Estimation of Pile Capacity by Optimizing Dynamic Pile Driving Formulae

by

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AUTHOR’S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners. I understand that my thesis may be made electronically available to the public.

ABSTRACT

Piles have been used since prehistoric times in areas with weak subsurface conditions either to reinforce existing ground, create new ground for habitation or trade, and support bridges and buildings. Originally piles were composed of timber and driven to depths of approximately 3 metres (10 feet) with drop hammers using very heavy ram weights. As technology improved so did the materials that piles are composed of; after timber, precast concrete, then cast in place or in situ concrete, then steel tube and H piles were developed and used. Similarly pile driving equipment evolved as well; after drop hammers, steam or air, diesel, hydraulic, and then vibratory hammers were developed. Alternative techniques for installing piles also exist such as excavating and boring methods; these advances in piling materials and driving hammers allow piles to be driven to greater depths and support larger loads than ever before. Currently, piling is a multibillion dollar a year industry, thus the need to develop more accurate prediction methods can potentially represent a significant savings in cost, material, and man power.

Multiple predictive methods have been developed to estimate developed pile capacity. These range from static theoretical formulae based on geotechnical investigation prior to pile driving even occurring using specific pile and hammer types to semi empirically based dynamic formulae used during actual driving operations to more recently developed computer modeling and signal matching programs which are calibrated with site condition during initial geotechnical investigations or test piling to full scale static load tests where piles are loaded to some predetermined value or failure condition.

In this thesis, dynamic formulae are used to predict pile capacity from those installed by drop and diesel hammers and are compared to the results from pile load tests, which are taken as the true measure of developed bearing capacity. The dynamic formulae examined are the Engineering News Record (ENR), Gates, Federal Highway Administration (FHWA) modified Gates, Hiley, and Ontario Ministry of Transportation (MTO) modified Hiley formulae.

The basis of the study is 77 piles driven throughout Ontario, Canada by the MTO. The predicted capacities are compared to the measured field estimated capacities and analysed by simple statistical factors such as regression analysis, coefficients of determination, standard deviations, percent differences, and correlation values. The findings are also compared to other studies using data mainly

form North America as well as a handful from around the world and it found that the variability in the estimated to field tested capacities are similar.

The second phase of research was to vary select parameters from each formula in order to improve predictive capacities, first by back calculating the specific parameter value on a pile by pile basis which result in a perfect match of predicted to measured capacity, taking the average value, and re-calculating the predictive capacities for each pile by each dynamic formula.

The third phase involved solving the inverse by using the solver function of Microsoft Excel to find a value of the parameters from each formula which results in the lowest difference between predicted and measured capacities. Along with altering the original parameters, others are added to the energy term of the ENR and Gates formulae as well as the coefficient of restitution term of the Hiley and MTO modified Hiley formulae and varied in order to improve predicated pile capacities.

The fourth and fifth phase of research re-examined the dynamic formulae from the second and third phases of analysis but included removing the safety factor to determine the resulting effect on the predicted pile bearing capacity. The piles installed are examined as a whole then subdivided by hammer type, then pile material as well as hammer type and pile material simultaneously.

In general after using the revised parameter values, all dynamic formulae predictive capacities improved despite hammer and material type, the only exception to this are timber piles which results in relatively poor predictions despite the type of driver used to advance the pile or the dynamic formula employed.

Even though the MTO modified Hiley formula had the largest amount of improvement, after the revised parameters are used, it still results in predictions with relatively large amount of variation. Contrarily, after the revised values were determined, the ENR and Gates formulae predicted values most closely match the measured pile capacities from field tests. When examining the results based on hammer types the ENR formula favoured diesel hammered piles while the Gates formula is best suited to predict capacities for drop hammered piles.

It was determined that for the ENR and Gates formula the most accurate piles predictions were obtained when the c and the $^{1}/_{7}$ coefficients were changed to 0.21 and 2.29, respectively. The most accurate predictions from the MTO modified Hiley formula occurred when the e and C coefficients were varied simultaneously to 1 and 55.66, respectively, as given by the multi regression analysis.

Thus, it is recommended that for drop hammered piles, the Gates formula with the revised coefficient is used to predicted pile capacities. Whereas for diesel hammered piles both the ENR and MTO modified Hiley formula are used and with the more conservative prediction being taken as the predicted pile bearing capacity.

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LIST OF ABBREVIATIONS

For convenience and reference, abbreviations used in this thesis are listed below:

AASHTO – American Association of State Highway and Transportation Officials
ASD – Allowable Stress Design
ASTM – American Society for Testing and Materials
CAPWAP – Case Pile Wave Analysis Program
CNBC – Canadian National Building Code
CPT – Cone Penetrometer Test
CV – Coefficient of Variation
DOT – Department of Transportation
ENR – Engineering News Record
FHWA – Federal Highway Administration
FLAC – Fast Lagrangian Analysis of Continua
FOSM – First Order Second Moment
LCPC – Laboratoire Central des Ponts at Chaussées
LRFD – Load and Resistance Factor Design
MTO – Ministry of Transportation, Ontario
MRA – Multi Regression Analysis
OCR – Over Consolidation Ratio
PCUBC – Pacific Coast Uniform Building Code
PDA – Pile Driving Analyzer
PDE – Pile Driving Equation
SPT – Standard Penetrometer Test
UDEC – Universal Distinct Element Code
UCD – Unconfined Compressive Strength
USD – Ultimate Stress Design
WEAP – Wave Equation Analysis Program
WSDOT – Washington State Department of Transportation

LIST OF SYMBOLS

For reference, common symbols and associated units in parentheses (if applicable) used throughout this thesis are given below:

A_p – Cross Sectional Pile Area (m^2 or ft^2)
α' – Skin Friction Coefficient
α – Soil Pile Adhesion Factor
b – Pile Tip or Expanded Base Width or Diameter (mm or in)
β – Earth Pressure Coefficient to Clay Friction Angle Factor
C – Measured rebound of pile (mm/blow or in/blow)
C_1, C_2, C_3 – Compression of Pile Cap and Head, Pile, and Soil (mm or in)
C_1 – Slope of Line from Pile Load Test ($1/kN \cdot \sqrt{mm}$ or $1/ton \cdot \sqrt{in}$)
C_2 – y-intercept of Line from Pile Load Test ($\sqrt{mm}$/kN or $\sqrt{in}$/tons)
C_N – SPT Correction Factor
c – Energy Loss Coefficient (inches)
c' – Effective Cohesion (kPa or lb/ft^2)
c_u – Undrained Shear Strength (kPa or lb/ft^2)
$c_{u(vst)}$ – Vane Shear Test Undrained Shear Strength (kPa or lb/ft^2)
D – Pile Diameter (m or ft)
Dr – Soil Density (g/cm^3 or lb/ft^3)
δ – Soil Pile Friction Angle (^{o})
δ – Elastic Pile Shortening (mm or in)
Δ – Pile Settlement (mm or in)
ΔL – Incremental Pile Length (m or ft)
Δ_u – y-intercept Divided by the Slope of the Brinch Hansen Pile Load Test Plot (mm or in)
E – Foundation Elastic (Young's) Modulus (kN/m^2 or ton/ft^2)
E_n – Imparted Hammer Energy ($kN \cdot m$ or $lb \cdot ft$)
E_p – Pile Material Modulus of Elasticity (kN/m^2 or lb/ft^2)
E_s – Horizontal Soil Modulus of Elasticity (kN/m^2 or lb/ft^2)
e – Coefficient of Restitution
e_f – Hammer Efficiency
f – Unit Frictional Resistance (kN/m^2 or lb/ft^2)
f_{av} – Unit Frictional Resistance over which $(\overline{N_1})_{60}$ is Constant (kN/m^2 or lb/ft^2)

f_c – Cone Frictional Resistance from CPT Data (kN/m^2 or lb/ft^2)

