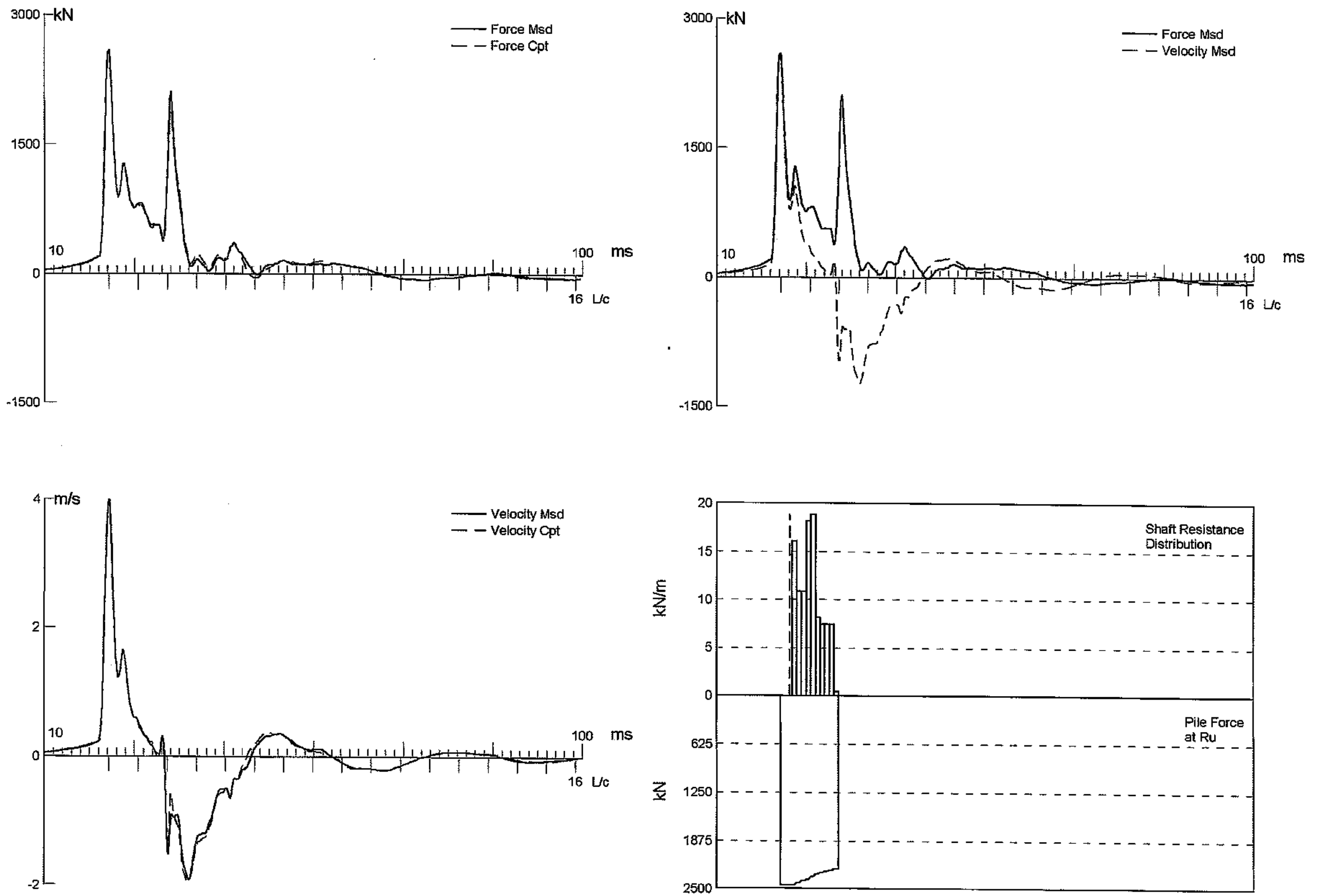
PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206842.7	77.287	1.220
25.01	158.70	206842.7	77.287	1.220

Toe Area 0.097 m²

Top Segment Length 1.00 m, Top Impedance 640.75 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 9.8 ms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0
-1500
10
100 ms
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Velocity Msd
4 m/s
2
-2
Velocity Cpt
20
15
5
kN/m
625
1250
1875
2500
kN
Shaft Resistance Distribution
Pile Force at Ru

[redacted]

[redacted]Pile 15 (RSTRK) Test: 21-Jul-2008 18:55:
July 21, 2008; Blow: 53; Delmag Hammer CAPWAP(R) 2006-2
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2450.1; along Shaft 211.4; at Toe 2238.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2450.1					
1	7.0	2.9	32.2	2417.9	32.2	11.13	9.12	1.313	1.842
2	9.0	4.9	21.8	2396.1	54.0	10.90	8.93	1.313	1.844
3	11.0	6.9	21.8	2374.3	75.8	10.90	8.93	1.313	1.844
4	13.0	8.9	36.4	2337.9	112.2	18.19	14.91	1.313	1.844
5	15.0	10.9	37.7	2300.2	149.9	18.84	15.44	1.313	1.844
6	17.0	12.9	16.2	2284.0	166.1	8.10	6.64	1.313	1.844
7	19.0	14.9	14.8	2269.2	180.9	7.40	6.06	1.313	1.844
8	21.0	16.9	14.8	2254.4	195.7	7.40	6.06	1.313	1.844
9	23.0	18.9	14.8	2239.6	210.5	7.40	6.06	1.313	1.844
10	25.0	20.9	0.9	2238.7	211.4	0.45	0.37	1.313	1.844
Avg. Shaft			21.1			10.11	8.29	1.313	1.844
Toe			2238.7				22997.82	0.449	3.368

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.433	1.569
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	94	164
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	96	
Resistance Gap (included in Toe Quake) (mm)			1.284

CAPWAP match quality = 1.92 (Wave Up Match) ; RSA = 0
Observed: final set = 1.000 mm; blow count = 1000 b/m
Computed: final set = 1.113 mm; blow count = 898 b/m

max. Top Comp. Stress = 164.0 MPa (T= 21.1 ms, max= 1.227 x Top)
max. Comp. Stress = 201.2 MPa (Z= 25.0 m, T= 26.0 ms)
max. Tens. Stress = -12.22 MPa (Z= 23.0 m, T= 41.8 ms)
max. Energy (EMX) = 17.85 kJ; max. Measured Top Displ. (DMX)=12.94 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2602.2	-66.9	164.0	-4.22	17.85	4.0	12.874
2	2.0	2604.4	-70.1	164.1	-4.42	17.79	4.0	12.697
4	4.0	2615.1	-80.9	164.8	-5.10	17.67	4.0	12.328
6	6.0	2686.7	-91.8	169.3	-5.78	17.53	3.8	11.917
8	8.0	2553.8	-81.9	160.9	-5.16	16.10	3.7	11.459
9	9.0	2578.5	-88.2	162.5	-5.56	15.98	3.7	11.195
10	10.0	2482.4	-82.2	156.4	-5.18	15.03	3.7	10.903
11	11.0	2516.2	-94.5	158.6	-5.95	14.87	3.6	10.593
12	12.0	2438.2	-90.6	153.6	-5.71	13.95	3.5	10.277
13	13.0	2475.2	-100.9	156.0	-6.36	13.78	3.5	9.959
14	14.0	2315.8	-84.0	145.9	-5.29	12.46	3.4	9.652
15	15.0	2336.8	-89.5	147.2	-5.64	12.29	3.4	9.313
16	16.0	2149.9	-93.6	135.5	-5.90	10.92	3.3	8.936
17	17.0	2160.1	-133.1	136.1	-8.39	10.57	3.3	8.495
18	18.0	2084.6	-142.2	131.4	-8.96	9.69	3.3	7.965
19	19.0	2208.5	-167.9	139.2	-10.58	9.16	3.2	7.412
20	20.0	2165.6	-167.9	136.5	-10.58	8.49	3.1	6.960
21	21.0	2170.4	-184.0	136.8	-11.60	8.21	3.0	6.553
22	22.0	2016.3	-183.9	127.1	-11.59	7.49	3.2	6.048
23	23.0	2320.5	-194.0	146.2	-12.22	6.82	3.4	5.416
24	24.0	2810.4	-180.3	177.1	-11.36	5.60	2.9	4.657
25	25.0	3192.7	-192.5	201.2	-12.13	5.40	2.0	3.920
Absolute	25.0			201.2			(T =	26.0 ms)
	23.0				-12.22		(T =	41.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3804.3	3659.3	3514.4	3369.4	3224.4	3079.5	2934.5	2789.5	2644.6	2499.6
RX	3833.9	3727.2	3620.5	3513.8	3407.1	3300.3	3193.6	3086.9	2980.2	2873.6
RU	3813.0	3668.9	3524.8	3380.7	3236.6	3092.5	2948.4	2804.3	2660.2	2516.1

RAU = 1683.3 (kN); RA2 = 1949.6 (kN)

Current CAPWAP Ru = 2450.1 (kN);

Case Method matching requires higher damping factor - please check with PDA-W

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.09	20.89	2618.3	2635.6	2635.6	12.939	1.010	1.000	18.1	2601.5

PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206842.7	77.287	1.220
25.01	158.70	206842.7	77.287	1.220

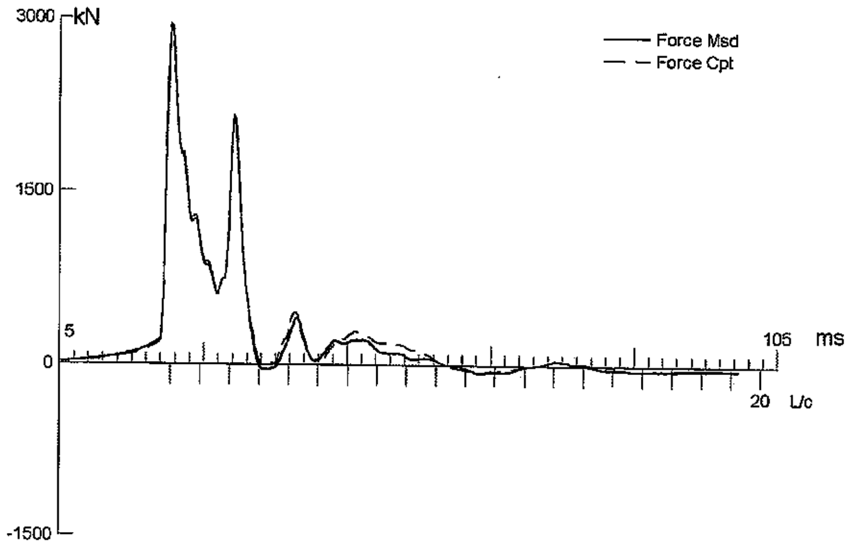
Toe Area 0.097 m^2

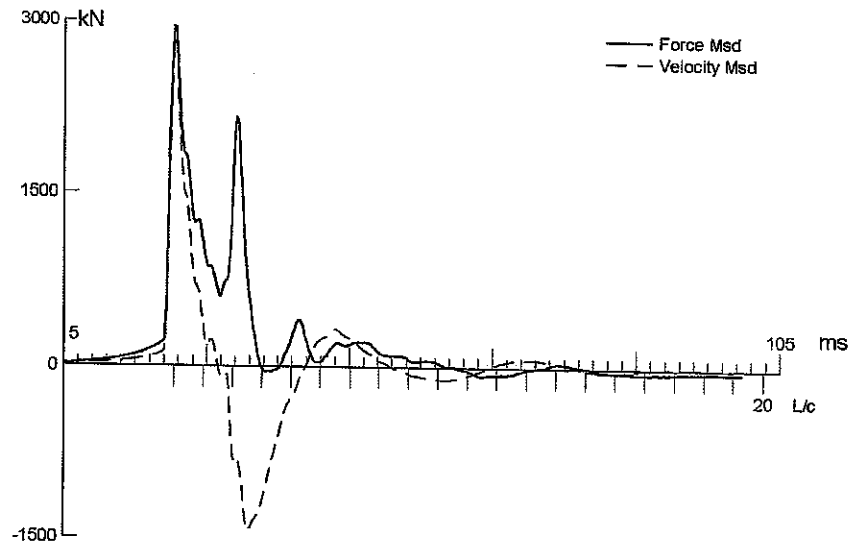
Top Segment Length 1.00 m, Top Impedance 640.75 kN/m/s

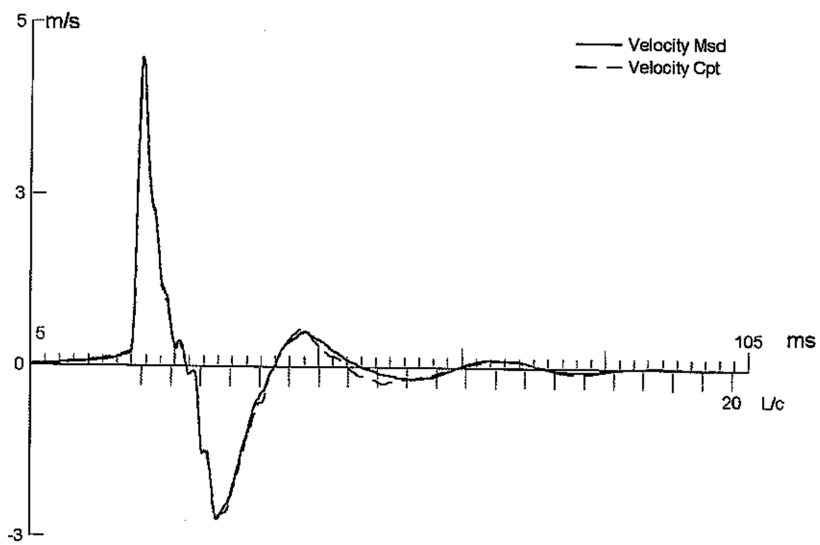
Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 9.8 ms

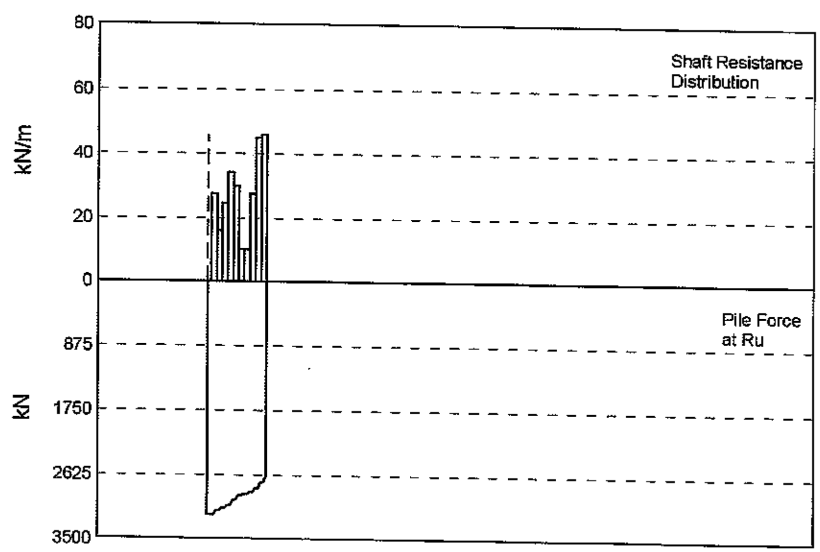
[redacted] Pile15 (RSTRK); July 30, 2008; Blow: 15 (Test: 30-Jul-2008 10:11:)

13-Jan-2009
CAPWAP(R) 2006-2









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[redacted]Pile15 (RSTRK)
July 30, 2008; Blow: 15
[redacted]

Test: 30-Jul-2008 10:11:
CAPWAP(R) 2006-2
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3166.0; along Shaft 541.0; at Toe 2625.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				3166.0				
1	3.0	3.0	55.0	3111.0	55.0	18.27	14.97	0.832
2	5.0	5.0	32.0	3079.0	87.0	15.94	13.07	0.832
3	7.0	7.0	49.0	3030.0	136.0	24.41	20.01	0.832
4	9.0	9.0	68.0	2962.0	204.0	33.88	27.77	0.832
5	11.0	11.0	60.0	2902.0	264.0	29.89	24.50	0.832
6	13.0	13.0	20.0	2882.0	284.0	9.96	8.17	0.832
7	15.1	15.1	20.0	2862.0	304.0	9.96	8.17	0.832
8	17.1	17.1	55.0	2807.0	359.0	27.40	22.46	0.832
9	19.1	19.1	90.0	2717.0	449.0	44.84	36.75	0.832
10	21.1	21.1	92.0	2625.0	541.0	45.84	37.57	0.832
Avg. Shaft			54.1			25.67	21.04	0.832
Toe			2625.0				28218.22	0.667

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	3.199	3.556
Case Damping Factor		0.702	2.732
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	94	142
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	73	
Resistance Gap (included in Toe Quake)	(mm)		1.092
Soil Plug Weight	(kN)		0.99

CAPWAP match quality = 1.93 (Wave Up Match) ; RSA = 0
Observed: final set = 0.300 mm; blow count = 3333 b/m
Computed: final set = 1.273 mm; blow count = 786 b/m

max. Top Comp. Stress = 187.1 MPa (T= 20.8 ms, max= 1.150 x Top)
max. Comp. Stress = 215.1 MPa (Z= 21.1 m, T= 25.1 ms)
max. Tens. Stress = -17.22 MPa (Z= 19.1 m, T= 37.2 ms)
max. Energy (EMX) = 25.37 kJ; max. Measured Top Displ. (DMX)=13.58 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2968.5	-70.5	187.1	-4.45	25.37	4.5	13.366
2	2.0	3020.1	-96.6	190.3	-6.09	25.28	4.4	13.148
3	3.0	3074.1	-150.6	193.7	-9.49	25.19	4.3	12.915
4	4.0	2853.6	-100.7	179.8	-6.34	23.21	4.2	12.685
5	5.0	2894.8	-140.3	182.4	-8.84	23.10	4.2	12.427
6	6.0	2803.0	-108.8	176.6	-6.85	21.93	4.1	12.140
7	7.0	2857.3	-119.4	180.0	-7.52	21.78	4.0	11.840
8	8.0	2711.5	-48.4	170.9	-3.05	20.18	3.9	11.559
9	9.0	2770.8	-52.6	174.6	-3.31	20.02	3.8	11.250
10	10.0	2541.6	-35.4	160.2	-2.23	17.94	3.7	10.899
11	11.0	2581.7	-90.0	162.7	-5.67	17.65	3.6	10.472
12	12.0	2363.8	-90.9	148.9	-5.73	15.77	3.6	10.021
13	13.0	2383.6	-145.4	150.2	-9.16	15.42	3.6	9.559
14	14.1	2319.4	-173.0	146.1	-10.90	14.55	3.6	9.066
15	15.1	2346.1	-206.2	147.8	-13.00	14.01	3.5	8.478
16	16.1	2533.7	-218.6	159.7	-13.78	12.82	3.5	7.794
17	17.1	2751.7	-257.4	173.4	-16.22	11.89	3.4	7.027
18	18.1	2778.1	-239.0	175.1	-15.06	10.04	3.3	6.259
19	19.1	2866.2	-273.3	180.6	-17.22	8.90	3.2	5.427
20	20.1	3094.7	-218.8	195.0	-13.79	6.64	3.0	4.524
21	21.1	3413.3	-242.8	215.1	-15.30	5.66	2.3	3.617
Absolute	21.1			215.1			(T =	25.1 ms)
	19.1				-17.22		(T =	37.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4182.2	4010.3	3838.4	3666.4	3494.5	3322.6	3150.7	2978.8	2806.8	2634.9
RX	4218.4	4079.6	3940.8	3802.0	3663.2	3524.4	3397.1	3288.6	3180.1	3071.7
RU	4333.1	4176.3	4019.4	3862.6	3705.8	3548.9	3392.1	3235.3	3078.4	2921.6