Φ, φ – Internal Angle of Friction (^{o})

Φ', φ' – Effective Internal Angle of Friction (^{o})

φ – tan^{-1} of Pile Diameter to Pile Spacing Ratio

φ'_R – Drained Friction Angle of Remolded Clay (^{o})

g – Gravitational Constant (9.81m/s^2 or 32.2 ft/s^2)

γ' – Effective Soil Unit Weight (kN/m^3 or lb/ft^3)

γ_T – Total Soil Unit Weight (kN/m^3 or lb/ft^3)

γ_w – Unit Weight of Water (9.81 kN/m^3 or 62.4 lb/ft^3)

h – Ram Fall Height (m or ft)

I – Foundation Moment of Inertia (kg·m^2 or lb·ft^2)

I_p – Pile moment of Inertia (kg·m^2 or lb·ft^2)

J_s – Smith Damping Factor (s/m or s/ft)

K – Effective Earth Coefficient

K_o – At Rest Earth Coefficient

k_b, k'_b – Bearing Capacity Factor

K_r – Relative Pile Stiffness (kN/m^2 or lb/ft^2)

L – Embedded Pile Length (m or ft)

M – Foundation Bending Moment (N·m or lb·ft)

m – Number of Rows in a Pile Group

N – Blow Count (blows/mm or blows/in)

n – Number of Piles within each Row in a Pile Group

n – Efficiency of Hammer Blow

N_{60}, $(N_1)_{60}$ – Average SPT Obtained Field Number

$(\overline{N_1})_{60}$ – Average Corrected SPT Values

N_c^* – Bearing Capacity Factor

N_q^* – Bearing Capacity Factor

N_φ – Rock Friction Angle Factor

P – Pile Perimeter (m or ft)

PI – Plasticity Index

p_a – Atmospheric Pressure (taken as 100 kN/m^2 or 2 000 lb/ft^2)

q – Displacement or Quake where Pile Behaves Plastically (m or ft)

q_l – Limiting Pile Point Bearing Resistance (kN/m^2 or lb/ft^2)

q' – Effective Vertical Stress at the Pile Tip (kN/m^2 or lb/ft^2)

$q_{c(eq)}$ – Equivalent Average CPT Resistance Value (kN/m^2 or lb/ft^2)

q_{c1}, q_{c2} – Average CPT Resistance Values Below and Above Pile Tip (kN/m^2 or lb/ft^2)

Q – Load Applied to Pile (kN or lb, ton, kip)

Q_p – Point Bearing Pile Capacity (kN or lb, ton, kip)

$Q_{p(ult)}$ – Ultimate Pile Point Bearing Capacity in Rock (kN or lb, ton, kip)

Q_s – Skin or Friction Bearing Pile Capacity (kN or lb, ton, kip)

Q_u – Ultimate Pile Bearing Capacity (kN or lb, ton, kip)

$\mathit{Q_u}$ – Ultimate Load Applied to Pile during Load Test (kN or lb, ton, kip)

q_p – Unit Pile Point Bearing Resistance (kN/m^2 or lb/ft^2)

q_u – Unconfined Compressive Strength of Rock (kN/m^2 or lb/ft^2)

$q_{u(design)}$ – Designed UCS of Rock (kN/m^2 or lb/ft^2)

R – Allowable Pile Bearing Capacity (kN or lb, ton, kip)

R_u – Ultimate Pile Bearing Capacity (kN or lb, ton, kip)

R_1 – Shear Strength to Atmospheric Pressure Ratio Factor for Cone Penetrometer

R_2 – Cone Penetrometer Model Factor

R^2 – Coefficient of Determination

SF – Factor of Safety

s – Final Pile Set (mm/blow or in/blow)

σ – Vertical stress (kN/m^2 or lb/ft^2)

σ’, σ_o' – Effective vertical stress (kN/m^2 or lb/ft^2)

$\overline{\sigma_o'}$ – Mean Effective vertical stress (kN/m^2 or lb/ft^2)

σ_c' – Preconsolidation Pressure (kN/m^2 or lb/ft^2)

t – Time (seconds)

u – Pile Displacement (m or ft)

v – Velocity of Ram Fall (m/s or ft/s)

W_p – Pile Weight (kg or lb, ton, kip)

W_r – Ram Weight (kg or lb, ton, kip)

x – Offset from Pile Load Test Graph (mm)

y – Pile Lateral Deflection (mm or in)

z – Pile Section Depth Below Ground Surface (m or ft)

UNIT CONVERSIONS

For reference, typical conversions for the units, both imperial and metric, commonly used throughout this thesis are given below (All conversions are approximate except those defined by convention):

1 m = 100 cm

1cm = 10 mm

1 in = 25.4 mm

1 m = 3.28 ft

1 ft = 12 in

1 lb/ft^3 = 16 kg/m^3

1 kip = 1 000 lb

1 kg force = 2.204 lb force

1 ton force = 2 000 lb force

1 ton force = 8.90 kN

1 kip force = 4.45 kN

1 lb force = 4.45 N

1 ft·lb = 1.36 N·m

1 kip·ft = 1.36 kN·m

1 kPa = 1 kN/m^2

1 kN/m^2 = 20.885 lb/ft^2

1 psi = 144 lb/ft^2

1 psi = 6.895 kN/m^2

1.0 INTRODUCTION

This thesis compares various dynamic pile driving formulae and attempt to identify which formula(e) results in the most accurate prediction of bearing capacity when compared to pile load tests. The term 'most accurate' is in reference to the prediction which gives the closest value to the actual pile bearing capacity as determined by the pile load test, has the lowest coefficient of variation (CV), and the highest coefficient of determination (R^2) values.

The basis of this thesis is from data provided by the Ontario Ministry of Transportation (MTO); both the MTO report EM-48 Rev. 1993; Pile Load and Extraction Tests 1954 – 1992 and MTO provided Microsoft Excel files. In total this database comprises 41 pile sites and contains 371 unique pile load tests. Of these, 48 are extraction tests, 24 are lateral load tests, 118 are multiple repeated (extraction, lateral and axial compression) tests, and 181 are unique axial compression pile load tests. From these 181 tests, 40 are not taken to failure, 9 are Franki displacement or cast in situ concrete piles, 2 are batter piles, and 53 are missing information required by specific dynamic pile driving formulae, leaving a subset of 77 piles for comparison purposes.