RAU = 593.8 (kN); RA2 = 2329.6 (kN)

Current CAPWAP Ru = 3166.0 (kN); Corresponding J(RP)= 0.59; J(RX) = 0.81

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.54	20.57	2912.0	2989.4	2989.4	13.579	0.333	0.300	25.7	3709.3

PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206874.8	77.300	1.220
21.07	158.70	206874.8	77.300	1.220

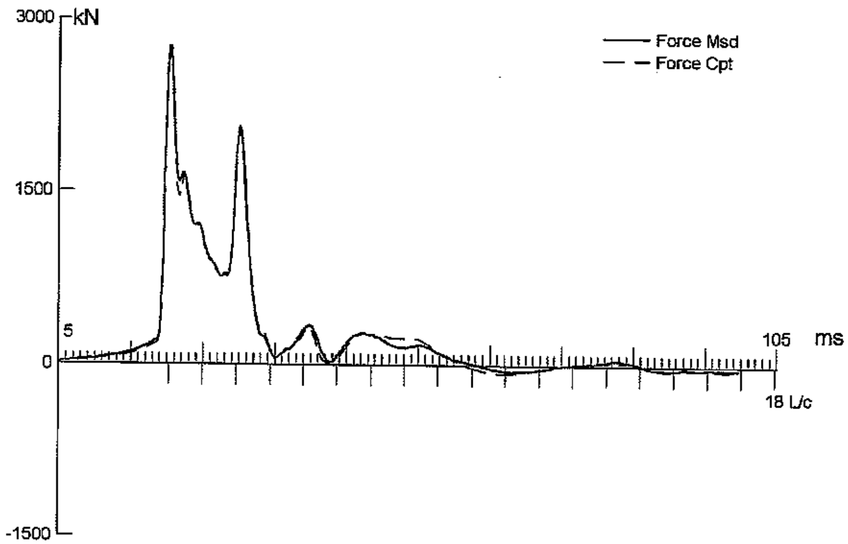
Toe Area 0.093 m^2

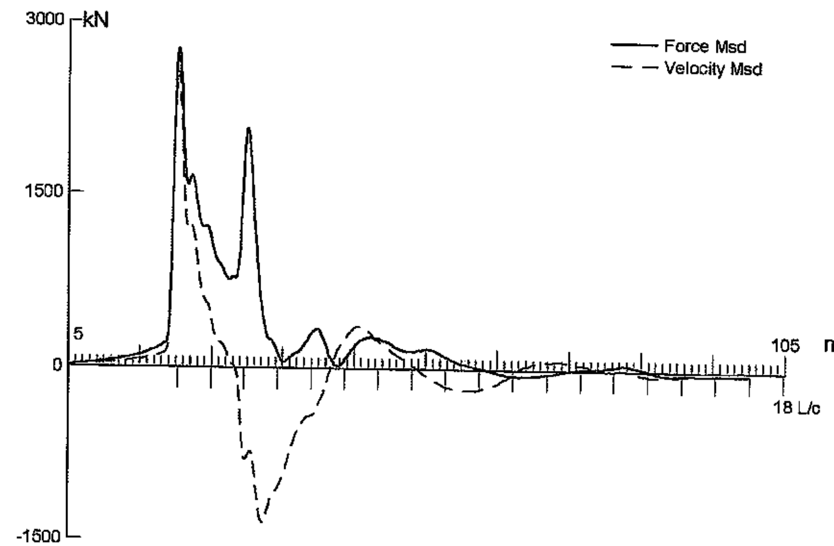
Top Segment Length 1.00 m, Top Impedance 640.86 kN/m/s

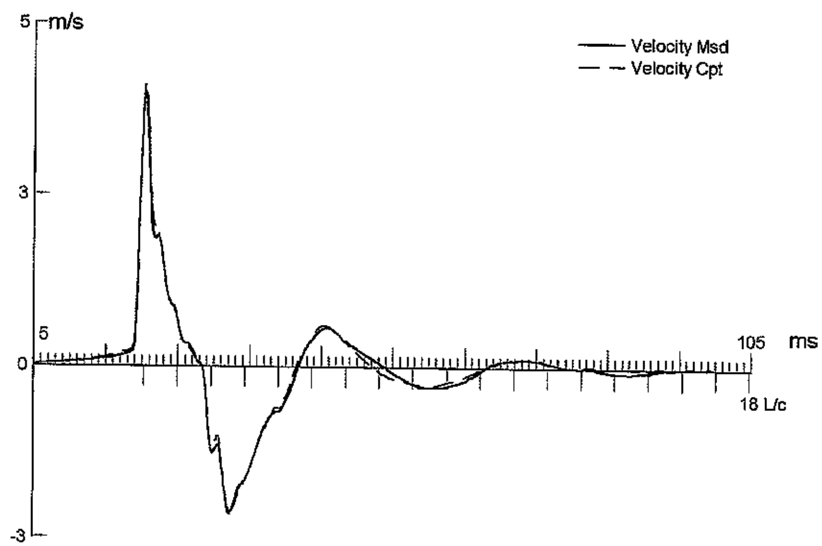
Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 5123.0 m/s, 2L/c 8.2 ms

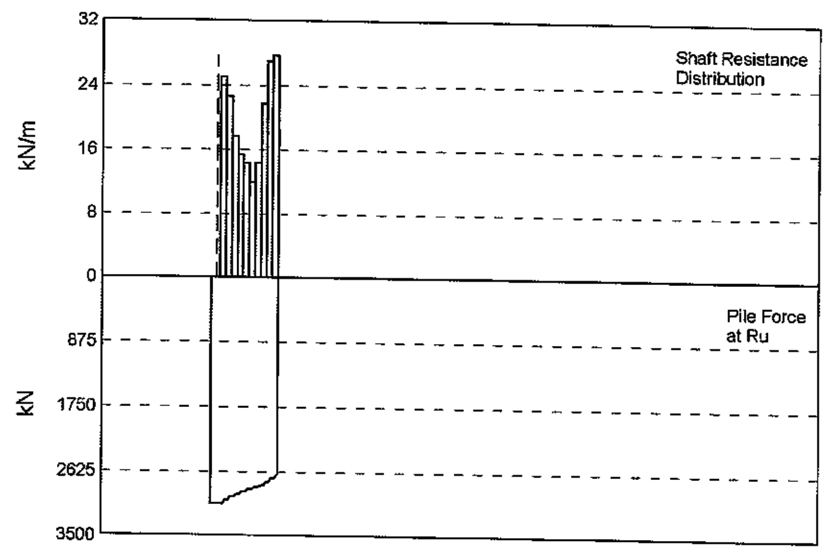
[redacted] Pile 16 (RSTRK); July 21, 2008; Blow: 3 (Test: 21-Jul-2008 15:20:)

13-Jan-2009
CAPWAP(R) 2006-2









[redacted]

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 3056.9; along Shaft 406.9; at Toe 2650.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				3056.9					
1	5.1	2.4	51.6	3005.3	51.6	21.65	17.75	1.398	3.791
2	7.2	4.4	46.7	2958.6	98.3	22.70	18.61	1.398	3.789
3	9.3	6.5	36.3	2922.3	134.6	17.64	14.46	1.398	3.789
4	11.3	8.6	31.6	2890.7	166.2	15.36	12.59	1.398	3.789
5	13.4	10.6	29.4	2861.3	195.6	14.29	11.71	1.398	3.789
6	15.4	12.7	24.5	2836.8	220.1	11.91	9.76	1.398	3.789
7	17.5	14.7	29.4	2807.4	249.5	14.29	11.71	1.398	3.789
8	19.5	16.8	44.8	2762.6	294.3	21.78	17.85	1.398	3.789
9	21.6	18.8	55.5	2707.1	349.8	26.98	22.11	1.398	3.553
10	23.7	20.9	57.1	2650.0	406.9	27.75	22.75	1.398	1.408
Avg. Shaft			40.7			19.47	15.96	1.398	3.423
Toe			2650.0				27223.04	0.806	2.302

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.888	3.333
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	67	229
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	75	
Resistance Gap (included in Toe Quake) (mm)			1.298
Soil Plug Weight	(kN)		0.87

CAPWAP match quality = 1.78 (Wave Up Match) ; RSA = 0
Observed: final set = 0.600 mm; blow count = 1667 b/m
Computed: final set = 1.327 mm; blow count = 753 b/m

max. Top Comp. Stress = 169.5 MPa (T= 20.6 ms, max= 1.229 x Top)
max. Comp. Stress = 208.4 MPa (Z= 23.7 m, T= 25.3 ms)
max. Tens. Stress = -11.80 MPa (Z= 21.6 m, T= 39.8 ms)
max. Energy (EMX) = 22.39 kJ; max. Measured Top Displ. (DMX)=13.50 mm

[redacted] - Pile 16 (RSTRK)
July 21, 2008; Blow: 3
[redacted]

Test: 21-Jul-2008 15:20:
CAPWAP(R) 2006-2
OP: RB

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2690.3	-80.2	169.5	-5.05	22.39	4.1	13.803
2	2.1	2694.8	-87.2	169.8	-5.50	22.28	4.1	13.543
4	4.1	2789.7	-99.4	175.8	-6.27	22.03	4.0	12.985
5	5.1	2873.2	-104.3	181.0	-6.57	21.87	3.9	12.677
6	6.2	2605.2	-91.2	164.2	-5.75	19.60	3.8	12.359
7	7.2	2675.9	-96.8	168.6	-6.10	19.42	3.7	12.023
8	8.2	2450.0	-85.7	154.4	-5.40	17.48	3.6	11.691
9	9.3	2499.2	-90.6	157.5	-5.71	17.28	3.6	11.332
10	10.3	2348.5	-83.5	148.0	-5.26	15.76	3.5	10.935
11	11.3	2389.3	-86.8	150.6	-5.47	15.45	3.4	10.479
12	12.3	2265.3	-82.5	142.7	-5.20	14.10	3.4	10.015
13	13.4	2300.4	-92.7	144.9	-5.84	13.76	3.3	9.543
14	14.4	2253.0	-87.8	142.0	-5.53	12.56	3.3	9.068
15	15.4	2251.0	-112.9	141.8	-7.11	12.10	3.3	8.538
16	16.5	2346.8	-122.2	147.9	-7.70	10.94	3.2	7.926
17	17.5	2487.4	-155.7	156.7	-9.81	10.24	3.2	7.261
18	18.5	2595.4	-148.7	163.5	-9.37	8.93	3.1	6.582
19	19.5	2603.3	-177.2	164.0	-11.17	8.28	3.1	5.928
20	20.6	2553.5	-162.0	160.9	-10.21	6.82	3.0	5.250
21	21.6	2767.8	-187.3	174.4	-11.80	5.81	2.7	4.431
22	22.6	3063.8	-157.9	193.1	-9.95	3.97	2.1	3.502
23	23.7	3307.1	-171.7	208.4	-10.82	2.74	1.2	2.524
Absolute	23.7			208.4			(T =	25.3 ms)
	21.6				-11.80		(T =	39.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3778.0	3623.6	3469.2	3314.8	3160.4	3006.1	2851.7	2697.3	2542.9	2388.5
RX	3946.8	3810.5	3674.2	3549.0	3440.5	3332.1	3223.6	3115.2	3006.7	2920.4
RU	3932.4	3793.5	3654.5	3515.6	3376.6	3237.7	3098.8	2959.8	2820.9	2682.0

RAU = 499.0 (kN); RA2 = 2668.9 (kN)

Current CAPWAP Ru = 3056.9 (kN); Corresponding J(RP)= 0.47; J(RX) = 0.75

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.00	20.42	2564.6	2757.2	2760.5	13.495	0.634	0.600	22.4	3178.8

PILE PROFILE AND PILE MODEL

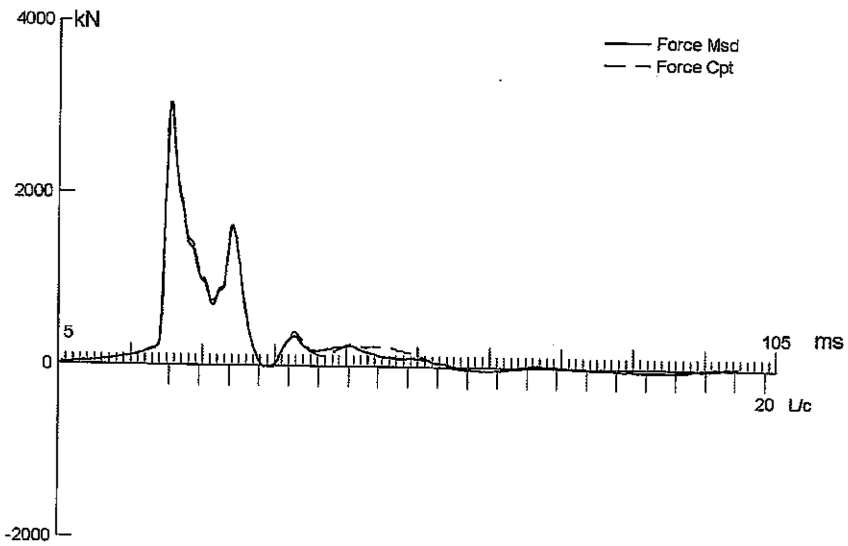
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206843.0	77.287	1.220
23.66	158.70	206843.0	77.287	1.220

Toe Area 0.097 m^2

Top Segment Length 1.03 m, Top Impedance 640.75 kN/m/s

Analysis: 13-Jan-2009

Pile Damping 1.0 %, Time Incr 0.204 ms, Wave Speed 5037.6 m/s, 2L/c 9.4 ms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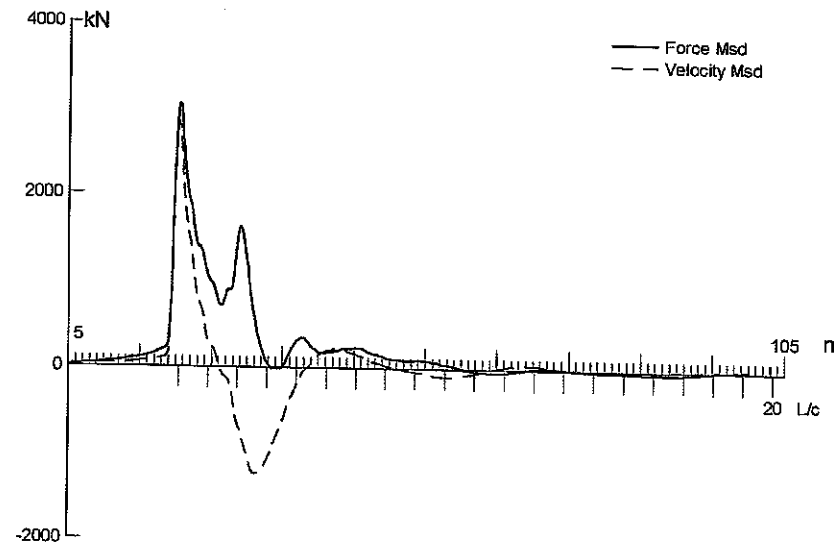
Force Msd
Force Cpt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ms
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L/c
Force Msd
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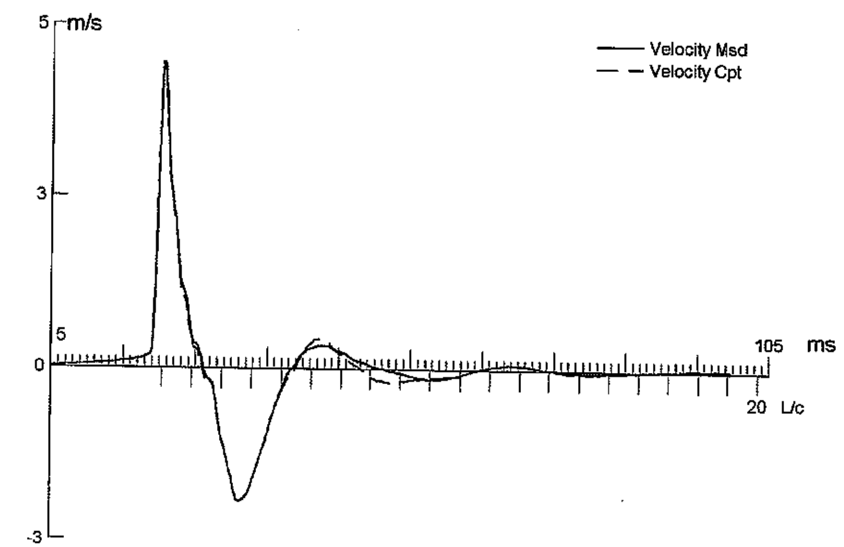
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m/s
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105
ms
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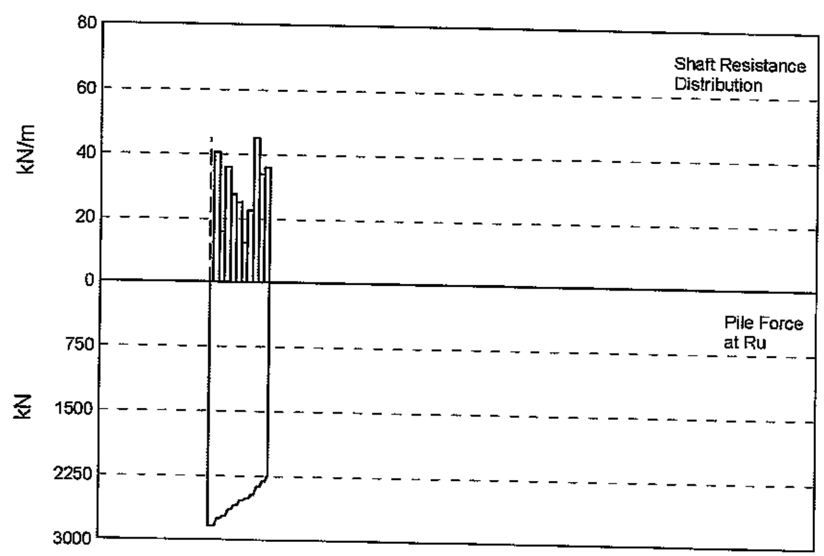
L/c
Velocity Msd
Velocity Cpt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kN/m
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2250
3000
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

[redacted]-16_(RSTRK) Test: 30-Jul-2008 09:41:
July 30, 2008; Blow: 4 CAPWAP(R) 2006-2
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2824.8; along Shaft 596.6; at Toe 2228.2 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2824.8				
1	3.0	3.0	81.9	2742.9	81.9	26.94	22.08	0.829
2	5.1	5.1	31.9	2711.0	113.8	15.74	12.90	0.829
3	7.1	7.1	73.0	2638.0	186.8	36.02	29.52	0.829
4	9.1	9.1	55.7	2582.3	242.5	27.48	22.53	0.829
5	11.1	11.1	50.7	2531.6	293.2	25.02	20.51	0.829
6	13.2	13.2	25.3	2506.3	318.5	12.48	10.23	0.829
7	15.2	15.2	45.5	2460.8	364.0	22.45	18.40	0.829
8	17.2	17.2	91.2	2369.6	455.2	45.00	36.89	0.829
9	19.3	19.3	68.4	2301.2	523.6	33.75	27.66	0.829
10	21.3	21.3	73.0	2228.2	596.6	36.02	29.52	0.829
Avg. Shaft			59.7			28.04	22.98	0.829
Toe			2228.2				23952.70	0.495

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.959	2.995
Case Damping Factor		0.772	1.721
Damping Type		Smith	
Unloading Quake	(% of loading quake)	81	143
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	67	
Resistance Gap (included in Toe Quake)	(mm)		0.916

CAPWAP match quality = 1.73 (Wave Up Match) ; RSA = 0
Observed: final set = 0.600 mm; blow count = 1667 b/m
Computed: final set = 1.308 mm; blow count = 764 b/m

max. Top Comp. Stress = 191.8 MPa (T= 20.8 ms, max= 1.050 x Top)
max. Comp. Stress = 201.4 MPa (Z= 3.0 m, T= 21.2 ms)
max. Tens. Stress = -9.28 MPa (Z= 3.0 m, T= 34.2 ms)
max. Energy (EMX) = 26.19 kJ; max. Measured Top Displ. (DMX)=13.44 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	3044.6	-61.1	191.8	-3.85	26.19	4.5	12.904
2	2.0	3123.9	-104.2	196.8	-6.57	26.06	4.4	12.634
3	3.0	3196.1	-147.3	201.4	-9.28	25.93	4.2	12.356
4	4.1	2861.6	-40.0	180.3	-2.52	23.10	4.2	12.081
5	5.1	2911.0	-63.3	183.4	-3.99	22.96	4.1	11.790
6	6.1	2838.6	-23.8	178.9	-1.50	21.82	4.0	11.479
7	7.1	2907.3	-40.7	183.2	-2.56	21.64	3.9	11.150
8	8.1	2650.1	0.0	167.0	0.00	19.42	3.8	10.825
9	9.1	2707.8	0.0	170.6	0.00	19.22	3.7	10.473
10	10.1	2521.7	0.0	158.9	0.00	17.54	3.6	10.088
11	11.1	2570.0	0.0	161.9	0.00	17.22	3.6	9.638
12	12.2	2393.8	0.0	150.8	0.00	15.65	3.5	9.164
13	13.2	2426.7	-40.2	152.9	-2.53	15.24	3.5	8.659
14	14.2	2369.1	-66.5	149.3	-4.19	14.28	3.4	8.128
15	15.2	2426.1	-109.8	152.9	-6.92	13.73	3.3	7.537
16	16.2	2327.4	-108.0	146.7	-6.81	12.27	3.3	6.892
17	17.2	2409.0	-139.7	151.8	-8.80	11.45	3.1	6.182
18	18.2	2406.8	-96.4	151.7	-6.08	9.43	3.1	5.452
19	19.3	2604.2	-117.6	164.1	-7.41	8.49	2.8	4.687
20	20.3	2766.6	-82.8	174.3	-5.22	7.04	2.4	3.925
21	21.3	2989.3	-104.7	188.4	-6.60	6.62	1.9	3.195
Absolute	3.0			201.4			(T =	21.2 ms)
	3.0				-9.28		(T =	34.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	4058.9	3869.9	3681.0	3492.0	3303.1	3114.1	2925.2	2736.2	2547.3	2358.4
RX	4128.4	3947.3	3766.2	3585.0	3403.9	3222.7	3045.3	2881.8	2718.3	2554.8
RU	4268.2	4100.2	3932.2	3764.2	3596.2	3428.2	3260.2	3092.1	2924.1	2756.1

RAU = 708.1 (kN); RA2 = 2349.2 (kN)

Current CAPWAP Ru = 2824.8 (kN); Corresponding J(RP)= 0.65; J(RX) = 0.73

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.51	20.57	2889.3	3059.0	3082.7	13.440	0.287	0.600	26.7	3800.6

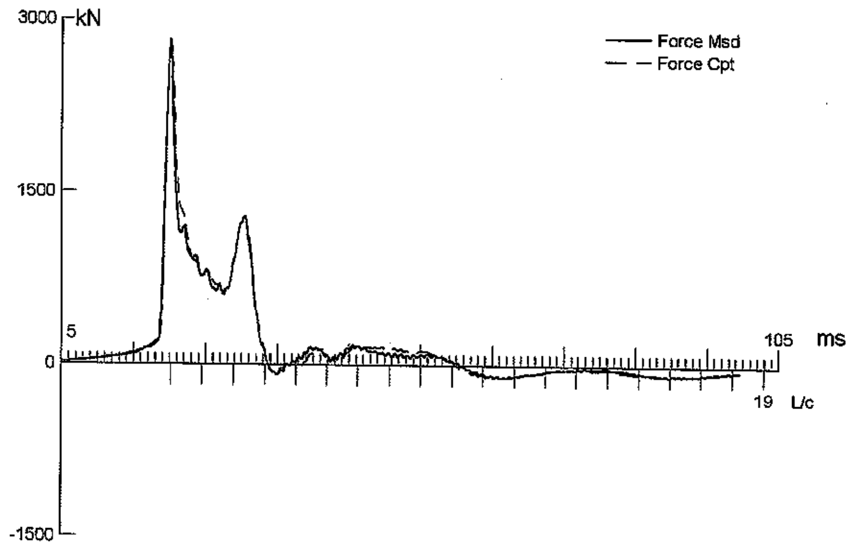
PILE PROFILE AND PILE MODEL

Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	158.70	206874.8	77.300	1.220
21.28	158.70	206874.8	77.300	1.220

Toe Area 0.093 m^2

Top Segment Length 1.01 m, Top Impedance 640.86 kN/m/s

Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 8.3 ms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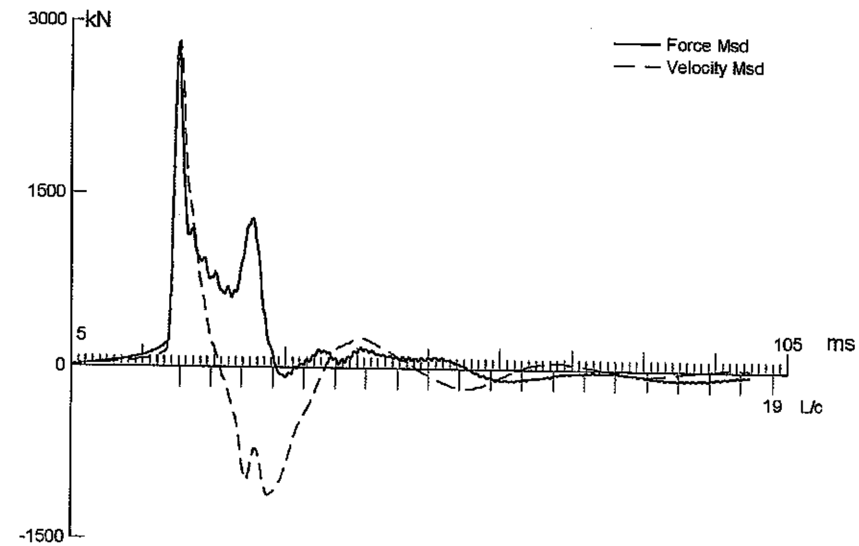
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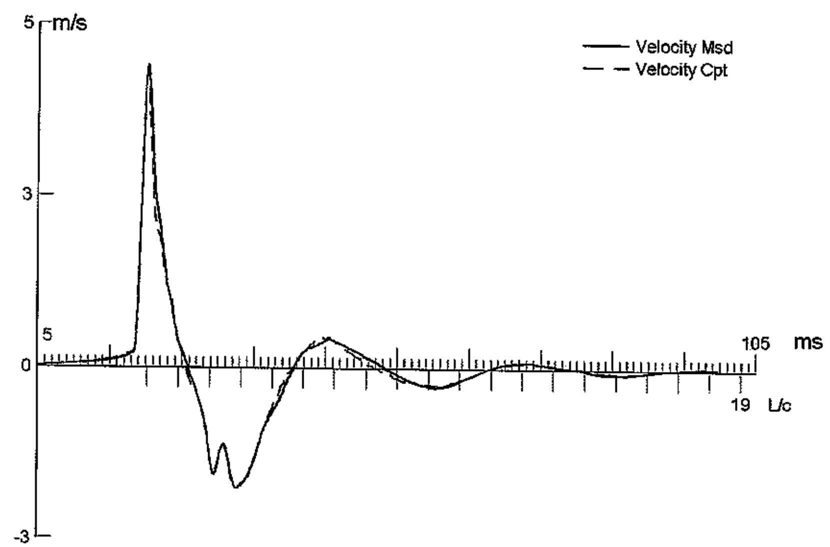
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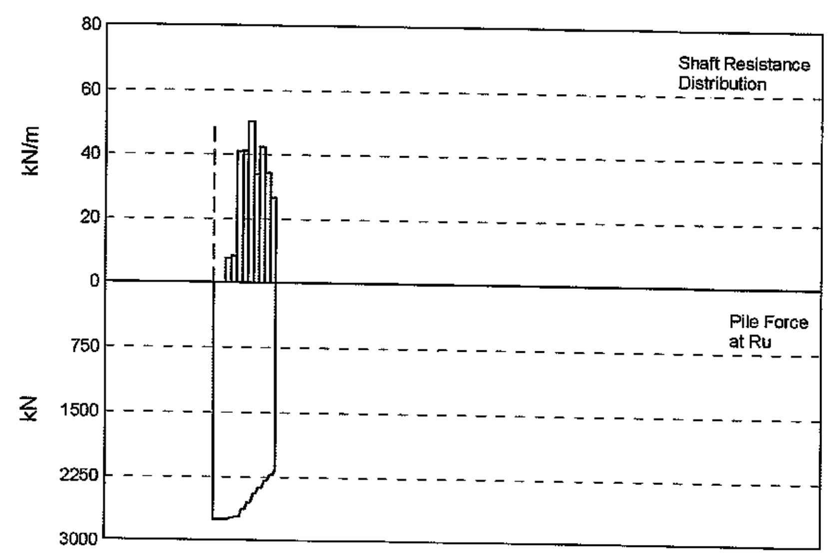
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kN/m
kN
Shaft Resistance
Distribution
Pile Force
at Ru

[redacted]

[redacted]-Pile2- (RSTRK) Test: 30-Jul-2008 17:57:
July 30, 2008; Blow: 9 CAPWAP(R) 2006-2
[redacted] OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2746.1; along Shaft 578.4; at Toe 2167.7 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2746.1					
1	2.0	2.0	0.0	2746.1	0.0	0.00	0.00	0.000	6.537
2	4.1	4.1	0.0	2746.1	0.0	0.00	0.00	0.000	6.535
3	6.1	6.1	15.5	2730.6	15.5	7.65	6.27	0.935	6.535
4	8.1	8.1	17.0	2713.6	32.5	8.39	6.87	0.935	6.535
5	10.1	10.1	83.2	2630.4	115.7	41.04	33.64	0.935	6.535
6	12.2	12.2	83.3	2547.1	199.0	41.09	33.68	0.935	6.535
7	14.2	14.2	102.7	2444.4	301.7	50.66	41.52	0.935	6.535
8	16.2	16.2	68.4	2376.0	370.1	33.74	27.66	0.935	6.535
9	18.2	18.2	85.5	2290.5	455.6	42.17	34.57	0.935	6.535
10	20.3	20.3	69.0	2221.5	524.6	34.04	27.90	0.935	5.652
11	22.3	22.3	53.8	2167.7	578.4	26.54	21.75	0.935	4.402
Avg. Shaft			52.6			25.94	21.26	0.935	6.232
Toe			2167.7				23302.34	0.289	4.371

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.844	0.978
Damping Type		Smith	
Unloading Quake	(% of loading quake)	47	130
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	40	
Resistance Gap (included in Toe Quake) (mm)			1.023
Soil Plug Weight	(kN)		0.35

CAPWAP match quality = 1.80 (Wave Up Match) ; RSA = 0
Observed: final set = 0.700 mm; blow count = 1429 b/m
Computed: final set = 0.068 mm; blow count = 14766 b/m

max. Top Comp. Stress = 178.0 MPa (T= 20.4 ms, max= 1.022 x Top)
max. Comp. Stress = 181.8 MPa (Z= 10.1 m, T= 22.2 ms)
max. Tens. Stress = -22.34 MPa (Z= 10.1 m, T= 36.4 ms)
max. Energy (EMX) = 20.85 kJ; max. Measured Top Displ. (DMX)=13.76 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2824.1	-108.7	178.0	-6.85	20.85	4.3	12.795
2	2.0	2824.0	-135.1	177.9	-8.51	20.73	4.3	12.533
3	3.0	2824.5	-170.6	178.0	-10.75	20.61	4.3	12.267
4	4.1	2827.1	-211.3	178.1	-13.31	20.49	4.3	11.993
5	5.1	2833.9	-247.4	178.6	-15.59	20.36	4.3	11.711
6	6.1	2846.9	-285.2	179.4	-17.97	20.21	4.3	11.418
7	7.1	2814.9	-296.9	177.4	-18.71	19.68	4.2	11.121
8	8.1	2837.7	-328.1	178.8	-20.67	19.53	4.2	10.818
9	9.1	2825.7	-329.2	178.1	-20.74	18.99	4.1	10.505
10	10.1	2885.3	-354.6	181.8	-22.34	18.81	4.0	10.171
11	11.2	2715.8	-271.3	171.1	-17.09	17.00	4.0	9.828
12	12.2	2774.1	-291.8	174.8	-18.39	16.76	3.9	9.444
13	13.2	2619.3	-203.9	165.0	-12.85	15.09	3.8	9.047
14	14.2	2679.1	-218.4	168.8	-13.76	14.81	3.7	8.629
15	15.2	2483.1	-181.1	156.5	-11.41	13.03	3.7	8.217
16	16.2	2525.6	-193.1	159.1	-12.17	12.70	3.6	7.761
17	17.2	2412.4	-185.4	152.0	-11.69	11.50	3.6	7.277
18	18.2	2460.4	-214.8	155.0	-13.54	11.03	3.5	6.751
19	19.3	2341.0	-193.1	147.5	-12.16	9.76	3.4	6.218
20	20.3	2451.9	-219.0	154.5	-13.80	9.28	3.3	5.660
21	21.3	2473.5	-201.8	155.9	-12.72	8.20	3.0	5.071
22	22.3	2754.0	-224.6	173.5	-14.15	8.04	2.7	4.426
Absolute	10.1			181.8			(T =	22.2 ms)
	10.1				-22.34		(T =	36.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3613.9	3426.4	3238.9	3051.4	2863.8	2676.3	2488.8	2301.3	2113.7	1926.2
RX	3613.9	3426.4	3238.9	3051.4	2863.8	2676.3	2489.6	2364.4	2290.9	2235.4
RU	3715.4	3538.1	3360.7	3183.3	3005.9	2828.6	2651.2	2473.8	2296.4	2119.1

RAU = 1293.8 (kN); RA2 = 1749.2 (kN)

Current CAPWAP Ru = 2746.1 (kN); Corresponding J(RP)= 0.46; J(RX) = 0.46

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.40	20.38	2819.2	2670.0	2807.6	13.759	0.653	0.700	20.8	2881.0

PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206874.8	77.300	1.220
22.30	158.70	206874.8	77.300	1.220

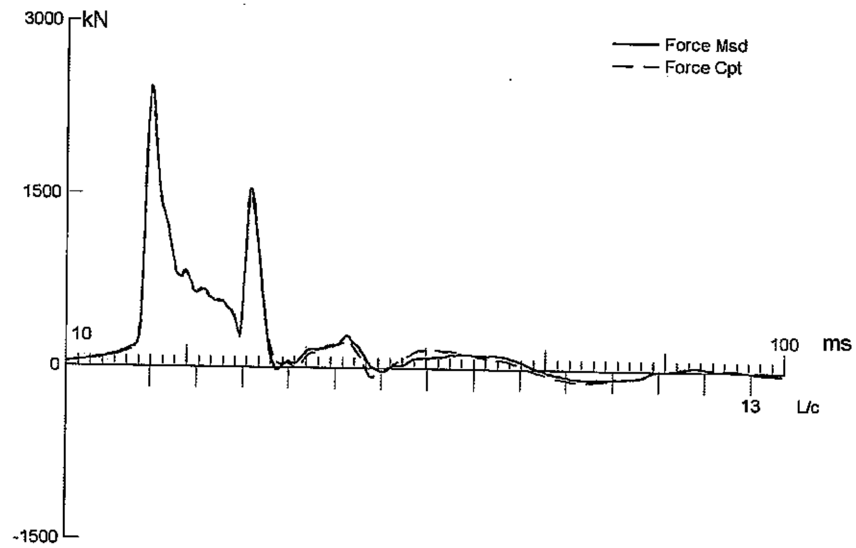
Toe Area 0.093 m^2

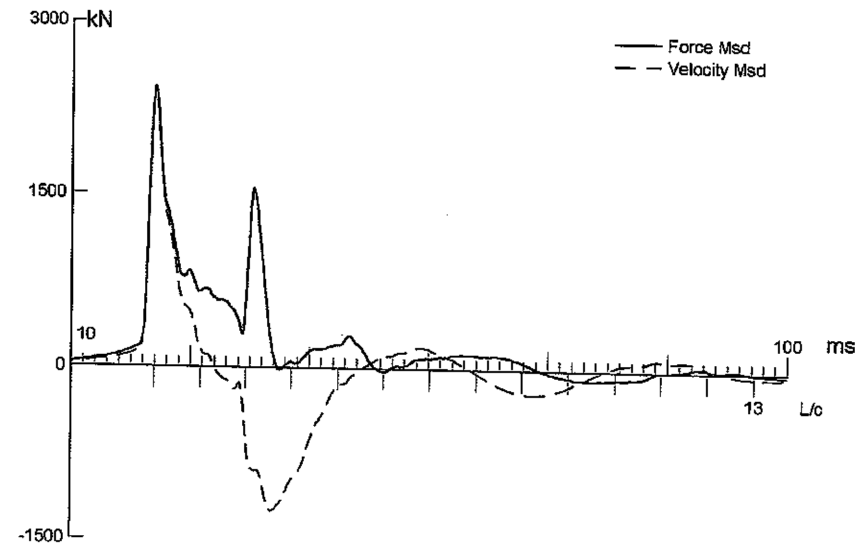
Top Segment Length 1.01 m, Top Impedance 640.86 kN/m/s

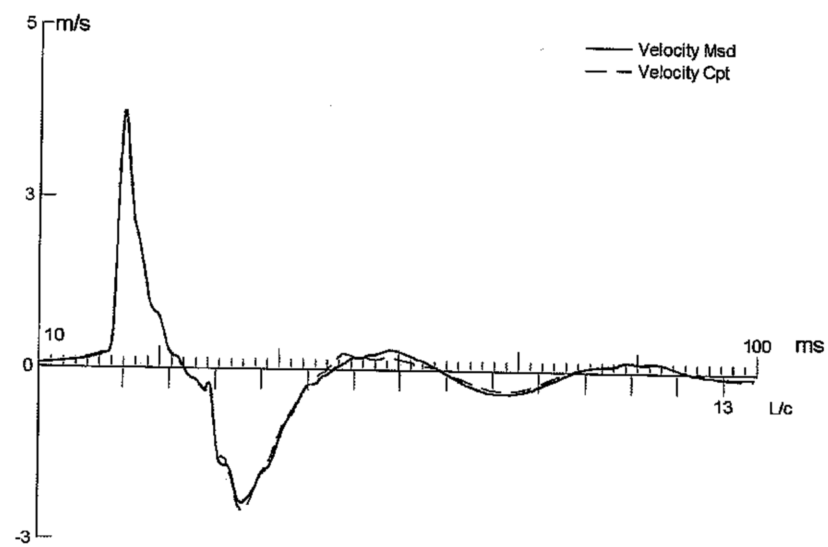
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 5123.0 m/s, 2L/c 8.7 ms