The piles are composed of timber, steel (H Pile, open and closed end tubes), and pre-cast concrete materials of varying cross sectional geometries, areas, and lengths. The information from the MTO report and Excel sheets with regards to pile materials, lengths driven, blow counts, pile driving equipment including hammer weights, fall lengths, rated hammer energy, and cap weights is used to predict the pile capacity based on site observations. Since the formulae used to predict pile capacity originated, in some cases, over 100 years ago many of them are derived in terms of imperial units thus this thesis presents both imperial and metric units throughout. Any measured or calculated values are presented the units are given and when values of pile capacity are presented metric units are always used.

This thesis is subdivided into chapters in order to present the material in a logical sequence to aide in understanding the criteria used in selecting specific formulae that may best predict pile bearing capacity, the results based on statistical analysis, and the recommendations made.

Chapter one presents background information on piling; a brief history of piling, the importance of piled foundations, pile uses, the various types of piles that exist, installation methods, and methods for determining bearing capacity.

Chapter two presents piling in depth; significant characteristics of piles, types of piles, pile classifications, load transfer mechanisms, uses of piles, limitations of particular piles, behaviour of piles in groups, and alternatives to conventional piles.

Chapter three presents information on pile driving equipment, going into detail about various pile drivers, when they are developed, and installation methods used.

Chapter four describes various methods for determining pile capacities from predictive static formulae, predictive dynamic formulae, computer models, dynamic signal matching methods, static pile load tests, and alternatives to allowable stress methods as well as discuss the limitations of each method.

Chapter five presents the MTO database in detail including the sites and pile tests used in this project, analysis of results from the dynamic formulae including statistical investigation, comparison to other similar studies in North America, and suggest modifications to existing coefficients and dynamic formulae to better match the predicted pile capacity to that of the measured pile load tests.

Chapter six presents the discussions and conclusions reached based on the above mentioned results including which predictive formula(e) has the closest correlation(s) to the measured pile capacity and which does not; as well as possible explanations for the trends observed.

Chapter seven provides recommendations to improve each specific formula examined, suggestions for data to be collected in future driving efforts which when applied to the dynamic formulae examined may result in more accurate predictions or increase confidence of the determined results, and potential future work to both improve the predictive ability of dynamic pile formulae as well as provide alternatives predictive methods which may yield more accurate results.

1.1 Background

Piles are defined as vertical or near vertical members that are installed into the earth to act as or aid in the support of large loads and are comprised of a variety of materials. They are a type of deep foundation; deep foundations are defined as any structure whose embedded length to width ratio greater than five (Terzaghi and Peck, 1960). They are used in areas where the soil layers near ground surface are deemed too weak to support the loads imposed on them by a superstructure above and used when excavation to a more competent soil layer is uneconomical. Piles have been used in foundation design since pre historic times; some records indicate timber being used as far back as 12 000 years ago in Switzerland (Sowers, 1979) with certain sites containing over 100 000 piles and driven to depths of up to three metres (Fleming et al., 1992). More recently entire cities have been founded on piles, such as those of Lake Maracaibo in Venezuela, Amsterdam in Holland and Venice in Italy as far back as 1 000 years ago on piles 15 to 20 metres deep (Fleming et al., 1992). These piles are used for many purposes such as building bridges to cross streams and rivers, creating settlements for the habitation and protection of people, and constructing areas to allow for trading by sea.

Prior to the 19th century all piles were made from felled timber, this material limited the amount of load that is safely supportable as well as the length driven and thus limits the size of the structures which are built upon them. Since the 1820s piles have been constructed from timber, concrete, steel, and a combination of these materials (Fleming et al., 1992). These new materials increase both the strength of the piles as well as the depths they are installed to thus increasing the carrying capacity of the piles which in turn increased the size of the loads or structures placed atop them.

As pile design becomes more sophisticated so does the methods used to install them. Originally, pile drivers use percussion methods to pile into the soil and are simple gravity driven machines where weights are raised by man power, pulleys, and leverage then dropped on to the heads of the piles. With the industrial revolution of the early 19th century steam powered drivers became invented, followed by diesel and eventually hydraulically powered machines in 1801, 1946, and the 1960s, respectively (Sowers, 1979; Hough, 1969). Besides impact based drivers, others are manufactured which install piles by excavation, jetting, boring, jacking, vibration, and electroosmosis.

Piles transfer the loads from structures above them by two basic means; one is when the weight of the structure is passed down to a more competent soil strata and the base of the pile supports the majority of the load, known as tip or end bearing and the other is when the weight is distributed through the soil along the length of the pile, known as friction bearing (Das, 2004). With these mechanisms known, a need to predict the actual amount of load that a pile can safely withstand arose.

Several methods have been developed to determine the bearing capacity of piles. Originally depending upon the type of soil the pile encountered and the material the pile is made from; rules of thumb arose to the amount of load that a pile could support. Next, theoretical formulae are developed based upon the transfer mechanism of the pile used to support the weight, as well as the sub surface conditions present, and the material of the pile. These calculations are usually done pre driving to determine an estimate of load that each pile can withstand. As is the nature of geotechnical engineering, it is evident that soil contains much heterogeneity which is not accounted for in laboratory testing or during theoretical calculations; thus another method to determine pile capacities became necessary to develop. Dynamic pile prediction formulae are created to account for site specific conditions as they occurred. With improvements in technology and the advent of computers came software able to predict pile capacity. Modeling programs, analogous to the theoretical formulae, use site data prior to pile installation while signal matching programs are similar to dynamic prediction formulae and use site conditions during the actual installation process. Still other methods are developed to better ascertain the actual bearing capacity values of piles, these include an ultimate working strength design method known as Load and Resistance Factor Design (LRFD) which uses stoichiometric parameters and computer software such as neural networks which uses back propagation algorithms to determine pile capacity.

The importance of using piles since prehistory is known from archaeological studies which shown that they allowed man to expand, develop, and provide protection for thousands of years. So much so that in some cultures driving piles is part of the everyday lives of its peoples. Herodotus in the 4th century BC wrote that the Peonions, a polygamous African tribe have a law stating that each time a man wishes to marry he has to first drive three piles (Fleming et al., 1992). This aids in building dwellings for a people with an ever expanding community. Piles are no less important today. In 2001, Trade and MSI projected that piling is almost a 600 billion Canadian dollar a year industry in England alone and in five years that number is expected to increase one and a half times that much to 900 billion Canadian dollars, meanwhile other types of foundations such as ground improvement and underpinning would stay approximately constant at 200 and 100 billion Canadian dollars a year, respectively (Knight, 2008). With the population of urban areas increasing in communities such as Waterloo, Ontario which are limiting urban sprawl by creating legislation restricting the extents to which cities can expand; land to build on or develop is becoming more and more scarce (Curie, 2003). The reasons for limiting urban sprawl are numerous and include environmental protection by restricting construction on greenlands, minimizing congestion on major roads, highways, and lowering costs of eventual infrastructure maintenance and replacement (Martins, 2004). With policies such as these, people are required to build heavier and taller structures not just for commercial or industrial applications but for residential housing a well. This means that in areas where shallow foundations such as mats are adequate to support small story buildings, piles may become necessary to support new construction. The need for a more accurate method to predict pile capacity then becomes obvious; even if bearing predictions improve by 10 percent; it may result in cost savings of at least 90 billion Canadian dollars a year in man hours, material, and transportation.