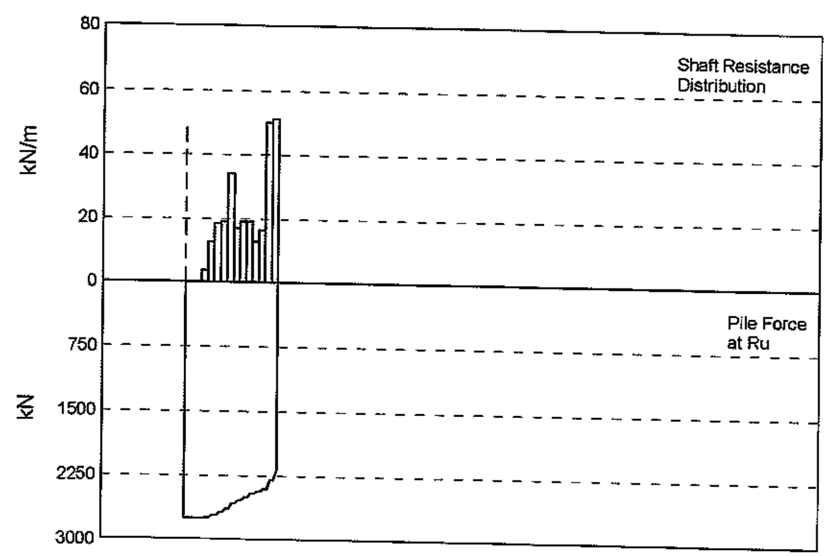
[redacted]-Pile5 (EOID); July 30, 2008; Blow: 29 (Test: 30-Jul-2008 15:16:); Delmag Hammer FS#4

13-Jan-2009
CAPWAP(R) 2006-2









[redacted]

[redacted]-Pile5 (EOID)
July 30, 2008; Blow: 29; Delmag Hammer FS#4
[redacted]

Test: 30-Jul-2008 15:16:
CAPWAP(R) 2006-2
OP: RB

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2750.0; along Shaft 560.0; at Toe 2190.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				2750.0					
1	3.1	3.1	0.0	2750.0	0.0	0.00	0.00	0.000	5.079
2	5.1	5.1	0.0	2750.0	0.0	0.00	0.00	0.000	5.080
3	7.2	7.2	7.6	2742.4	7.6	3.71	3.04	0.702	5.080
4	9.2	9.2	25.7	2716.7	33.3	12.55	10.28	0.702	5.080
5	11.3	11.3	38.0	2678.7	71.3	18.55	15.20	0.702	5.080
6	13.3	13.3	39.0	2639.7	110.3	19.04	15.60	0.702	5.080
7	15.4	15.4	69.5	2570.2	179.8	33.93	27.81	0.702	5.080
8	17.4	17.4	34.8	2535.4	214.6	16.99	13.92	0.702	5.080
9	19.5	19.5	39.0	2496.4	253.6	19.04	15.60	0.702	5.080
10	21.5	21.5	39.0	2457.4	292.6	19.04	15.60	0.702	5.080
11	23.6	23.6	26.2	2431.2	318.8	12.79	10.48	0.702	5.080
12	25.6	25.6	33.3	2397.9	352.1	16.25	13.32	0.702	5.080
13	27.7	27.7	102.9	2295.0	455.0	50.23	41.17	0.702	5.080
14	29.7	29.7	105.0	2190.0	560.0	51.25	42.01	0.702	3.951
Avg. Shaft			40.0			18.86	15.46	0.702	4.868
Toe			2190.0				23542.06	0.362	3.951

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.613	1.237
Damping Type		Smith	Smith
Unloading Quake	(% of loading quake)	44	109
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	15	
Resistance Gap (included in Toe Quake) (mm)			0.302
Soil Plug Weight	(kN)		0.60

CAPWAP match quality = 2.25 (Wave Up Match) ; RSA = 0
Observed: final set = 0.900 mm; blow count = 1111 b/m
Computed: final set = 0.100 mm; blow count = 9999 b/m

max. Top Comp. Stress = 153.2 MPa (T= 20.8 ms, max= 1.097 x Top)
max. Comp. Stress = 168.0 MPa (Z= 29.7 m, T= 26.8 ms)
max. Tens. Stress = -28.90 MPa (Z= 27.7 m, T= 42.6 ms)
max. Energy (EMX) = 18.73 kJ; max. Measured Top Displ. (DMX)=13.49 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2431.5	-111.3	153.2	-7.01	18.73	3.8	13.351
2	2.0	2431.0	-116.9	153.2	-7.36	18.66	3.8	13.153
4	4.1	2444.9	-127.4	154.1	-8.03	18.53	3.7	12.744
6	6.1	2469.3	-169.7	155.6	-10.69	18.38	3.7	12.322
8	8.2	2479.8	-212.2	156.3	-13.37	18.08	3.6	11.944
10	10.2	2454.2	-229.8	154.6	-14.48	17.42	3.6	11.571
12	12.3	2406.3	-234.3	151.6	-14.77	16.51	3.5	11.141
14	14.3	2373.4	-240.1	149.6	-15.13	15.62	3.4	10.702
16	16.4	2281.5	-225.6	143.8	-14.22	14.24	3.3	10.203
17	17.4	2301.8	-252.5	145.0	-15.91	14.09	3.3	9.911
18	18.4	2250.1	-240.3	141.8	-15.14	13.36	3.3	9.593
19	19.5	2272.1	-278.5	143.2	-17.55	13.19	3.2	9.267
20	20.5	2215.3	-314.8	139.6	-19.83	12.46	3.2	8.953
21	21.5	2235.5	-364.0	140.9	-22.94	12.27	3.2	8.614
22	22.5	2179.8	-391.4	137.4	-24.66	11.53	3.2	8.222
23	23.6	2196.3	-422.0	138.4	-26.59	11.19	3.1	7.783
24	24.6	2167.2	-423.8	136.6	-26.71	10.47	3.1	7.264
25	25.6	2190.2	-430.3	138.0	-27.11	9.95	3.1	6.699
26	26.6	2123.6	-427.1	133.8	-26.91	9.04	3.1	6.096
27	27.7	2164.2	-458.6	136.4	-28.90	8.38	3.3	5.447
28	28.7	2268.7	-438.9	143.0	-27.65	6.98	3.2	4.772
29	29.7	2666.9	-455.5	168.0	-28.70	6.47	2.7	4.053
Absolute	29.7			168.0			(T =	26.8 ms)
	27.7				-28.90		(T =	42.6 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	3171.4	2997.4	2823.3	2649.2	2475.2	2301.1	2127.0	1953.0	1778.9	1604.8
RX	3276.7	3151.1	3042.0	2932.8	2856.1	2780.4	2704.7	2629.1	2571.2	2520.5
RU	3309.7	3149.5	2989.2	2829.0	2668.8	2508.5	2348.3	2188.0	2027.8	1867.6

RAU = 1101.7 (kN); RA2 = 1650.4 (kN)

Current CAPWAP Ru = 2750.0 (kN); Corresponding J(RP)= 0.24; J(RX) = 0.54

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
3.82	20.59	2449.5	2462.6	2462.6	13.494	-1.025	0.900	18.9	2622.7

PILE PROFILE AND PILE MODEL

Depth m	Area cm²	E-Modulus MPa	Spec. Weight kN/m³	Perim. m
0.00	158.70	206874.8	77.300	1.220
29.70	158.70	206874.8	77.300	1.220

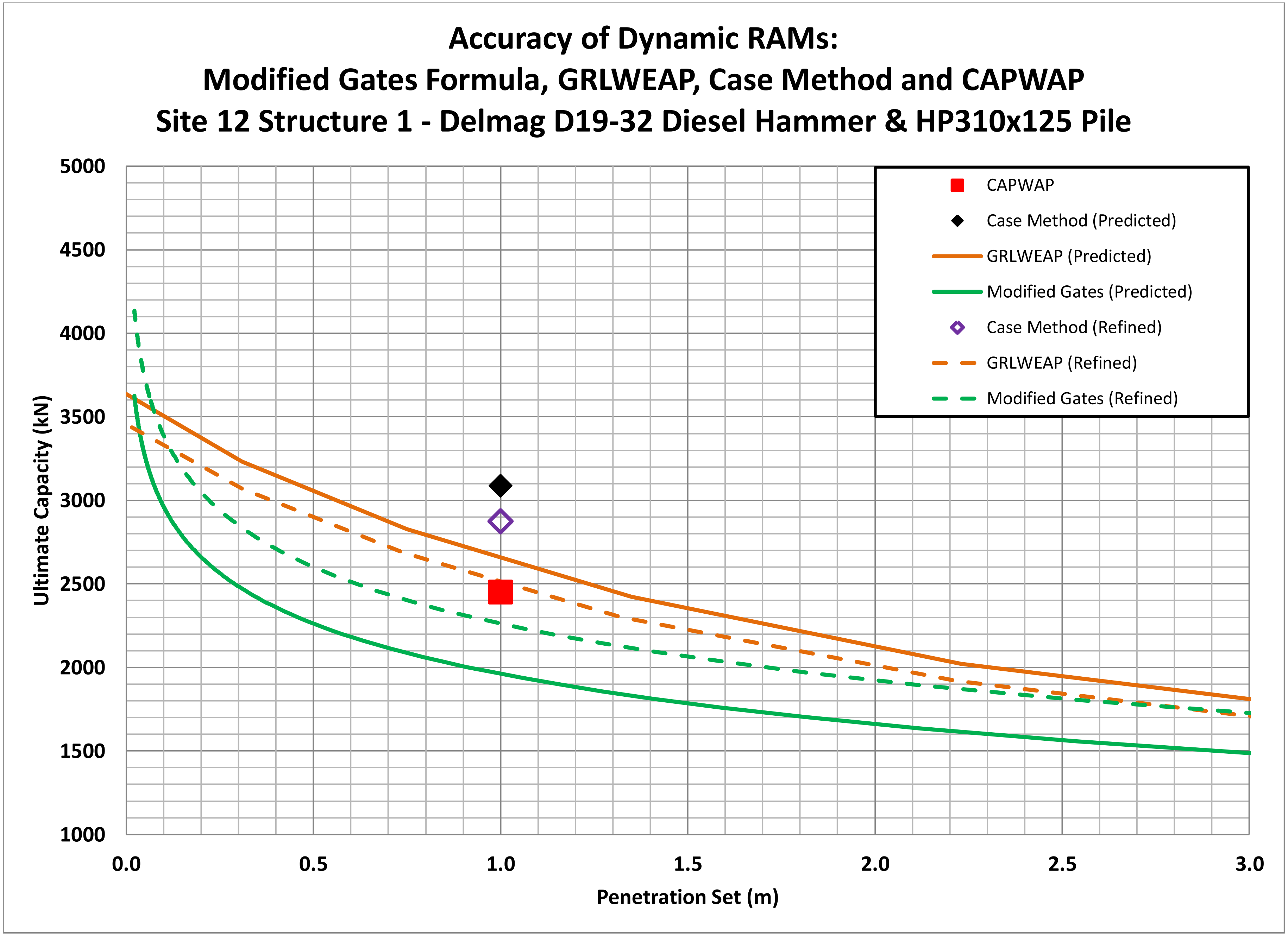
Toe Area 0.093 m^2

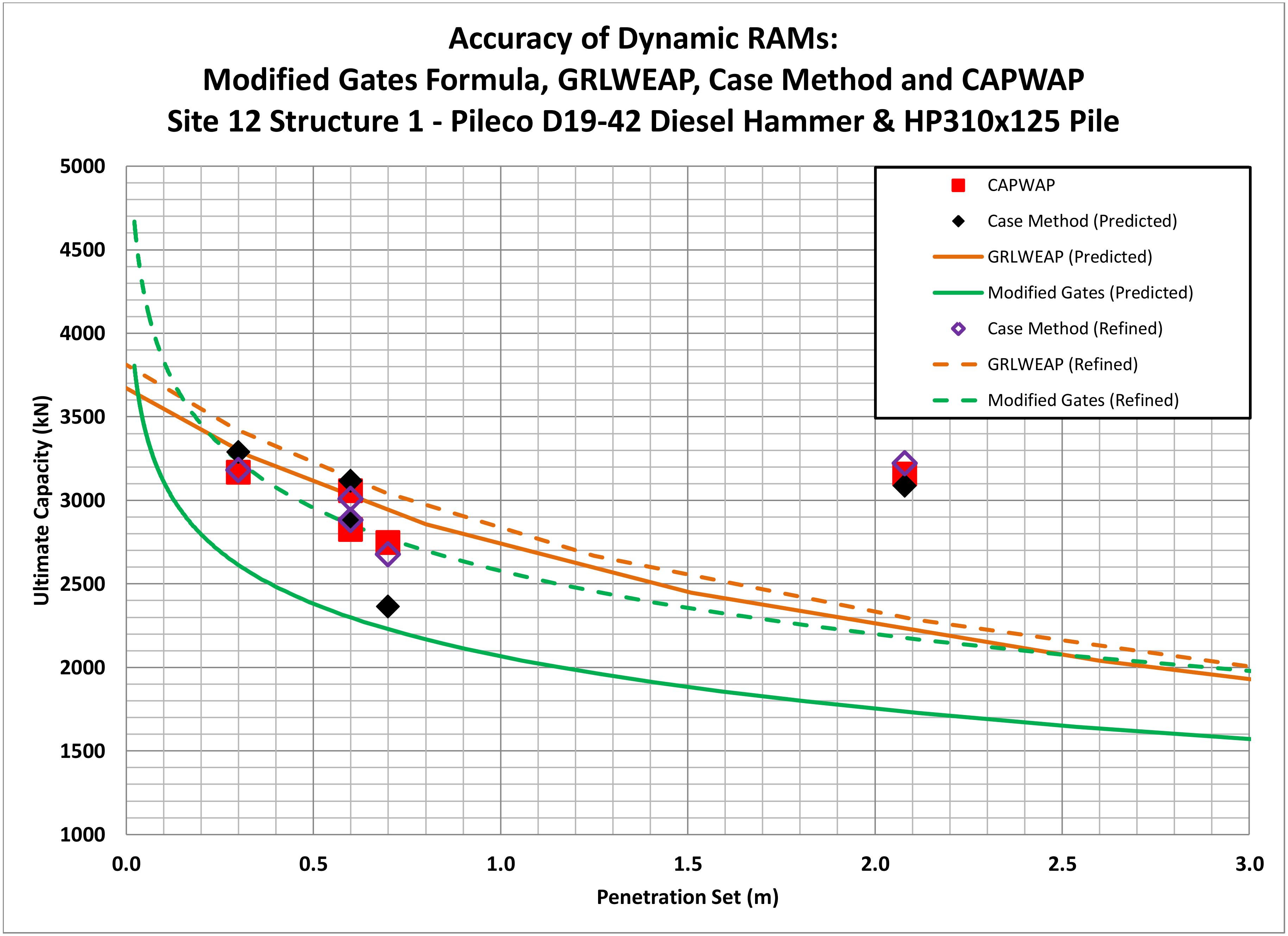
Top Segment Length 1.02 m, Top Impedance 640.86 kN/m/s

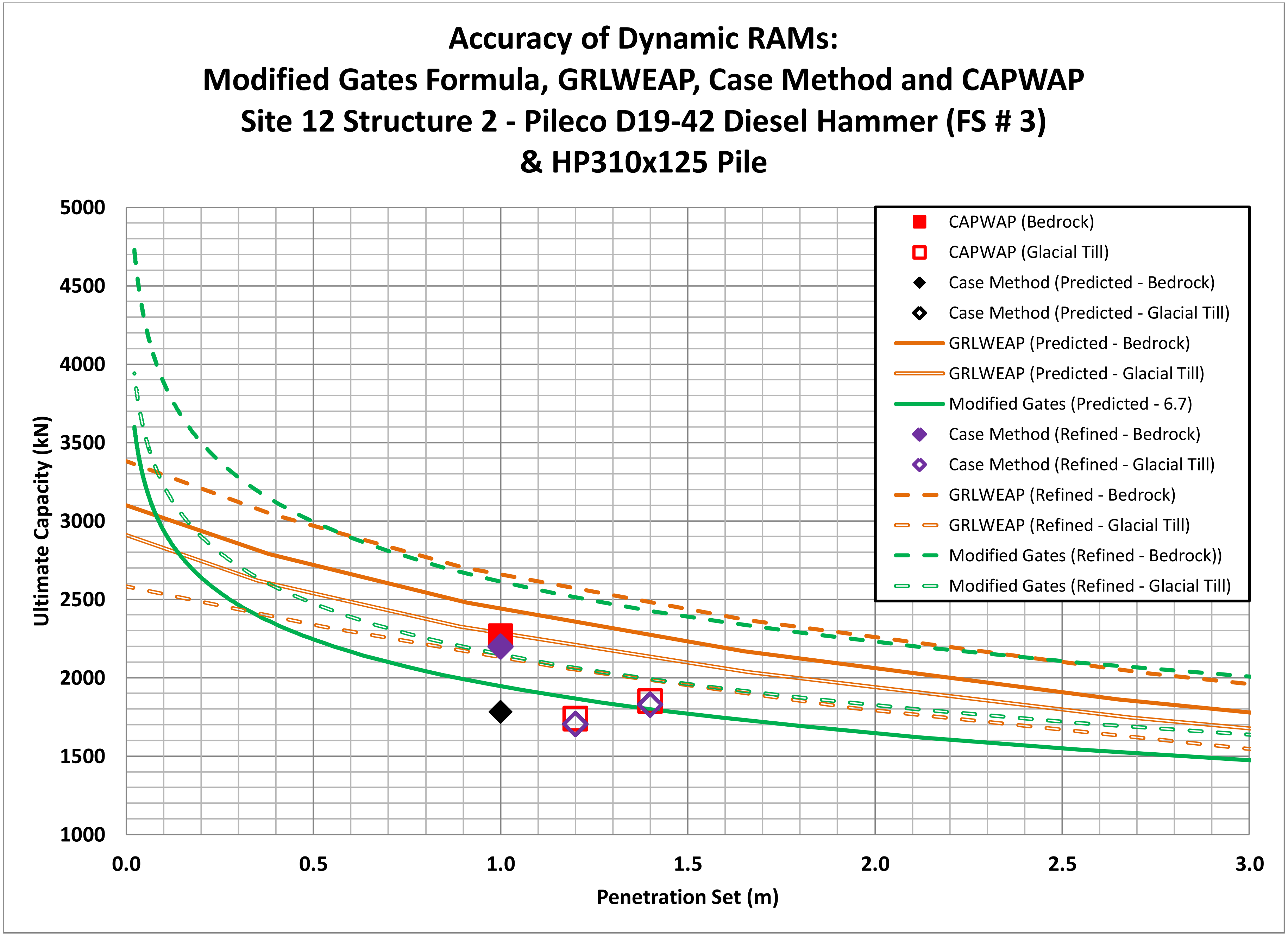
Pile Damping 1.0 %, Time Incr 0.200 ms, Wave Speed 5123.0 m/s, 2L/c 11.6 ms

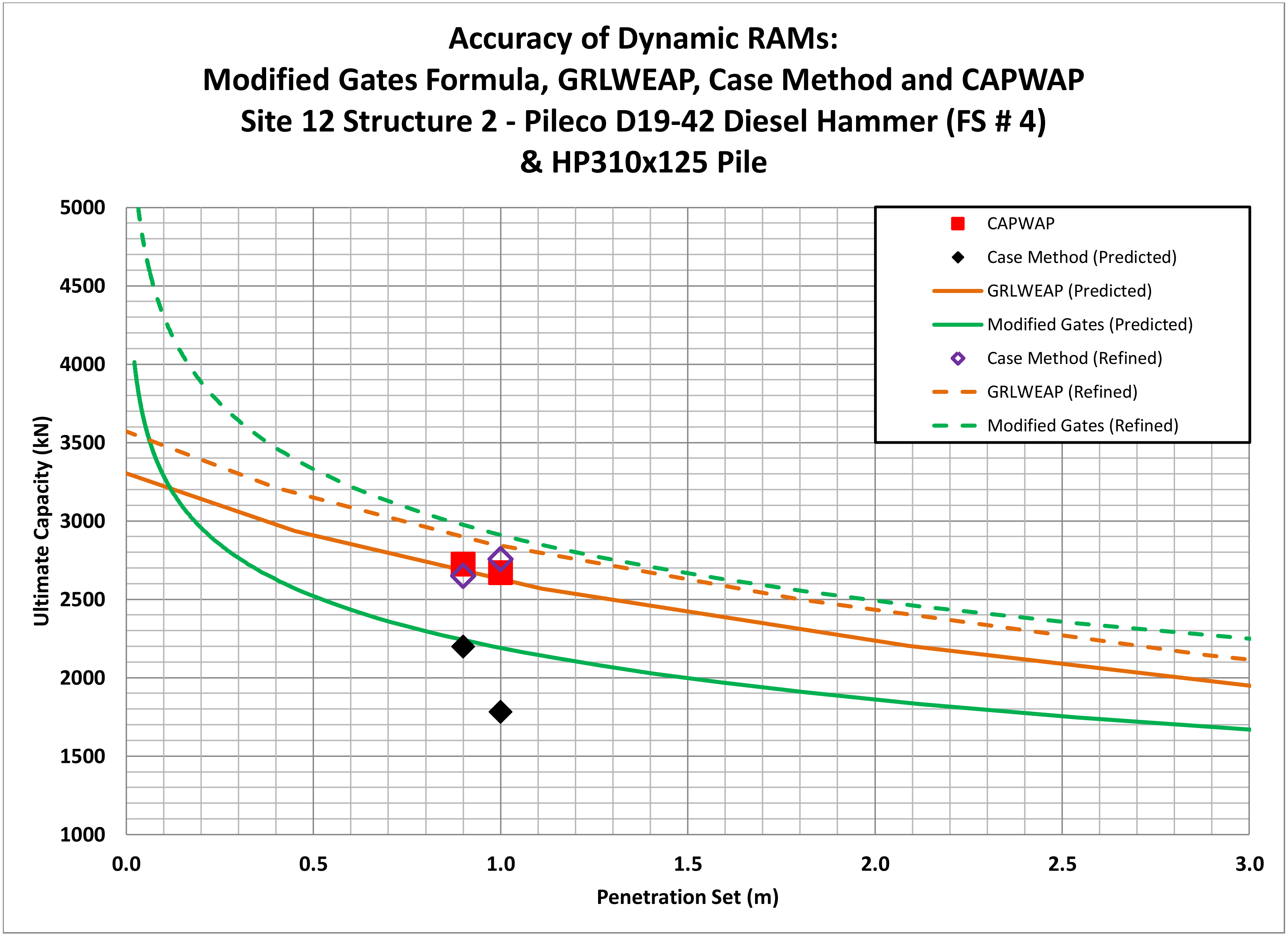
Appendix C:

Dynamic RAM Correlation Figures

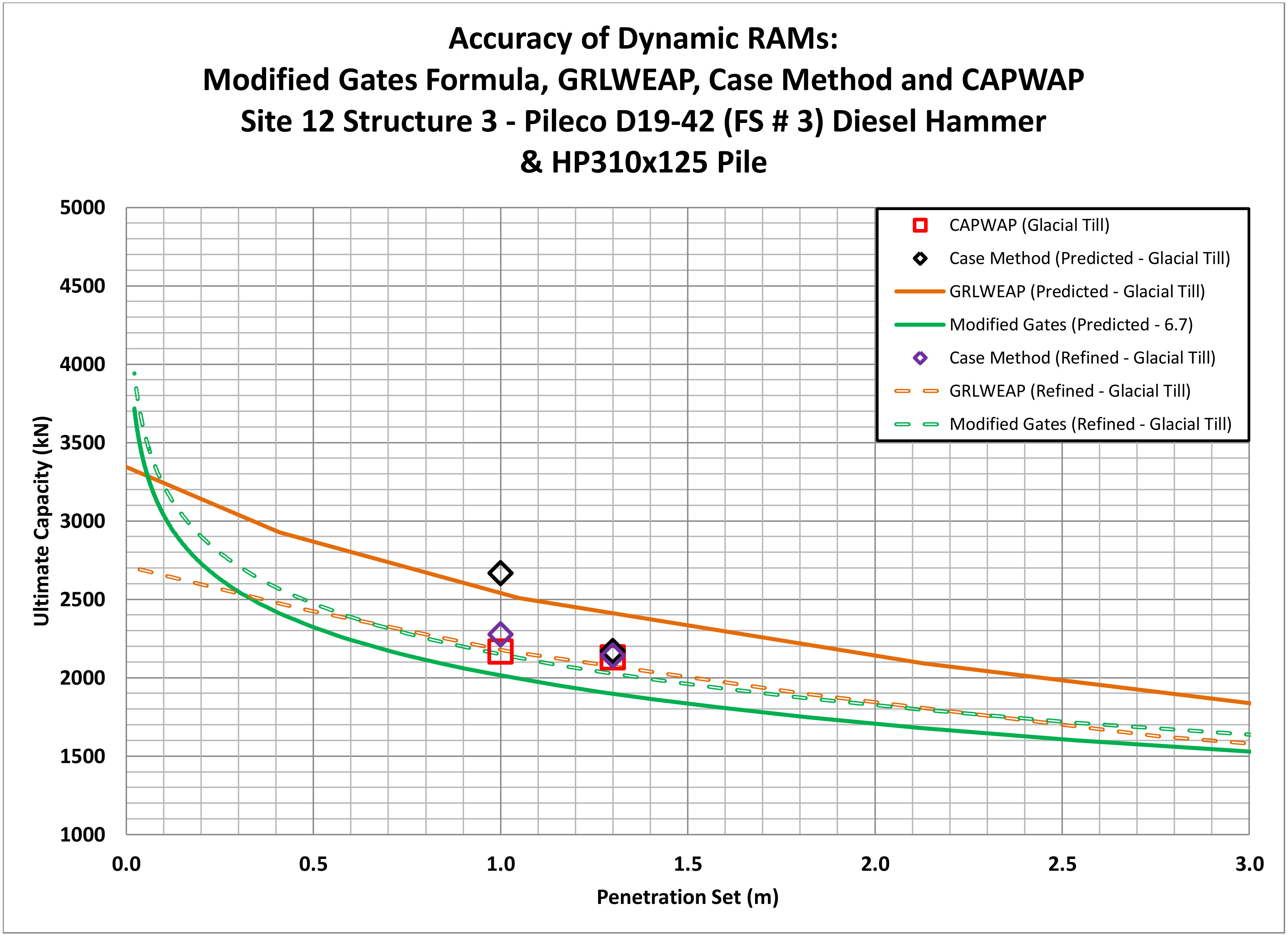

Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 1 - Delmag D19-32 Diesel Hammer & HP310x125 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

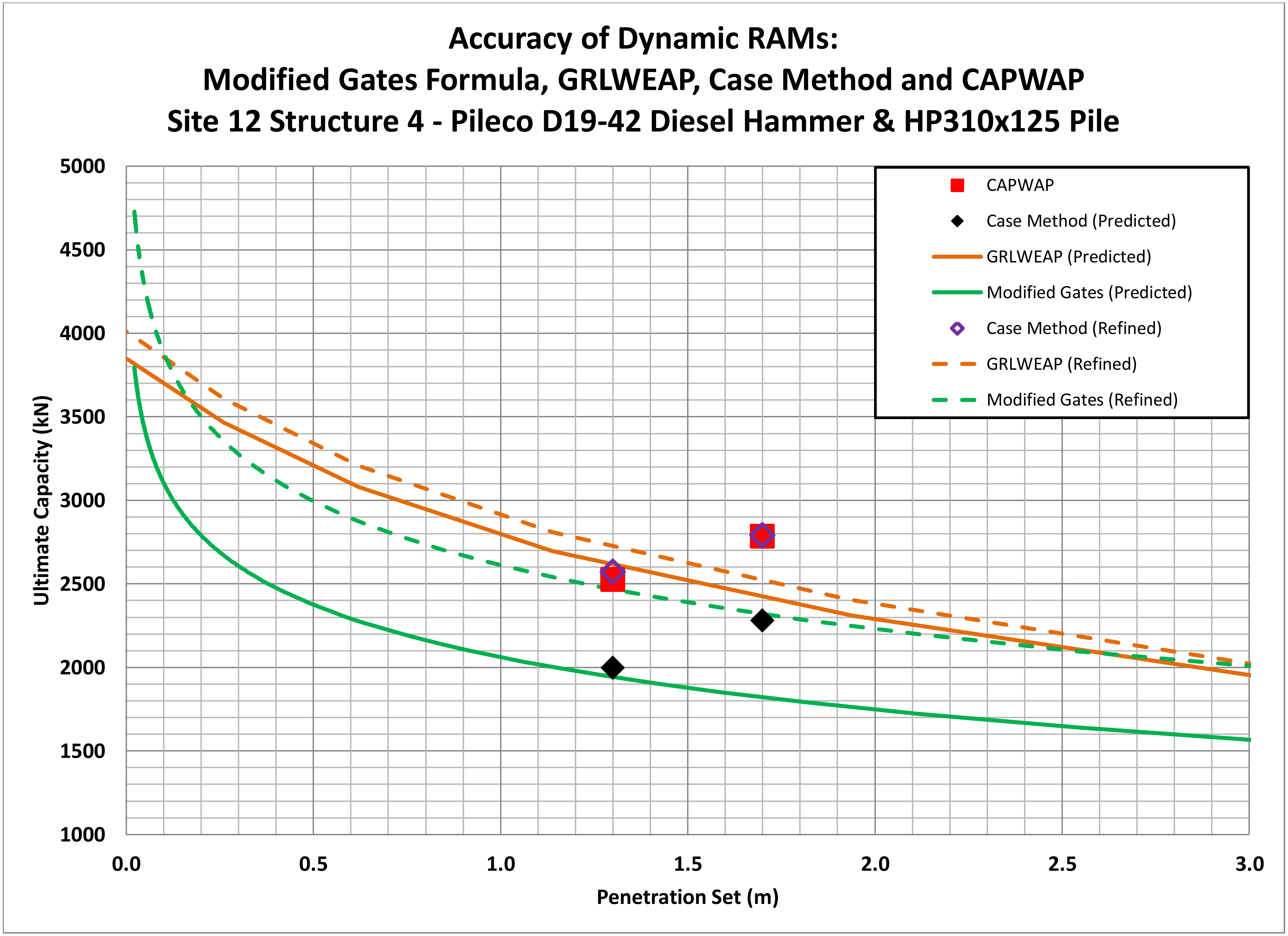

Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 1 - Pileco D19-42 Diesel Hammer & HP310x125 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

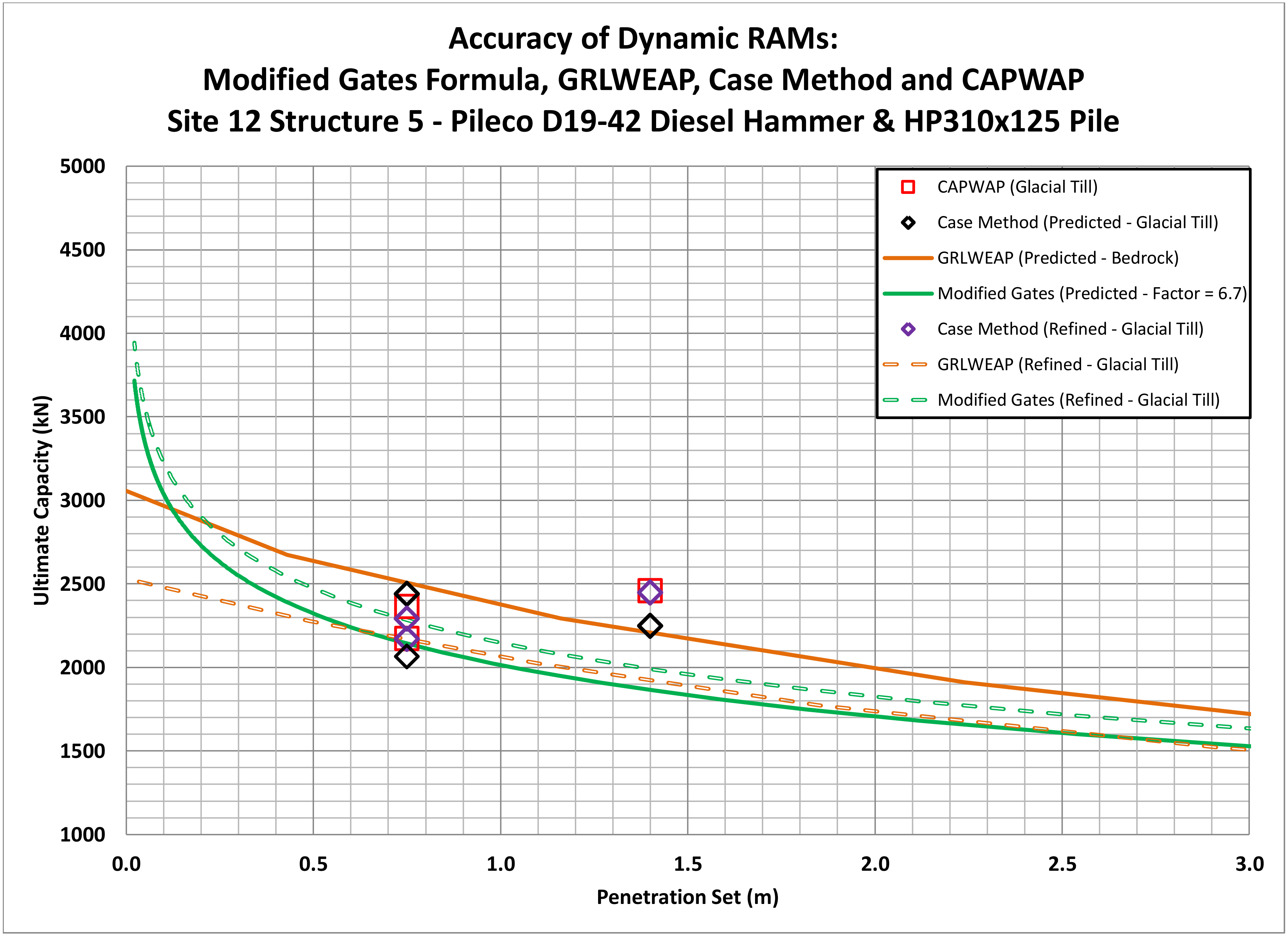

Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 2 - Pileco D19-42 Diesel Hammer (FS # 3)
& HP310x125 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP (Bedrock)
CAPWAP (Glacial Till)
Case Method (Predicted - Bedrock)
Case Method (Predicted - Glacial Till)
GRLWEAP (Predicted - Bedrock)
GRLWEAP (Predicted - Glacial Till)
Modified Gates (Predicted - 6.7)
Case Method (Refined - Bedrock)
Case Method (Refined - Glacial Till)
GRLWEAP (Refined - Bedrock)
GRLWEAP (Refined - Glacial Till)
Modified Gates (Refined - Bedrock))
Modified Gates (Refined - Glacial Till)



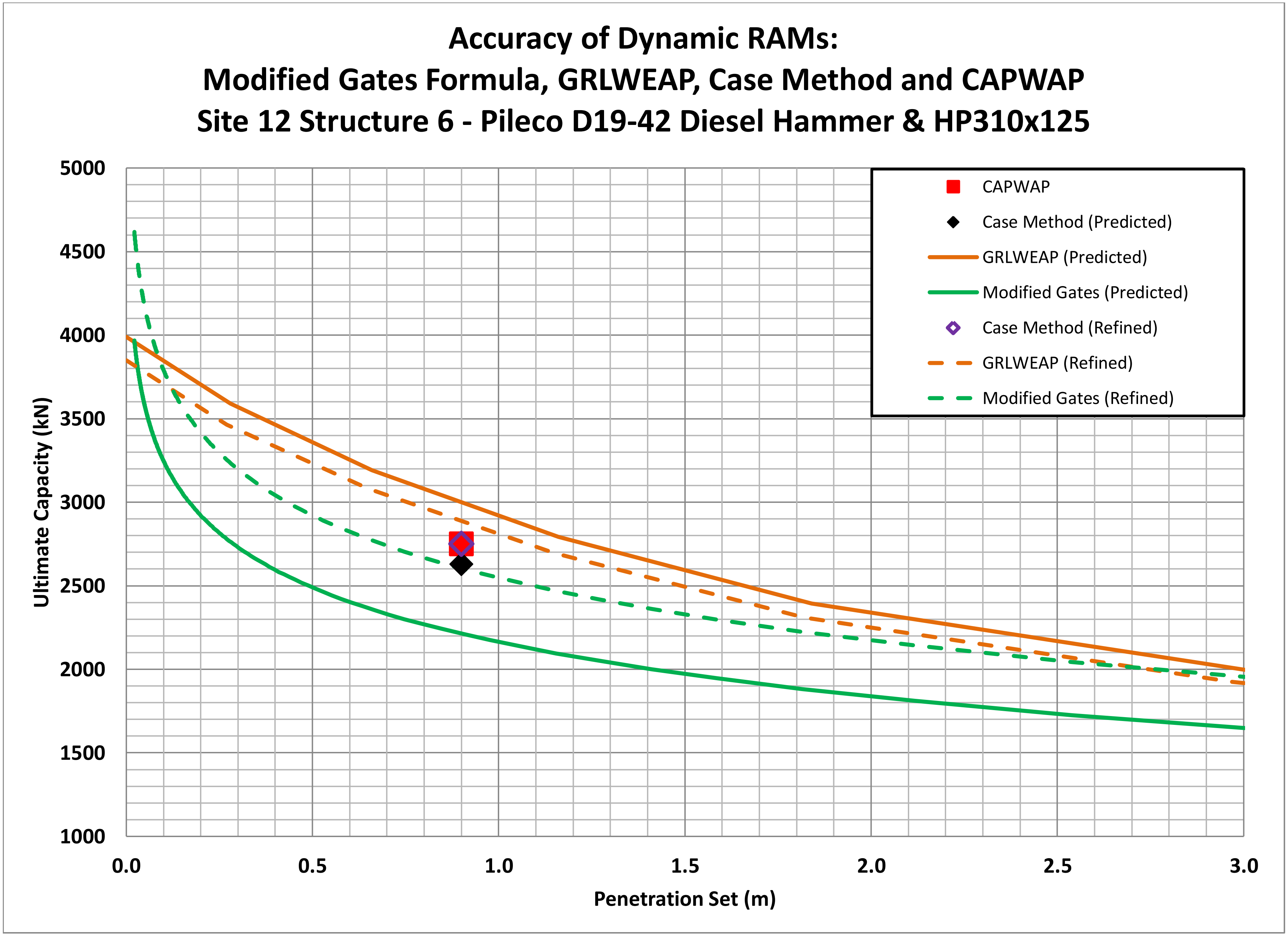
Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 2 - Pileco D19-42 Diesel Hammer (FS # 4)
& HP310x125 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