2.0 PILES

In the most general sense there are three types of piles; wide thin piles, wide flange piles, and cylindrical shaped piles. Wide thin and flange piles are known as sheet and soldier piles, respectively (Coduto, 2001). Sheet and soldier piles are attached together to form walls, as shown below in Figures 1 and 2, and are usually used as a lateral temporary support to hold back water, groundwater, and or soil in excavations and open water areas. Sheet piles are made from steel, reinforced concrete, wood, aluminum, fiberglass, vinyl or polyvinyl chloride whereas soldier piles are made from vertically installed steel members with horizontal wood supports between them (Coduto, 2001).

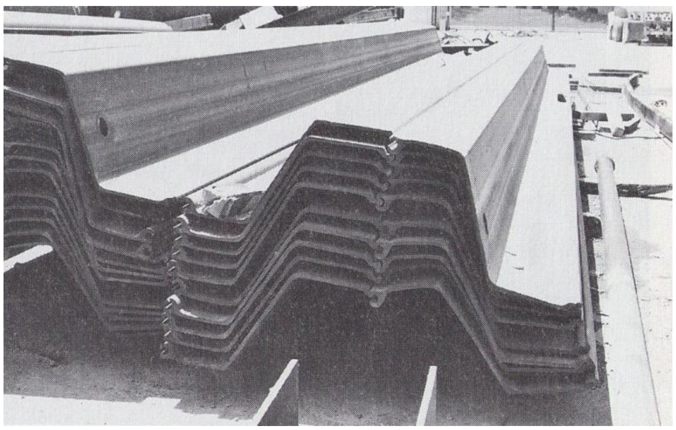


Figure 1: Sheet piles (Coduto, 2001)

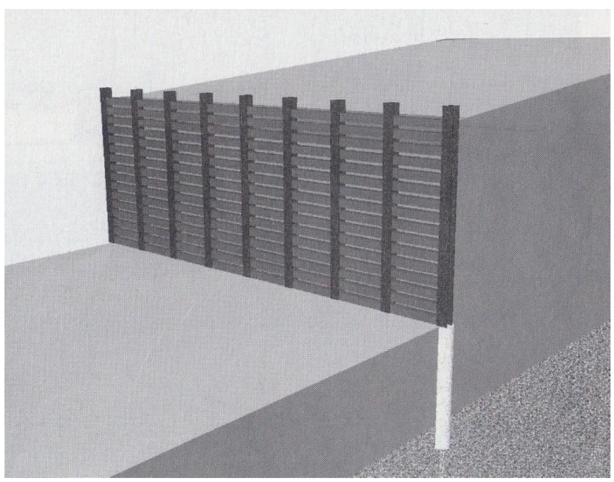


Figure 2: Soldier pile wall (Coduto, 2001)

Cylindrical piles are made from steel, concrete, timber or a combination of these materials and are circular, square, octahedral or H in cross sectional design. Cylindrical piles are used mainly for axial compression, tensile or lateral load resistance. Since the main focus of this thesis is on cylindrical piles only they are discussed further. Cylindrical piles are classified according to many different criteria which include; installation orientation, material type, installation methods, displacements produced, load transfer mechanisms, and uses.

2.1 Pile Classification

2.1.1 Installation Orientation

One of the simplest ways to classify piles is their orientation below ground surface. Two possible orientations exist; piles which are installed at an angle and piles which are installed vertically.

Inclined piles, also known as batter or raker piles are installed at horizontal to vertical ratios up to 1:4 (Coduto, 2001). Initially, designers assume that piles could resist only axial loads, thus engineers installed batter piles to provide support when horizontal loads are present so that the pile is only subjected to loads along its axis. Batter piles are traditionally used to resist lateral loads such as those induced by wind, earthquake action, downhill slope movement, open water forces, and cable/wire support for towers, etc. (Coduto, 2001). When piles are installed at an angle the foundations they support becomes very rigid. Historically these foundations do not perform well during earthquakes whose large forces are known to cause extreme curvatures in the piles, the piles themselves to buckle, and fail. This can also lead to the structure on top of the piles to become damaged. This poor behaviour is the reason that the popularity of batter piles diminished and accordingly most of the piles installed now are vertical piles. Batter piles are only presented for completeness and since pile driving formulae are developed for piles driven in vertically, only they are discussed further.

Vertical piles are used to resist loads in two main ways. They resist axial compressive or axial tensile loads through frictional and toe resistance and lateral loads through bending moments and shear forces.

Piles are used to resist axial compressive loads such as those imposed by heavy structures above ground surface, axial tensile loads such as those induced by expansive and collapsible soils or structures subjected to uplift forces such as basement foundations below the water table, offshore platforms or towers (Das, 2004). Piles are also used to resist lateral loads such as those caused by wind and earthquakes, and provide support for bridge abutments and piers near open water locations where the ground surface is susceptible to erosion by wave action.

Labelling piles as either batter or vertically orientated is straightforward and clearly understandable; however, there are many types of vertical piles that exist and grouping them together in one category is not helpful during pile capacity investigation and may in fact hinder bearing capacity analysis.

2.1.2 Installation Methods

Another technique to classify piles is according to the method in which they are installed into the subsurface. Numerous technologies exist to install piles which include percussion or hammer action from gravity, steam, diesel, or hydraulic powered pile driving rigs; vibratory methods, pile jacking, jetting, excavating, boring, and electroosmosis, etc.

The difficulty with this classification system is that as technologies evolve and new techniques for installing piles develop, specific pile installations may be classified into one or more categories which may cause confusion when trying to determine the most suitable method for piling from certain predictive methods.

Since the installation method is a result of the machinery used when piling, it is further discussed in Chapter 3: Pile Driving Equipment.

2.1.3 Soil Displacement Types

An alternative pile classification system and perhaps the most significant to pile analysis is to examine the effect that the pile produces in the soil that it is driven into. There are three basic types of piles according to how they are installed; large displacement, low displacement, and non-displacement piles. Large soil displacements occur when timber, pre-cast concrete piles, and closed end pipe or tube piles are driven into the subsurface. Research shows that the area affected ranges from two to two and a half pile diameters from the centre of driving (Das, 2004). Low displacements occur when H and open end tube piles are driven into the subsurface; accordingly the area affected is less than high displacement piles. When precast concrete piles are excavated, screwed or bored into place or concrete piles are formed in situ, they are referred to as non-displacement piles.