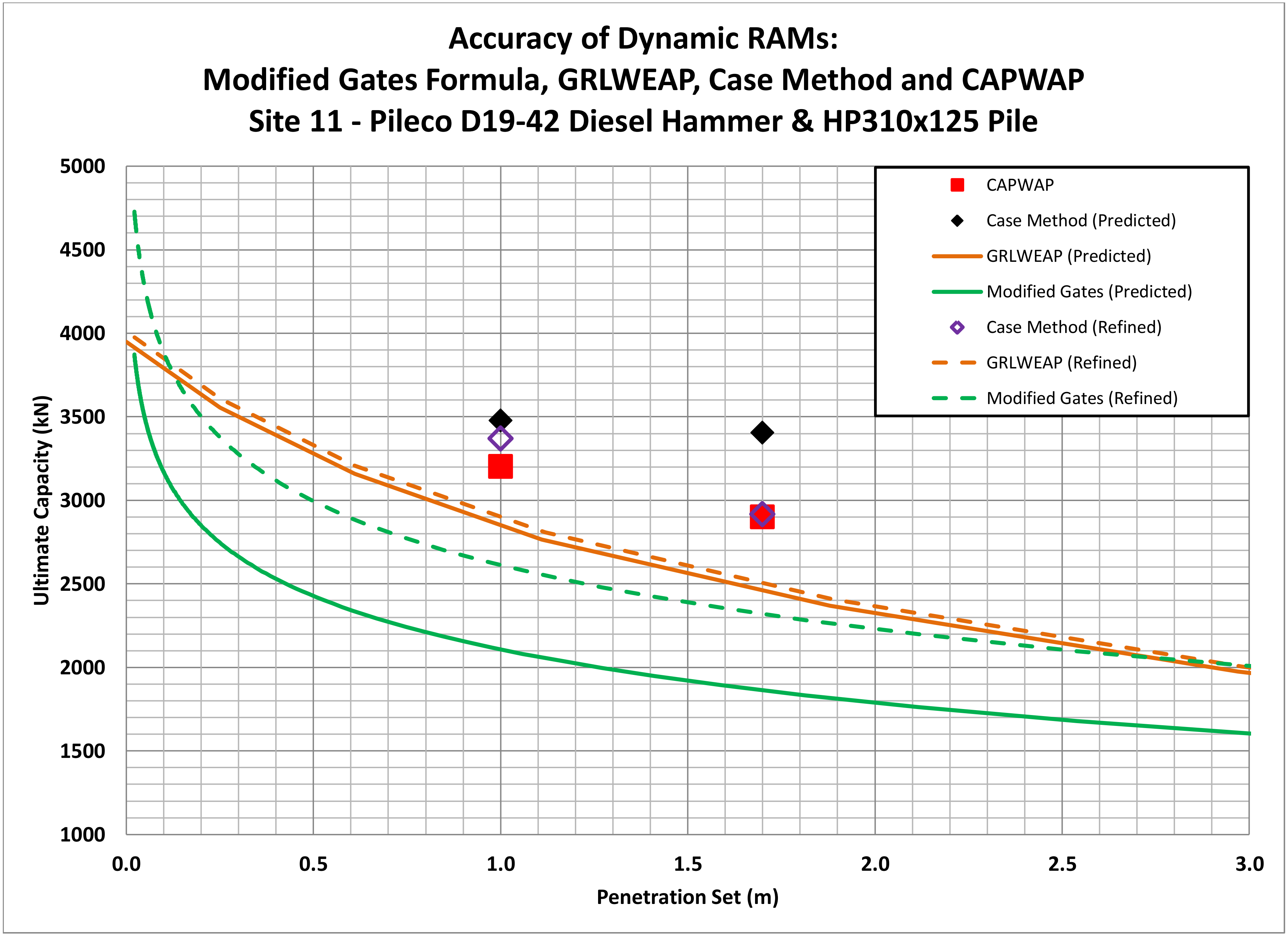

Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 3 - Pileco D19-42 (FS # 3) Diesel Hammer
& HP310x125 Pile
Ultimate Capacity (kN)
Penetration Set (m)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
CAPWAP (Glacial Till)
Case Method (Predicted - Glacial Till)
GRLWEAP (Predicted - Glacial Till)
Modified Gates (Predicted - 6.7)
Case Method (Refined - Glacial Till)
GRLWEAP (Refined - Glacial Till)
Modified Gates (Refined - Glacial Till)


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 4 - Pileco D19-42 Diesel Hammer & HP310x125 Pile
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)

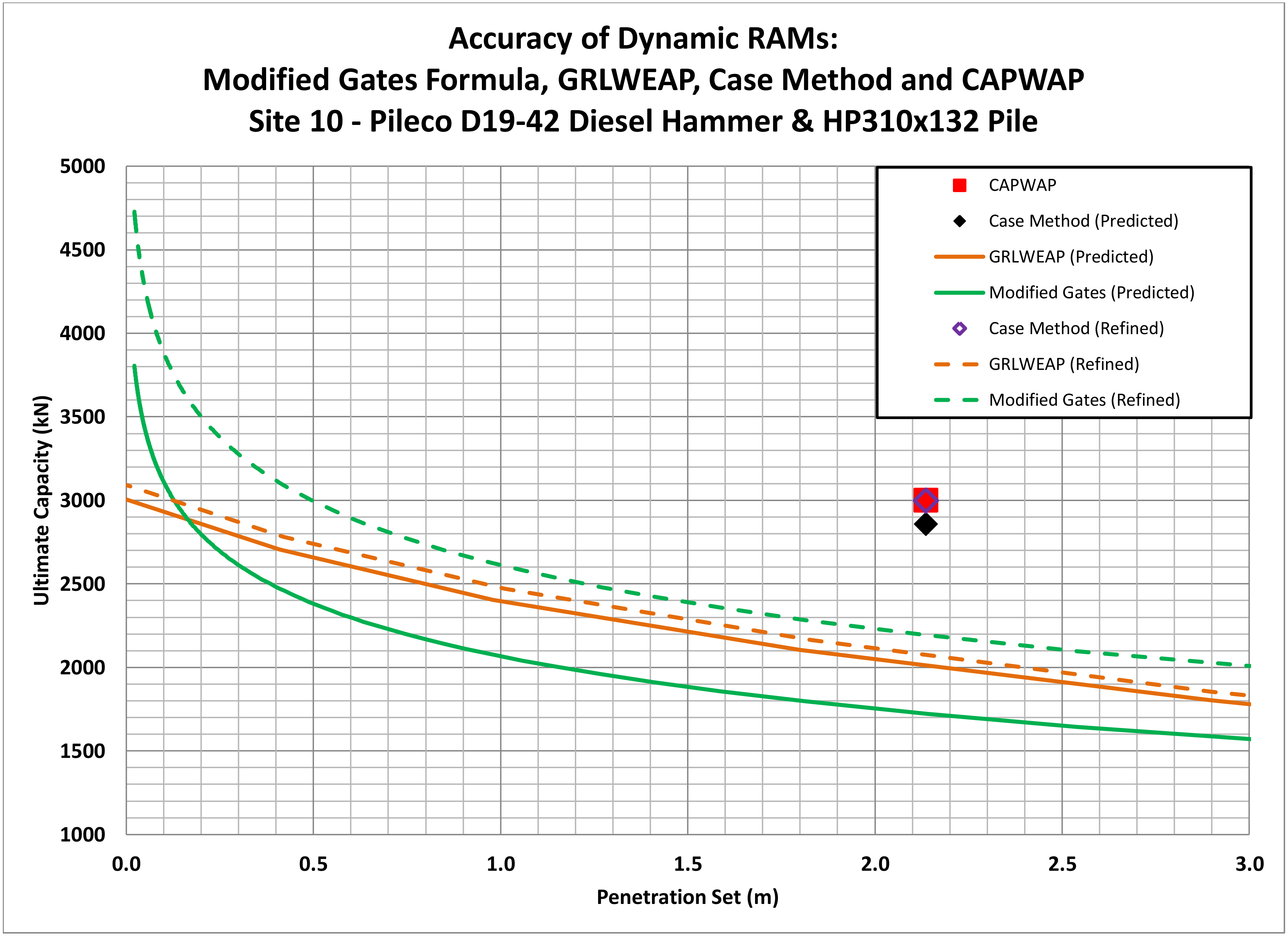


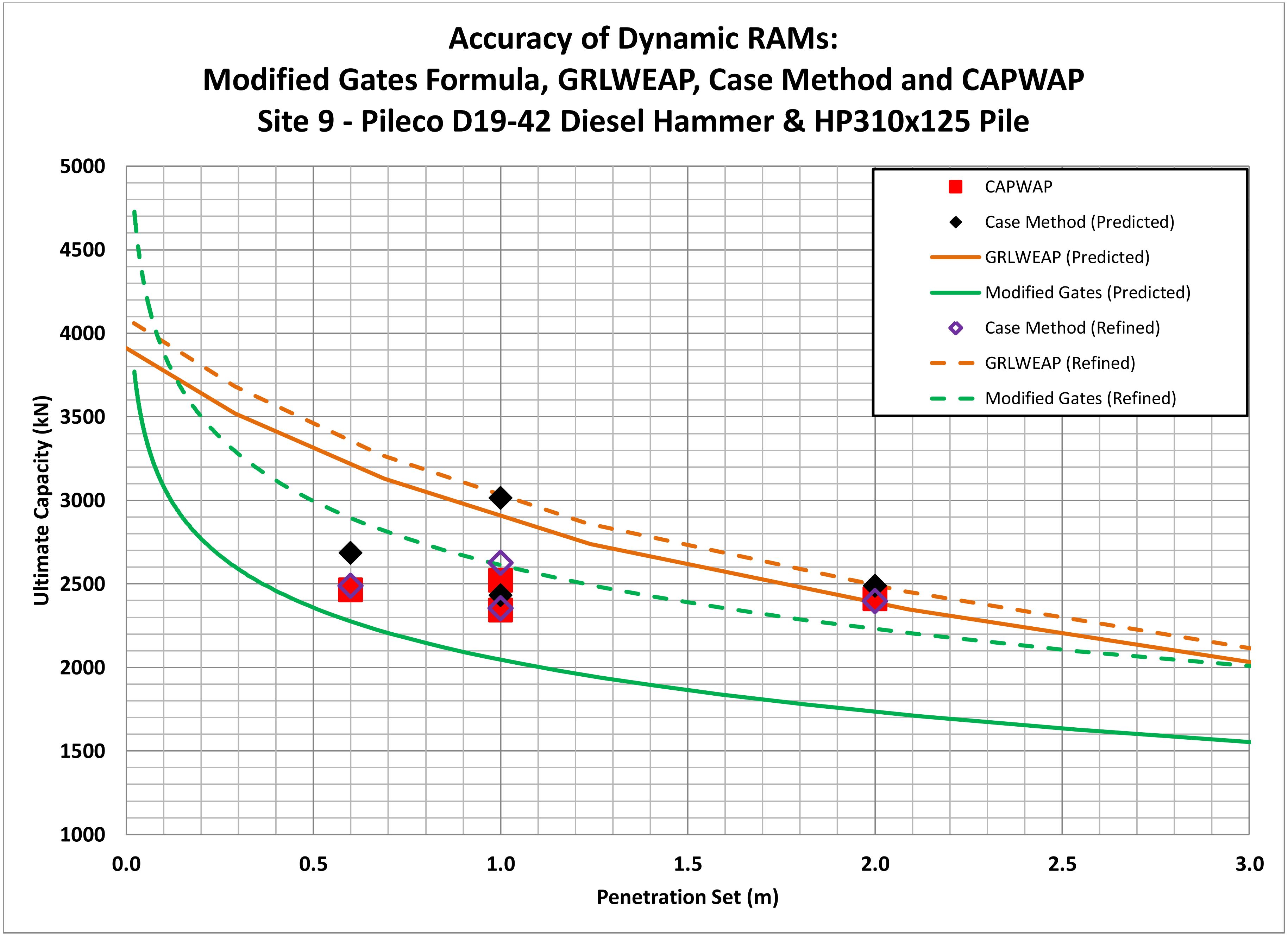
Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 5 - Pileco D19-42 Diesel Hammer & HP310x125 Pile
CAPWAP (Glacial Till)
Case Method (Predicted - Glacial Till)
GRLWEAP (Predicted - Bedrock)
Modified Gates (Predicted - Factor = 6.7)
Case Method (Refined - Glacial Till)
GRLWEAP (Refined - Glacial Till)
Modified Gates (Refined - Glacial Till)
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 12 Structure 6 - Pileco D19-42 Diesel Hammer & HP310x125
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

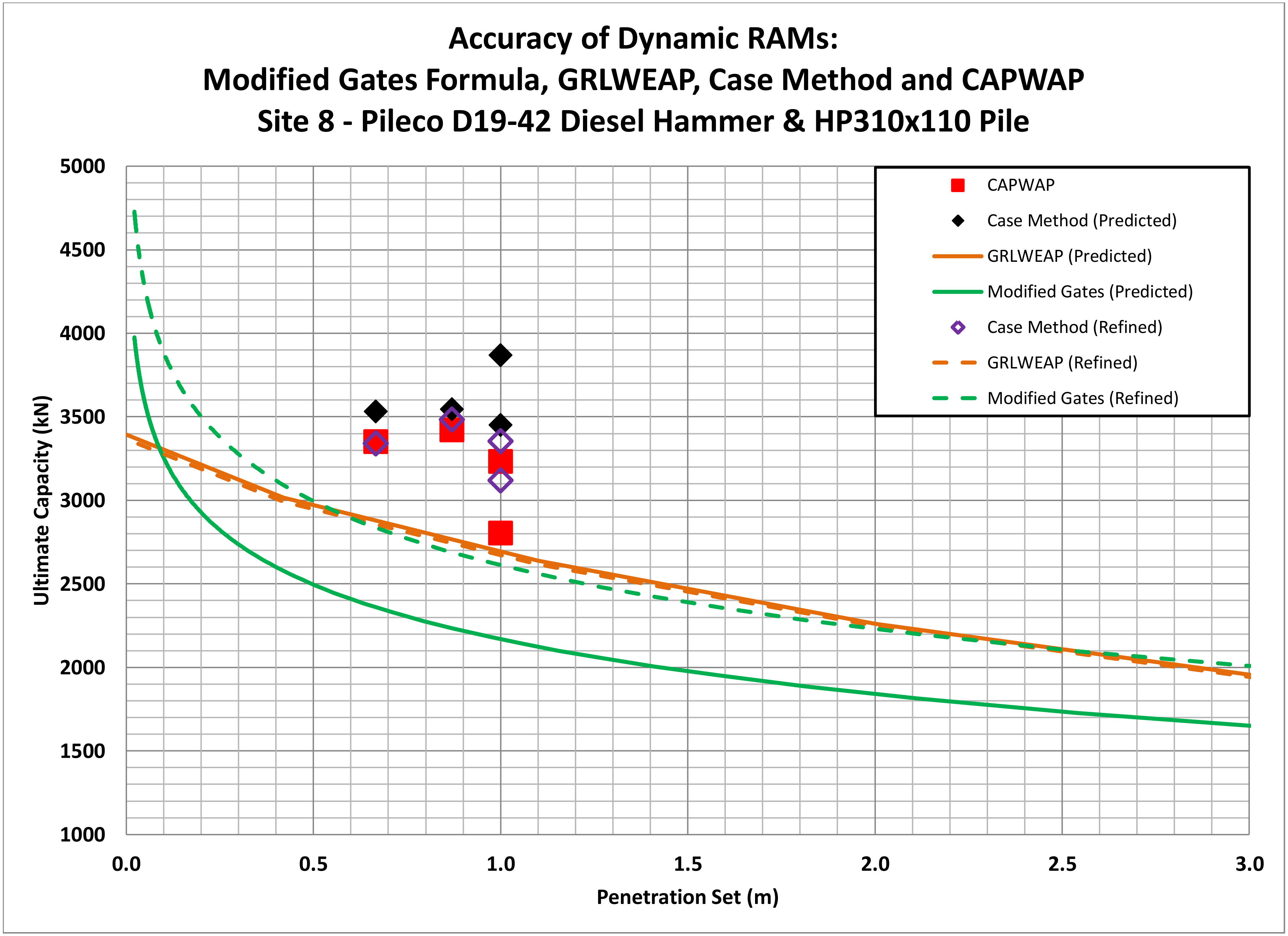


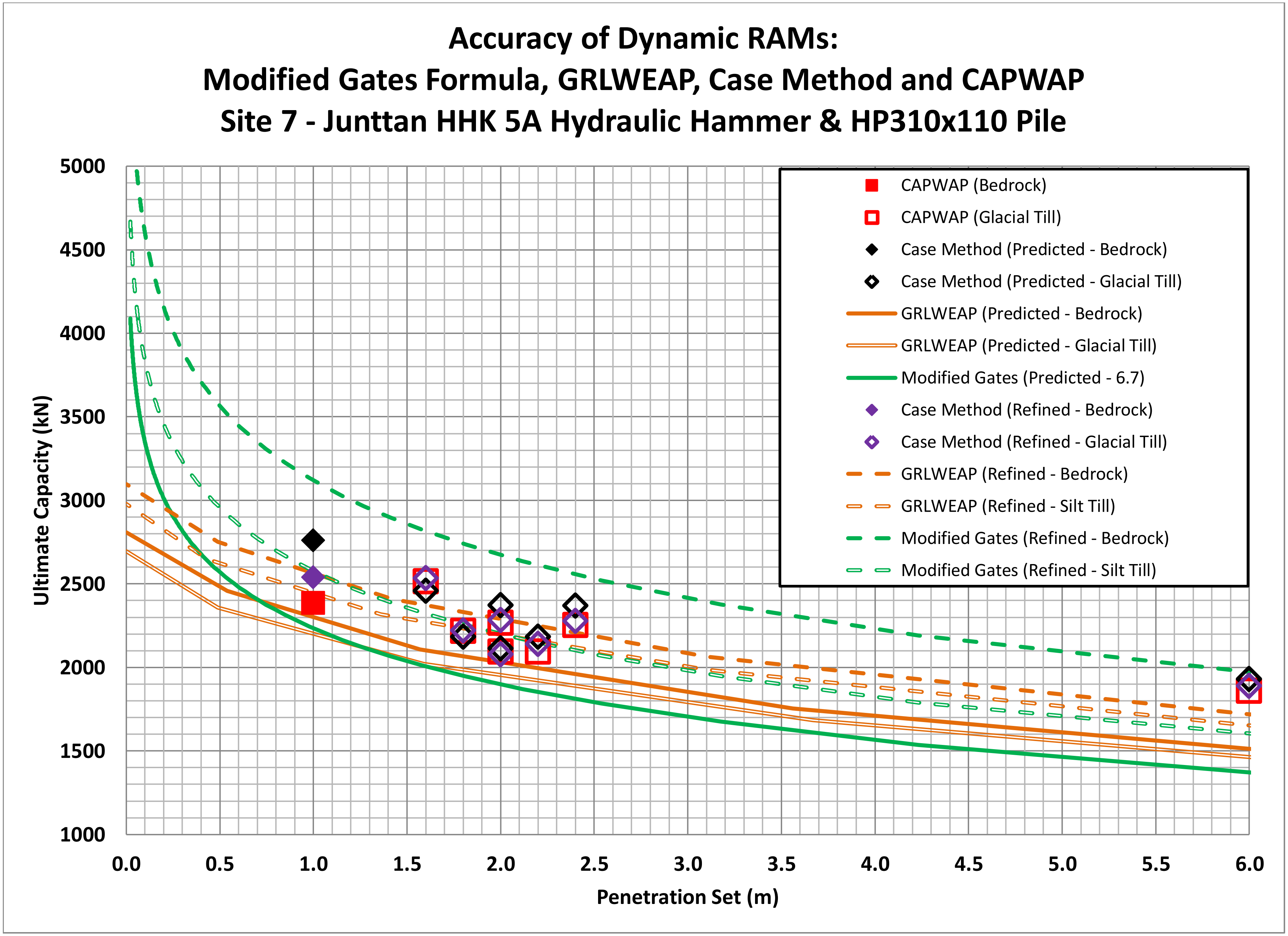
Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 11 - Pileco D19-42 Diesel Hammer & HP310x125 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 10 - Pileco D19-42 Diesel Hammer & HP310x132 Pile
Ultimate Capacity (kN)
Penetration Set (m)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