High displacement piles cause the soil around the pile circumference to displace downwards and move laterally away from the pile while the soil further away from the pile is thrust upwards (Coduto, 2001). In cohesionless soils, very loose to compact sands, and gravels this action causes an increase in the density and internal friction angle of the soil nearest to the pile which increases the bearing capacity of the soil. In dense to very dense sands and gravels the act of driving results in the opposite effect and causes dilation and temporary negative pore pressure generation. This causes an increase in apparent driving resistance and implies larger than actual bearing capacities; once the pore pressure dissipates the piles load bearing resistance lowers to its natural state as governed by the soil – pile system (Sowers, 1979). An additional effect of driving is the development of excess pore pressure; in sands and gravels the high hydraulic conductivity causes the excess pressure to dissipate quickly in hours to days however, in cohesive soils the excess pore pressure of the soil can take days to weeks to dissipate. The act of driving in cohesive soils cause the soil to undergo successive shear failures which results in a loss of shear strength and an increase excess pore water pressure. After the excess pore pressure dissipates the soil regains its original shear strength in a phenomenon known as set up (Coduto, 2001).

Low displacement piles cause similar reactions within the subsurface as large displacements piles but to a smaller degree. H and open end tube piles are classified as low displacement piles because only the soil below and directly adjacent to the flange and web of H piles and the walls of tube piles is compacted; these sections are generally at most 25 millimeters or 1 inch in thickness and often only half that much (Das, 2004).

Non displacement piles, such as predrilled, augured, excavated or bored cast in place concrete piles cause the opposite reaction in sub surface soil. Rather than increasing the density or internal angle of friction of the soil, the soil surrounding the walls of the boring become looser and the internal angle of friction decreases as well as the angle of the soil – pile interface (Sowers, 1979). Non displacement piles are constructed by drilling out soil and either pouring in concrete with or without a reinforced steel cage or hammering in a precast concrete pile or steel tube. To ensure adequate frictional support is formed, the pre formed pile installed is slightly larger in diameter than the hole formed, this causes a small amount of displacement but less than when low displacement piles are driven.

Additional selected methods are presented below for completeness only. Since the piles examined from the MTO records are only high and low displacement piles these other methods are only summarized.

This list is in no way exhaustive and other methods which exist are not mentioned here.

Other categories of displacement piles include hybrid installation methods (a combination of high and non-displacement or low and non-displacement piles) such as screwing, jetting, spudding; pressure injected footings such as Franki displacement piles, pile installed by vibratory methods, and electroosmosis. Since these classifications are determined by the driving rig employed they are further discussed in Chapter 3: Pile Driving Equipment.

The importance of displacement categories is that they are the used in static pile driving predictive formulae and are discussed further in Chapter 4: Pile Bearing Capacities.

2.1.4 Pile Composition

The material the pile is composed of is presented as another method to classify piles. The majority of piles are constructed from timber, concrete, steel or a combination of these materials such as timber–concrete piles, steel-concrete piles and are discussed below. In this study piles examined are made from timber, steel, concrete, and a combination of steel and concrete however others are described for completeness.

In Canada timber piles are most commonly made from Spruce, Douglas Fir, Western Red Cedar, Jack Pine, and Red Pine. Within Ontario the most common types of timber piles are those made from Jack Pine and Red Pine (CITC, 1962). The maximum compressional strength of timber piles depends upon the species of tree used; in general Douglas Fir is the strongest and Cedar is the weakest. Strengths range from 36 680 to 49 850 kN/m^2 (5 320 to 7 230 psi) and have BCLMA and CSA standard allowable loading stresses of 5 170 to 11 380 kN/m^2 (750 to 1 650 psi) (CITC, 1962). Timber piles are used since they are relatively inexpensive, readily available, easy to drive, supports small to medium loads, and can last for 100s to 1000s of years if properly installed and protected as shown by a the ones in Venice which were reused in 1902 to support a new tower after the original structure, built in 900 AD, fell (Sowers, 1979). Timber pile properties are given in Table 1 below.

Modern concrete, as it is now, was developed in 1824 by Joseph Aspdin and used as piling material by A. A. Raymond since 1897 (Fleming et al., 1992). Two main types of concrete piles exist; precast concrete and in situ cast in place piles. Cast in place piles are further subdivided into two categories; cased and uncased. Precast concrete piles are solid or hollow, whole or comprised from pieces jointed together.

Table 1: Timber Pile Properties
(compiled Sowers, 1979; Fleming et al., 1992; Das, 2004; Coduto, 2001;
Terzaghi and Peck, 1960; Chellis, 1961; CITC, 1962; and Hunt, 1974)

Wood Species	Oak, Mixed Hardwoods	Southern Pine	Spruce, Douglas Fir, Jack Pine
Length m (ft)	18 (60)	24 (80)	30 – 40 (100 – 130)
Toe Diameter mm (in)	Up to 475 (18)	Up to 500 (20)	150 – 500 (6 – 20)
Tip Diameter mm (in)	125 – 250 (5-10)		
Working loads kN (tons)	150 – 300, maximum 500 (15 – 30, maximum 50)		

Originally precast concrete piles are reinforced with post tensioned rebar; however, now almost all concrete is pre stressed (Coduto, 2001), manufactured with cables with a typical strength of 900 – 1 300 MN/m^2 (130 000 – 190 000 psi) to a maximum of 1 800 MN/m^2 (260 000 psi), whereas the concrete itself ranges in strength from 34.5 to 41.4 MN/m^2 (5 000 to 6 000 psi) depending on how much cable is used in the concrete (Das, 2004). Concrete is used as a piling material because it is less expensive than steel piles, has high compressional strength, can withstand high driving forces, support large loads, and is easily combined with an overlying concrete pad or superstructure. See Table 2 below for precast pile properties.

Table 2: Precast Pile Properties
(compiled from Sowers, 1979; Fleming et al., 1992; Das, 2004; Coduto,
2001; Terzaghi and Peck, 1960; Chellis, 1961; CITC, 1962; and Hunt, 1974)

Concrete Type	Precast Piles		
	Solid	Hollow	Jointed
Length m (ft)	15 – 45 (50 – 150)	60 (200)	30 (100)
Sectional length m (ft)	–	–	1 – 13 (3 – 43)
Wall/shell thickness mm (in)	–	100 – 150 (4 – 6)	–
Diameter or Width mm (in)	200 – 760 (8 – 30)	900 – 1 500 (35 – 60)	250 – 400 (10 – 16)
Working loads kN (tons)	150 – 8 500 (15 – 850)	2 000 (200)	700 – 2 500 (70 – 250)

Cast in place piles are either poured directly into open excavations/borings or within a shell system. Because of this cast in place piles are often considered as composite piles, either on the basis of materials (concrete and steel) or piling methods (percussion hammers forming a hole then pouring concrete into it to form the pile) used. They have the advantage of the hole being inspected prior to concreting, are easy to extend to any length required or formed to any diameter required, depending upon pile driving equipment used. Many patented types are in use, a few are presented below in Table 3 for completeness.

Table 3: Cast in Place Pile Properties
(compiled from Sowers, 1979; Fleming et al., 1992; Das, 2004; Coduto, 2001; Terzaghi and Peck, 1960; Chellis, 1961; CITC, 1962; and Hunt, 1974)

Concrete Type	Cast in Place							
	Raymond Standard	Raymond Step Taper	Franki PIF	Auger	Cobi Hercules	Bored		Wests Shell
						Uncased	Cased	
Length m (ft)	10–50 (30–165)	29 (96)	30 (100)	18–60 (60–200)	30 (100)	15–40 (50–130)	30 (100)	36 (120)
Sectional length m (ft)	–	2.4 (8)	–	–	–	–	–	0.914 – 1.220 (3 – 4)
Wall/Shell thickness mm (in)	–	–	–	–	200–500 (8–20)	–	254–610 (10–24)	92 – 127 (35/8 – 5)
Diameter Width mm (in)	400–600 (16–24)	220–350 ($8\frac{5}{8}$–$13\frac{3}{8}$)	560–610 (22–24) Base 1 (3)	300–3 000 (12–120)	–	254–750 (10–30)	–	276–610 ($10\frac{7}{8}$–24)
Working loads kN (tons)	300–500 (30–50)	400–750 (40–75)	1000–10 000 (100 – 1000)	1000–20 000 (100–2 000)	300–600 (30–60)	150–2 600 (15–260)	150–1 500 (15–150)	45–1 200 (45–120)

Steel piles came into use in the early 1900s with the advent of H rolled steel and tube piles. H piles quickly became common place replacing I beams from the 1890s and cast iron used since the late 1830s (Chellis, 1961). Steel piles became popular since they are light weight, can withstand high driving stresses, penetrate hard layers, support heavy loads, are easy to handle, cut, and splice. Lengths of piles are controlled by transportation and equipment limitations. Single piles can be driven as deep as 39 metres (127 feet) below ground surface (Chellis, 1961) and spliced together to lengths as much as 210 metres (700 feet) long (Coduto, 2001). Properties of H and tube piles are given below in table 4 including typical lengths driven, sectional lengths, wall thicknesses, diameters, and working loads.

Table 4: Steel Pile Properties
(compiled from Sowers, 1979; Fleming et al., 1992; Das, 2004; Coduto, 2001; Terzaghi and Peck, 1960; Chellis, 1961; CITC, 1962; and Hunt, 1974)

Steel	H Pile	Tube Pile
Length m (ft)	15 – 60, max 92 (50 – 200, max 304)	30 – 120 (100 – 400)
Sectional length cm x kg/m (in x lb/ft)	20 – 36 x 54 - 174 (8 – 14 x 36 – 117)	–
Wall/Shell thickness mm (in)	–	2.78 – 38, max 76 ($\frac{7}{64}$ – 1.5, max 3)
Diameter or Width mm (in)	–	150 – 1 000, max 3 000 (6 – 39, max 10)
Working loads kN (tons)	300 – 1 800 (30 – 180)	450 – 7 000 (45 – 700)

2.2 Load Transfer Mechanisms

As previously stated, piles primarily resist two types of loads; lateral and axial.

Similar to batter piles, lateral loads are only mentioned for completeness and after this section the thesis focuses only on axial loads; their mechanisms of load transfer, calculating, and predicting the amount of axial loads that a pile can support. Laterally loaded piles are sub divided into two categories; long and short. Long piles behave rigidly and short piles behave elastically (Das, 2004).

Long or rigid piles are ones whose toe or end is fixed against movement or rotation and short or elastic piles are ones where rotation is allowed to occur. Because of this, lateral loads acting on short piles cause the soil to fail in shear before the pile, whereas on long piles, the pile fails due to bending before the soil, as depicted in Figure 3.

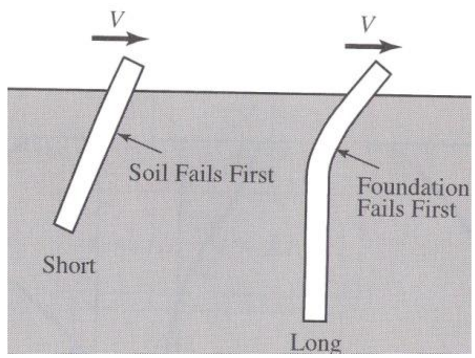


Figure 3: Lateral loads affecting short piles versus long piles (Coduto, 2001)

The classification of a "long" or "short" pile is dependent upon the stiffness of the soil and the pile itself. In general for timber piles, long piles are one where the embedded depth to width ratio is greater than 20 and for concrete or steel, the ratio is greater than 35 (Coduto, 2001). Alternatively, in 1995 Meyerhof defined a long or rigid pile as one whose relative stiffness (K_r) is greater than 0.01 as defined by Equation 1 below (Das, 2004).

$$K_r = \frac{E_p I_p}{E_S L^4} \tag{1}$$

where: E_p is the modulus of elasticity of the pile material (kN/m^2 or lb/ft^2); I_p is the moment of inertia of the pile (kN·m^2 or lb·ft^2); E_s is the horizontal soil modulus of elasticity (kN/m^2 or lb/ft^2); and L is the length of the pile (metres or feet).

When lateral loads are placed on a pile, the pile resists it by utilizing the passive pressure in the soil surrounding it (Das, 2004). Long and short piles resist lateral loads by inducing shear and moments as seen in Figures 4 and 5, respectively.

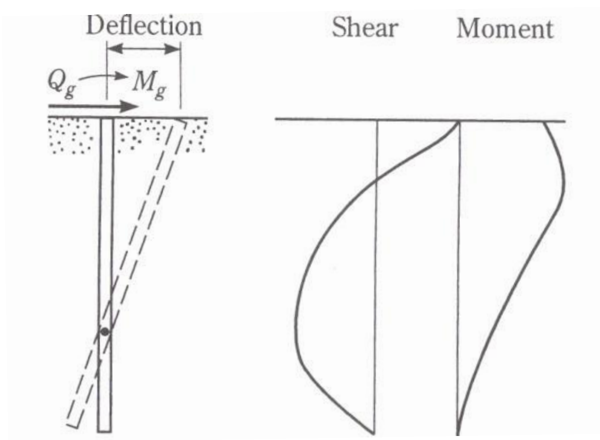


Figure 4: Reaction of a rigid pile to lateral loading (Das, 2004)

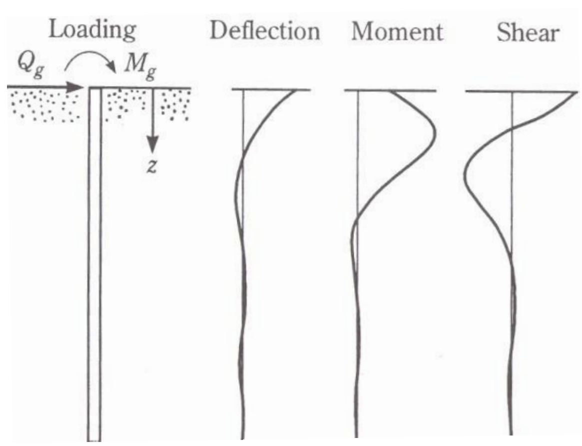


Figure 5: Reaction of an elastic pile to lateral loading (Das, 2004)

The methods used to determine the amounts of lateral load that a pile can support are numerous and range from the simple to the very complex for rigid and elastic piles. For long piles, they include straightforward equations from structural mechanics such as defining the bending moment (M) in the foundation as:

$$M = EI\frac{d^2y}{dz^2} \tag{2}$$

where: M is the bending moment in the foundation; E is the modulus of elasticity of the foundation; I is the moment of inertia of the foundation; y is the lateral deflection, in units of length; and z is the depth below ground surface being examined, in units of length.