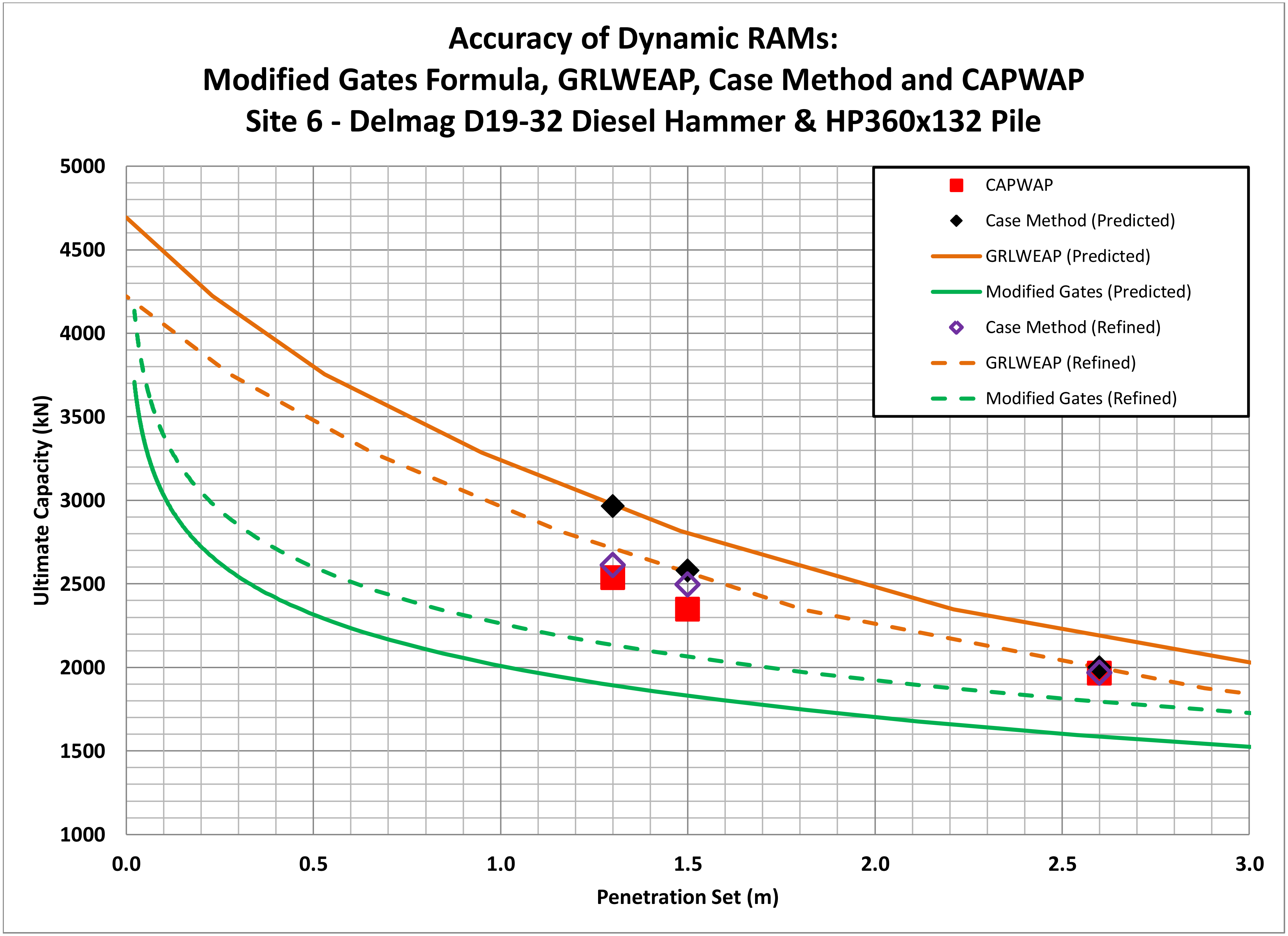


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 9 - Pileco D19-42 Diesel Hammer & HP310x125 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

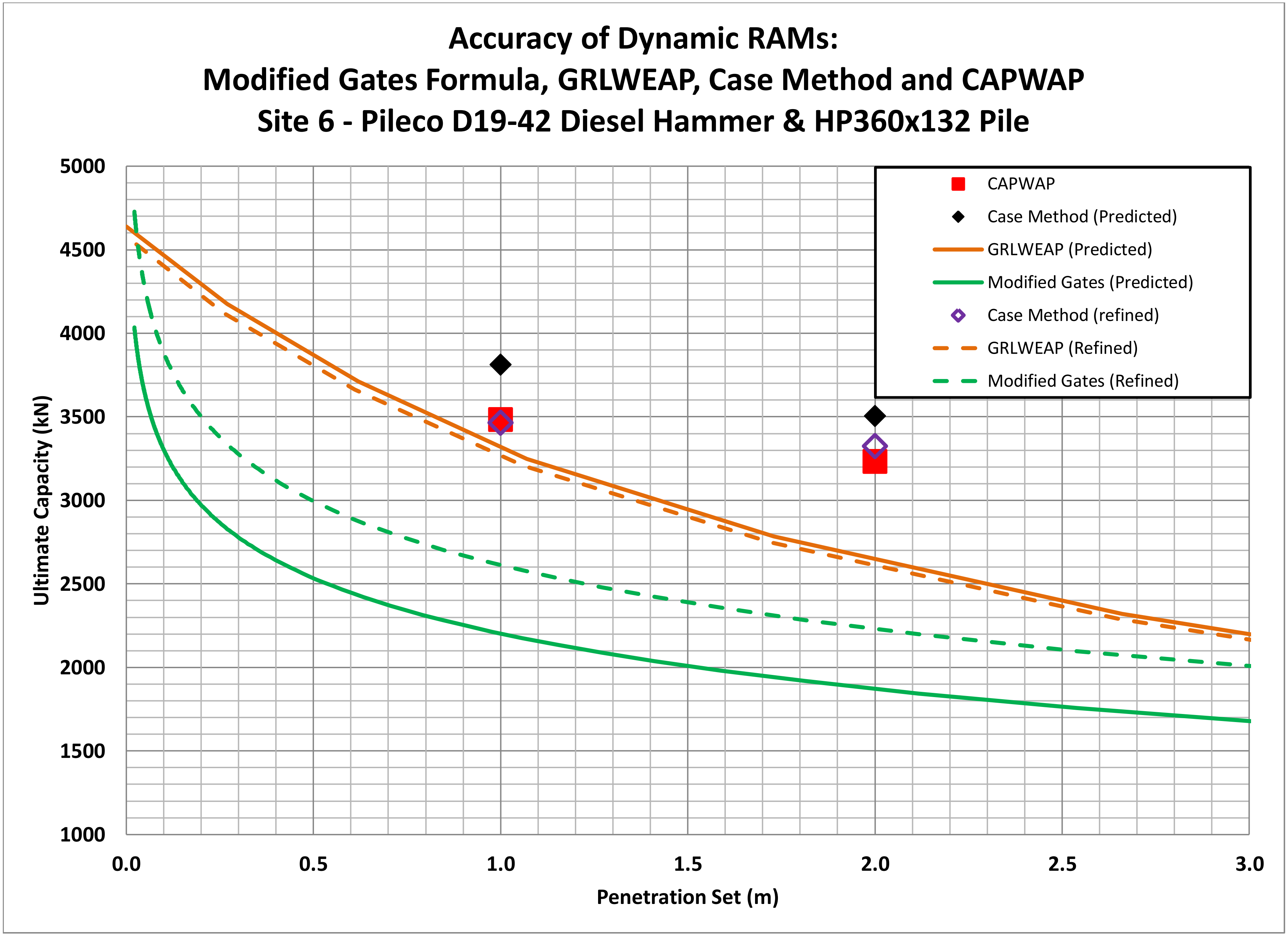

Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 8 - Pileco D19-42 Diesel Hammer & HP310x110 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)



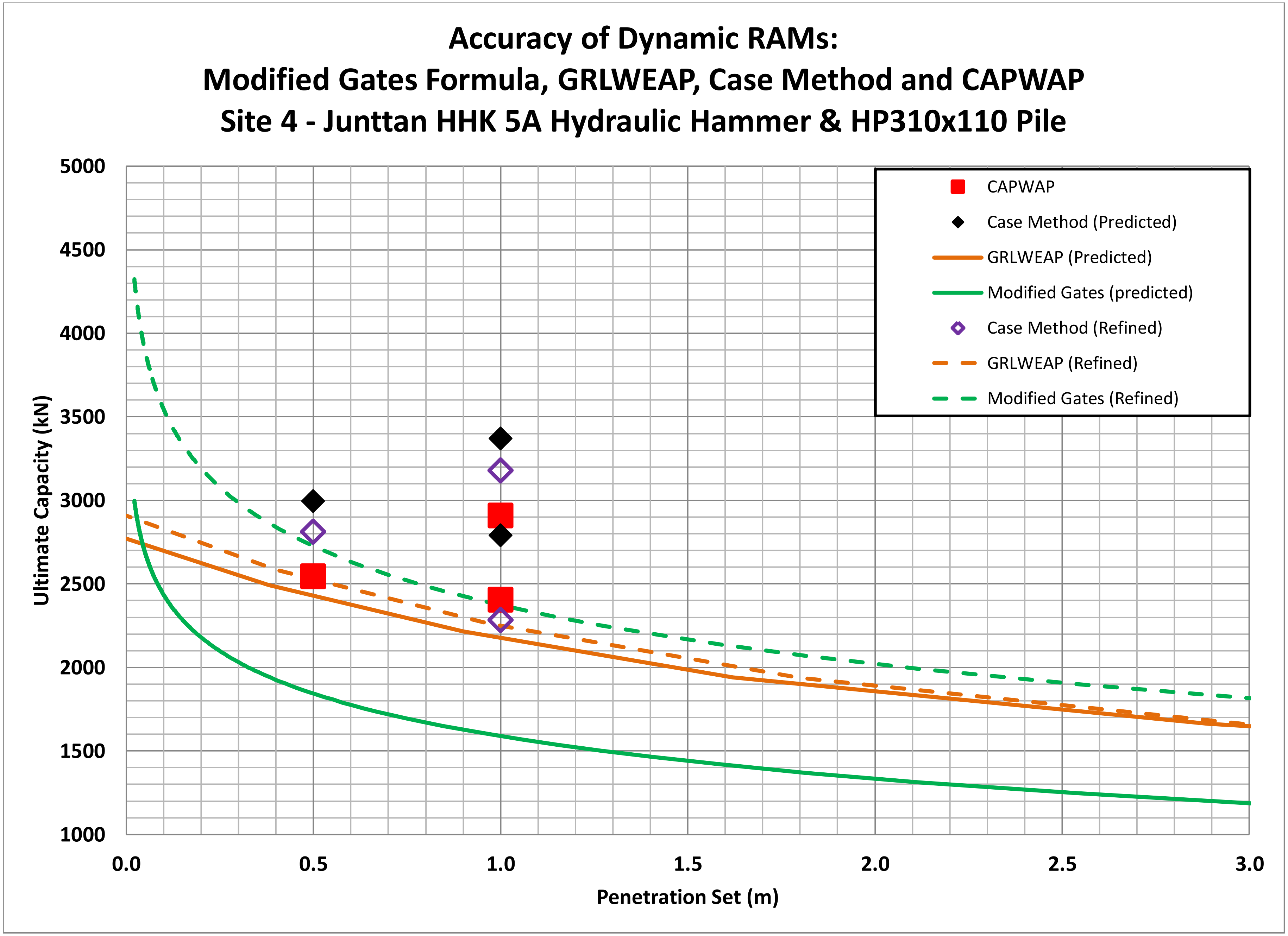
Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 7 - Junttan HHK 5A Hydraulic Hammer & HP310x110 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
Penetration Set (m)
CAPWAP (Bedrock)
CAPWAP (Glacial Till)
Case Method (Predicted - Bedrock)
Case Method (Predicted - Glacial Till)
GRLWEAP (Predicted - Bedrock)
GRLWEAP (Predicted - Glacial Till)
Modified Gates (Predicted - 6.7)
Case Method (Refined - Bedrock)
Case Method (Refined - Glacial Till)
GRLWEAP (Refined - Bedrock)
GRLWEAP (Refined - Silt Till)
Modified Gates (Refined - Bedrock)
Modified Gates (Refined - Silt Till)

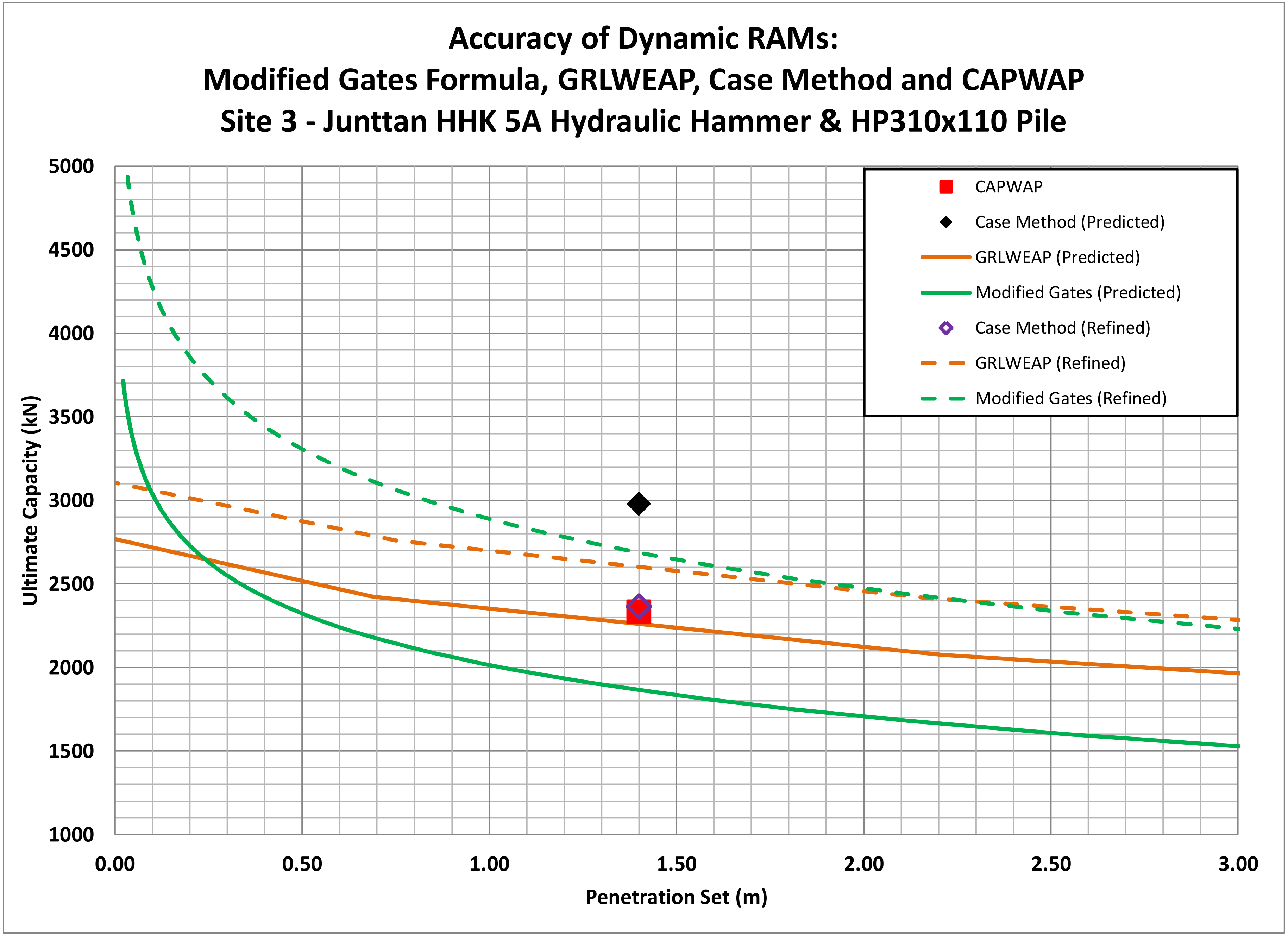


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 6 - Delmag D19-32 Diesel Hammer & HP360x132 Pile
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)