The shear force experienced by the foundation and lateral soil resistance per unit length of the foundation is determined by differentiating Equation 2 with respect to depth once and twice, respectively (Das, 2004).

For elastic foundations analysis becomes more complicated but is computed using finite element methods and finite difference models found in commercial software such as COM624, Florida Pier, and LPILE which uses p-y methods where the pile - soil system is modeled as a series of fixed non-linear springs attached to a series of nodes as seen in Figure 6 (Das, 2004).

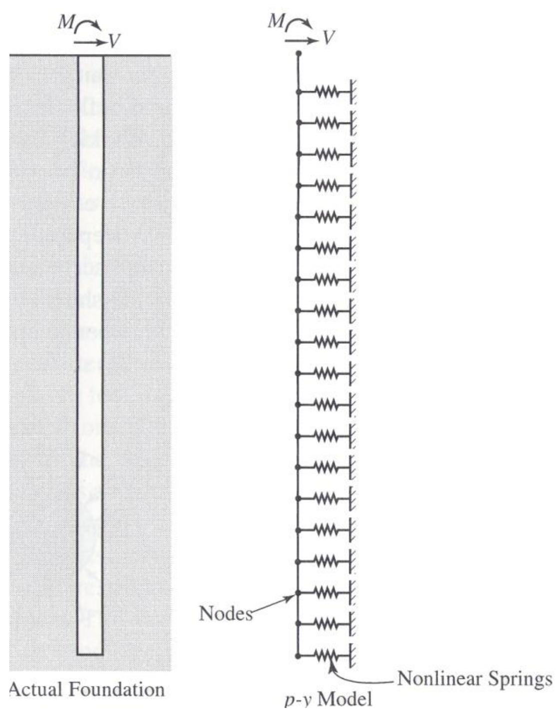


Figure 6: p-y model used in analytical analysis of lateral loads (Coduto, 2001)

Models both full scale and bench scale such as ASTM D3966: Standard Test Methods for Deep Foundations Under Lateral Load, lateral statnamic tests, and centrifuge tests are used to determine lateral load capacities of single piles in the field and laboratory.

Axial loads are further sub divided into two categories; axial compression and axial tensile loads.

Axial compressive or downward loads are transferred to the pile via skin frictional and end bearing resistance. Axial tensile or upwards/uplift loads are resisted by the pile via skin friction only, see Figure 7 below.

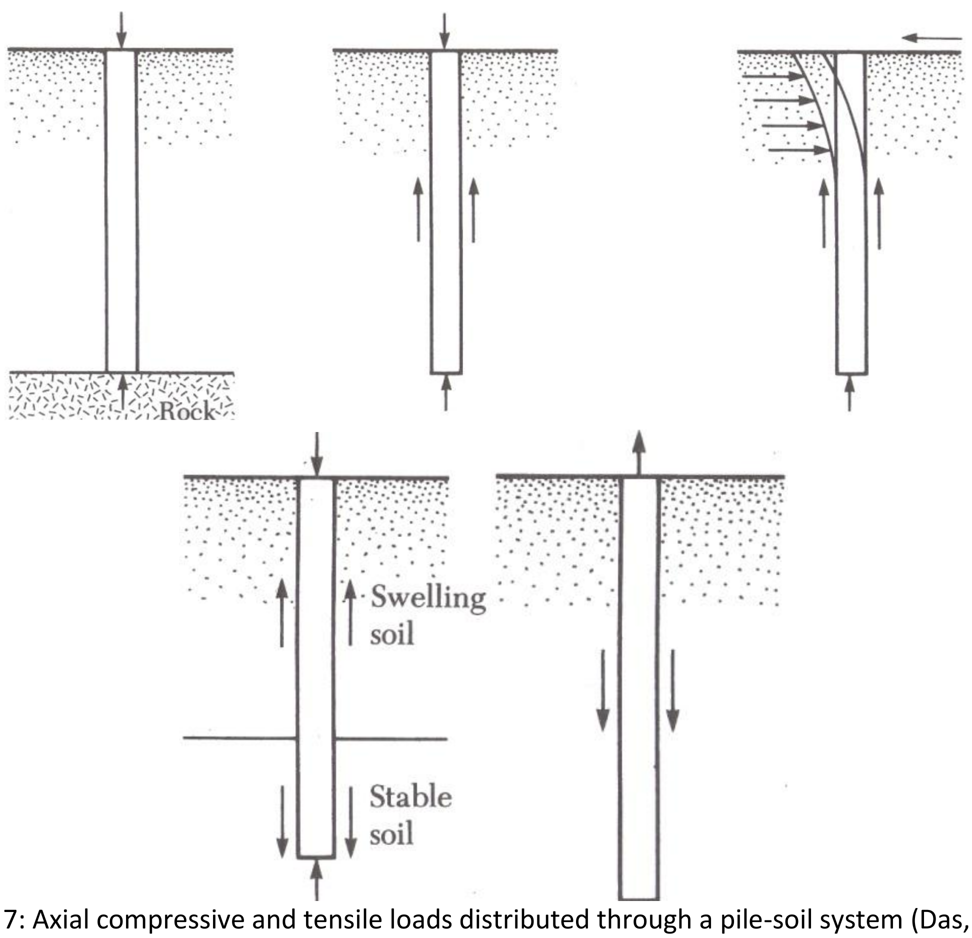


Figure 7: Axial compressive and tensile loads distributed through a pile-soil system (Das, 2004)

Friction bearing piles develop carrying capacity as they are installed into the subsurface. For driven piles the resistance is increased layer by layer for each incremental length driven, commonly a few centimetres or inches at a time. In cohesionless soil this is accomplished by densification of the surficial area around the pile shaft. Research shows that this area extends in width from one to two and a half pile diameters from the centre of the pile being driven (Sowers, 1979, Das, 2004). Meyerhof (1961) shows that the internal friction angle of soil in the affected zone typically increases by 4 and 6 degrees for driven piles in loose and compact sand, respectively. For the frictional capacity of the pile to become fully mobilized, the pile must settle on average 0.5 to 2 percent of the pile diameter (Fleming et al., 1992) or 5 to 10 millimetres (0.2 to 0.4 inches) (Coduto, 2001). For example, a 300 millimetre or 12 inch wide pile would require a settlement of 1.5 to 6 millimetres (0.06 to 0.24 inches) to fully mobilize the available frictional resistance.