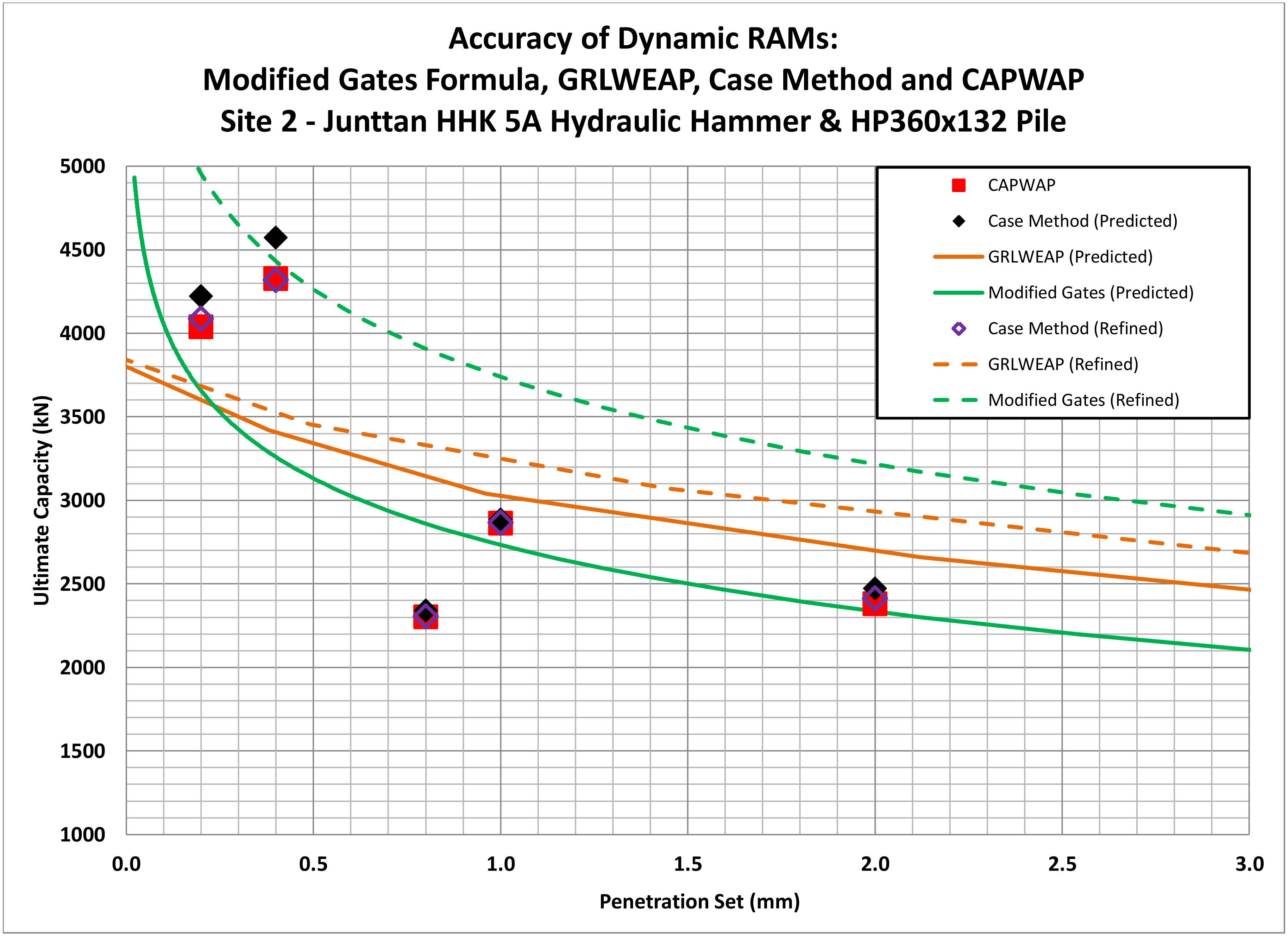


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 6 - Pileco D19-42 Diesel Hammer & HP360x132 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (refined)
GRLWEAP (Refined)
Modified Gates (Refined)

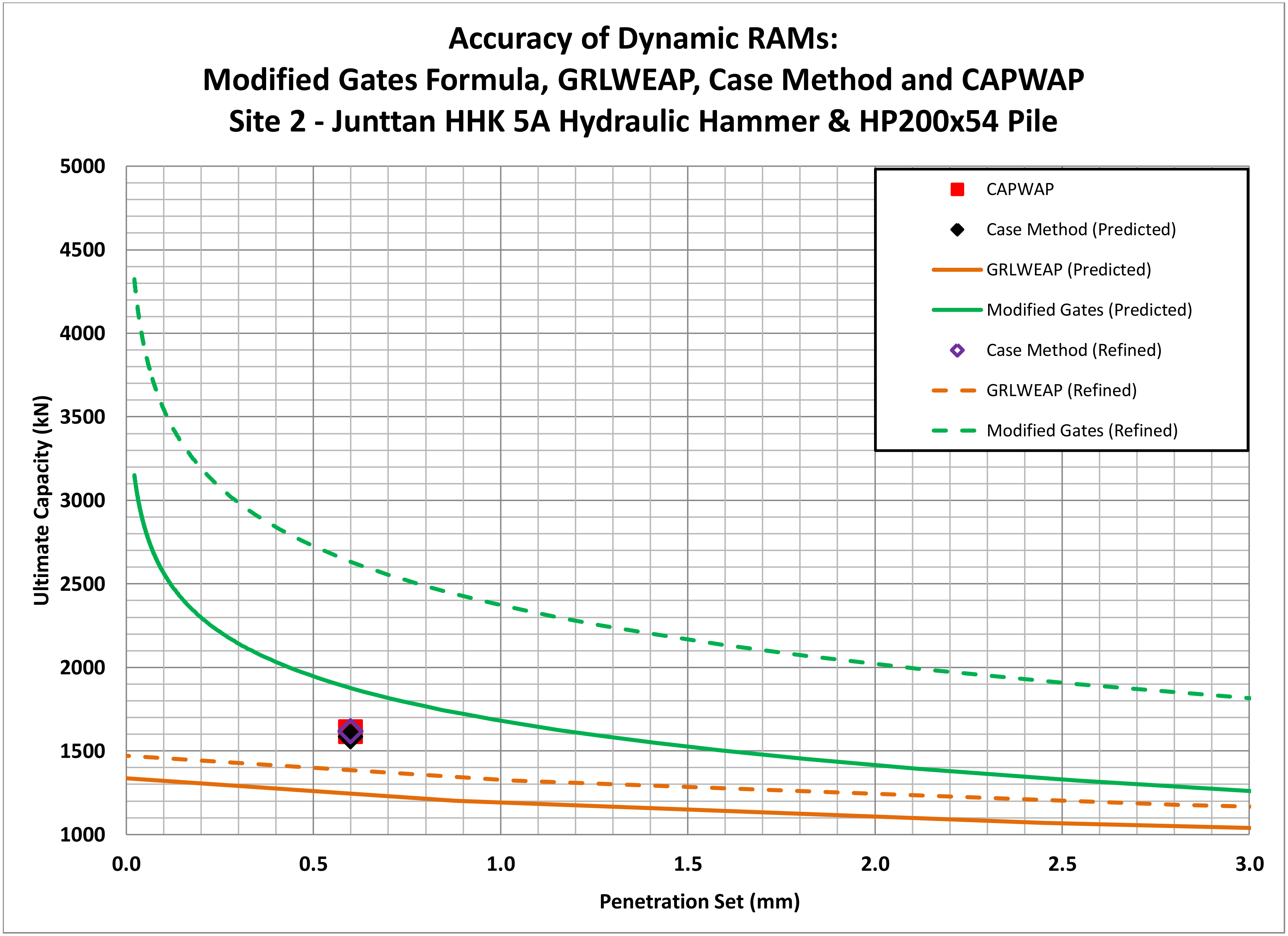

Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 4 - Junttan HHK 5A Hydraulic Hammer & HP310x110 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

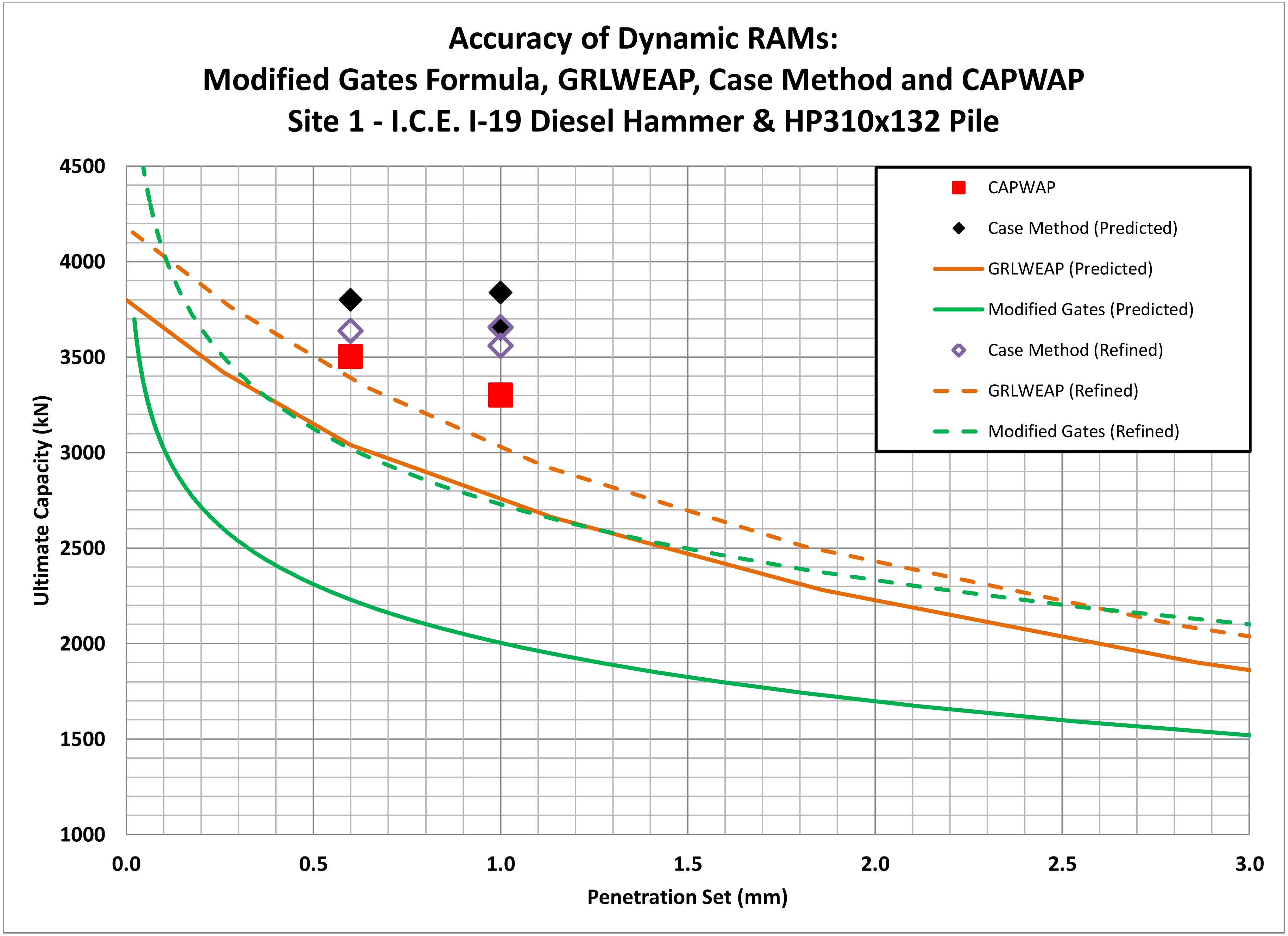


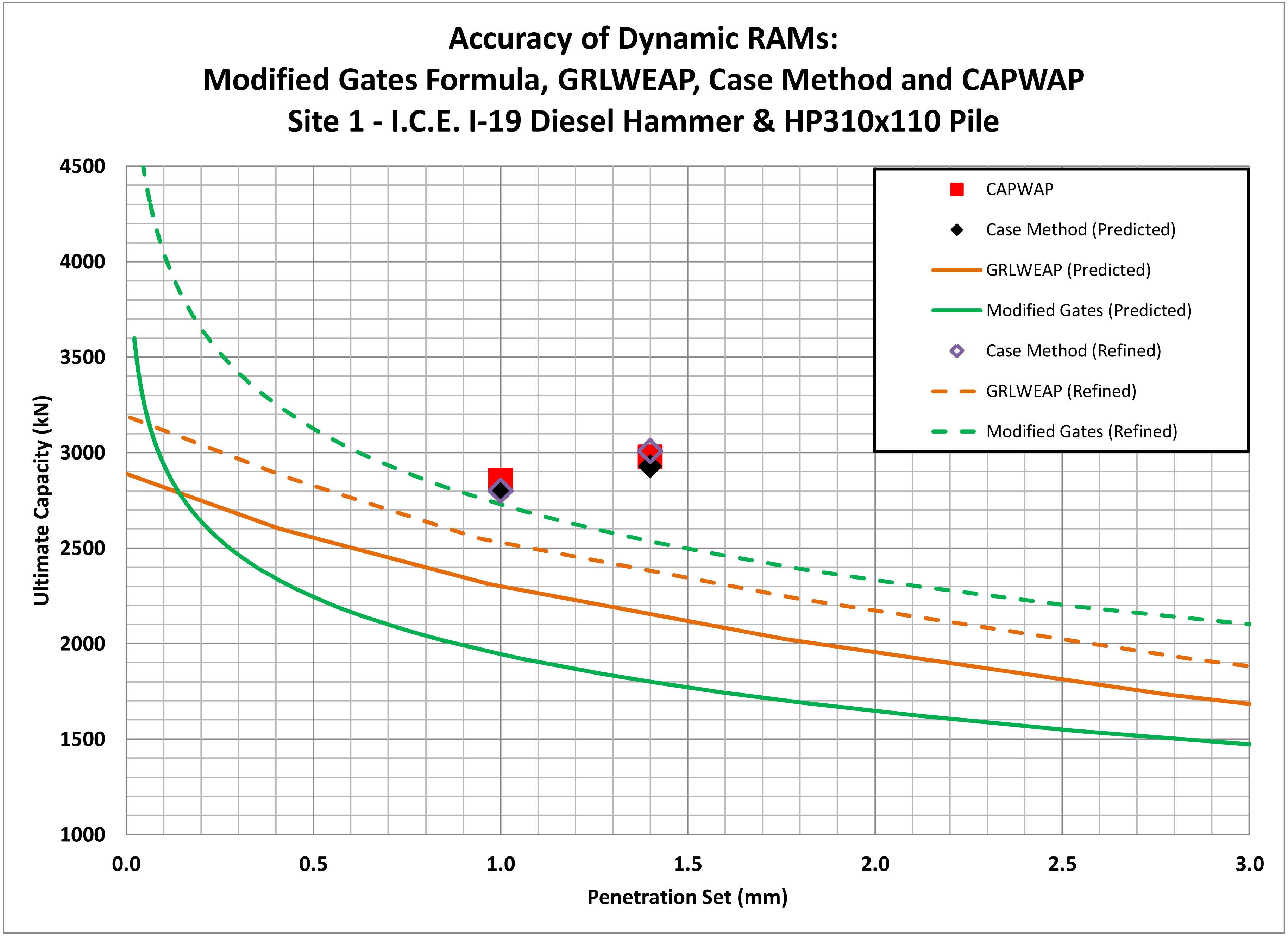
Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 3 - Junttan HHK 5A Hydraulic Hammer & HP310x110 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.00
0.50
1.00
1.50
2.00
2.50
3.00
Penetration Set (m)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)

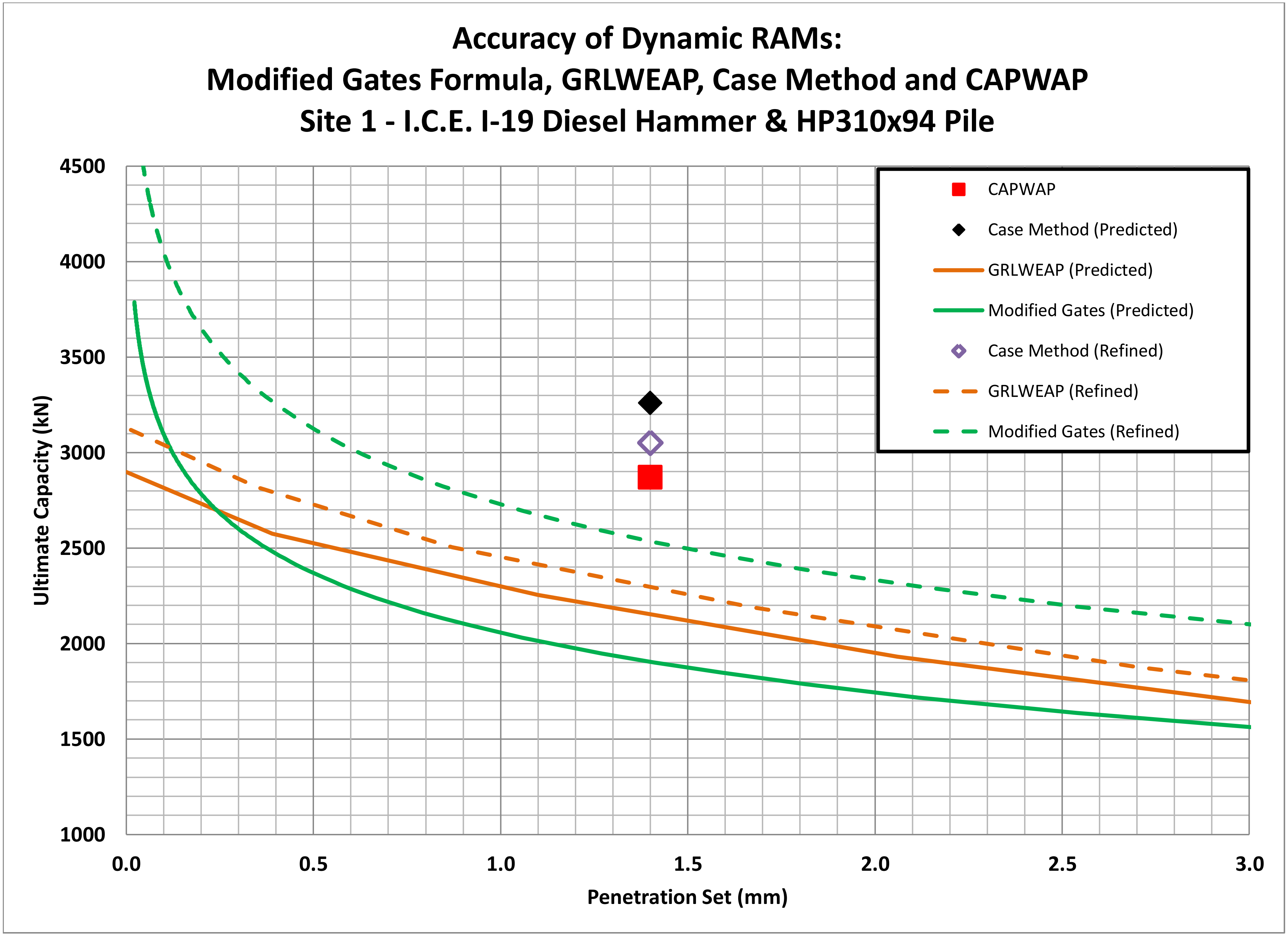


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 2 - Junttan HHK 5A Hydraulic Hammer & HP360x132 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (mm)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 2 - Junttan HHK 5A Hydraulic Hammer & HP200x54 Pile
Ultimate Capacity (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (mm)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 1 - I.C.E. I-19 Diesel Hammer & HP310x132 Pile
Ultimate Capacity (kN)
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (mm)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 1 - I.C.E. I-19 Diesel Hammer & HP310x110 Pile
Ultimate Capacity (kN)
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (mm)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)


Accuracy of Dynamic RAMs:
Modified Gates Formula, GRLWEAP, Case Method and CAPWAP
Site 1 - I.C.E. I-19 Diesel Hammer & HP310x94 Pile
Ultimate Capacity (kN)
4500
4000
3500
3000
2500
2000
1500
1000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Penetration Set (mm)
CAPWAP
Case Method (Predicted)
GRLWEAP (Predicted)
Modified Gates (Predicted)
Case Method (Refined)
GRLWEAP (Refined)
Modified Gates (Refined)