In cohesive soils, the zone affected due to pile driving may be as large as nine pile diameters away from the centre of the pile for large displacement piles. This remoulded zone initially experiences a loss of shear strength but regains full strength within a few days to weeks (Coduto, 2001) as the excess pore water pressure, caused by the driving action, dissipates.

End bearing piles develop carrying capacity as a reaction to being driven against solid material such as a dense layer of sand or gravel or bedrock, which for all intended purposes has an infinite bearing capacity. Working bearing capacities of rock layers common in Southern Ontario are given below.

Table 5: Typical Unconfined Compressive Strengths of Rocks (after Das, 2004)

Rock Type	q_u (kN/m^2)	q_u (lb/ft^2)
Sandstone	70 000 – 140 000	1 500 000 – 2 900 000
Limestone	105 000 – 210 000	2 200 000 – 4 400 000
Shale	35 000 – 70 000	730 000 – 1 500 000
Granite	140 000 – 210 000	2 900 000 – 4 400 000
Marble	60 000 – 70 000	1 300 000 – 1 500 000

Comparing Tables 1 through 4 with Table 5 it is seen that the rated pile capacities and pile strengths are far below the amount of load that bedrock can safely support.

Piles under normal conditions are not usually loaded to any amount near their ultimate strength; 36 830 to 50 050 kN/m^2 (5 320 to 7 230 psi) for timber, 34 500 to 41 400 kN/m^2 (5 000 to 6 000 psi) for pre stressed concrete or the 4 000 kN (400 tons) yield point for steel H piles (Sowers, 1979). The load required to cause an acceptable amount of settlement on most piles is nowhere near the maximum strengths listed. When piles are driven to bedrock, if the stresses imparted on them by driving does exceed their compressive/tensile strength it can cause the piles to crack, split or buckle.

To fully mobilize the end bearing resistance, the pile tip must undergo a settlement of 10 to 25 percent of the pile width (Das, 2004). For example, a 300 millimetre or 12 inch wide pile requires a settlement of 30 to 75 millimetres or 1.2 to 3 inches. Since most building codes such as that of Canada only allow settlement of up to 25.4 millimetres or 1 inch (CITC, 1962); this means that most piles do not reach their full end bearing potential capacity, thus frictional resistance supports the majority of the applied load.

Sowers (1979) estimates that for long piles, 66 to 75 percent of the load supported is due to frictional resistance and only 25 to 33 percent of the load supported is due to end bearing capacity. For short piles, the end bearing capacity increases proportionally as the ratio of pile width to embedded pile length increases. The need to properly estimate the proportion of load supported via frictional resistance and end bearing becomes significant; this is discussed further in Chapter 4: Pile Bearing Capacities.

The National Building Code of Canada gives guidelines for the allowable end bearing capacity for various soils, see Table 6 below.

Table 6: End Bearing Capacity of Various Soils (after Hunt, 1974 and Chellis, 1961)

Soil (National Building Code, 1967)	kN/m^2	lb/ft^2
Soft clay, medium	100 – 140	2 100 – 3 000
Medium stiff clay	240 – 260	5 000 – 5 400
Stiff clay	310	6 500
Silt	80	1 600
Loamy sand, dense, well compacted	180	3 800
Sand, fine loose	140 – 190	3 000 – 4 000
Sand, coarse loose; compact fine sand; and loose sand-gravel mixture	290 – 310	6 000 – 6 500
Gravel, loose; and compact coarse sand	380 – 420	8 000 – 8 700
Sand-gravel mixture, compact	470 – 570	9 800 – 12 000
Hardpan and exceptionally compacted or partially cemented gravels or sands	960	20 000
Sedimentary rocks, such as hard shales, sandstones, silt stones in sound condition	1 440	30 000
Limestone	500 – 2 510	10 500 – 52 500
Foliated rock, such as schist or slate in sound condition	3830	80 000
Granite	1 000 – 10 000	21 000 – 210 000
Massive bed rock – such as granite, diorite, gneiss, trap rock – in sound condition	9 580	200 000

Frictional resistance of various soils and rock types is given by Terzaghi and Peck and summarized in Table 7.

Table 7: Developed Pile Skin Friction (after Terzaghi and Peck, 1960)

Soil Type	kN/m^2	lb/ft^2
Soft clay and silt	10 – 30	200 – 600
Sandy silt	20 – 50	400 – 1 000
Stiff clay	40 – 100	800 – 2 000
Loose Sands / Long Piles	25	500
Very Dense Sands / Short Piles	100	2 000

2.3 Pile Limitations

Each type of pile has its own advantages, disadvantages, and each is used in accordance to specific circumstances which best exploits their unique strengths. The merits of various piles are already discussed in section 2.1.4. As previously stated piles are classified according to installation orientation, displacements produced, and material composition.

Piles are either installed vertically or at an angle up to 20 degrees from the vertical, known as batter piles. Batter piles, originally designed to support lateral loads are shown to perform poorly during earthquakes strong repeated horizontal forces (Harn, 2004), whereas long vertical piles are quite effective in minimizing soil movement and settlement.

As piles are installed the displacements developed may be high, low or none. Each type is suitable for specific applications. High displacement piles are used where large load support is required and mobilizing as much frictional resistance as well as end bearing support as possible is required.

Low displacement piles are used when driving a large number of piles as close to each other as possible is necessary or where soil heaving is kept to a minimum such as near pre-existing structures or underpinning.

Non displacement piles are used when very large loads require support and or where soils would require very high blow counts to penetrate hard layers, such as very dense clays. However non-displacement piles are most often formed by boring or excavating which causes the soil around the circumference of the hole to relax, which reduces the soils internal angle of friction and if it weakens too much, soil may collapse into the hole.

Timber piles are limited, by species, to the depths they are driven, usually 20 to 30 metres (65 to 100 feet) as well as safe loads supported, usually limited to 500 kN (50 tons). Timber piles are also vulnerable to biological decay such as from bacteria, fungi, land, and marine insects. Water both beneath and at surface can damage timber piles, especially in areas which undergoes wet dry cycles such as swash zones in open water areas and the vertical distance in which the groundwater table rises and drops to with seasonal changes or pumping activities. Timber piles which are constantly dry or constantly wet last well with time. Temperatures also affect pile behaviour, if constant the pile integrity is more likely to remain intact over its lifespan however, freeze thaw cycles may deteriorate piles prematurely.

During driving, timber piles may become damaged by the hammer used, if its weight is too heavy, causing brooming of the pile head and cracks within the shaft (Fleming et al., 1992). Splicing timber piles is difficult, costly, and historically performs poorly while undergoing tensile and lateral loads (Das, 2004).

Concrete piles are driven as precast members or formed in situ. They handle large loads and are cheaper than steel; however, precast piles are heavy, more likely damaged during handling and transport, cannot endure hard driving forces as well as steel piles, and are difficult to cut and splice thus knowing the exact pile length needed beforehand is critical but also unrealistic. Cast in place piles are formed to any depth and width required, subject to equipment limitations; however, during pouring in open excavations a risk of soil collapse and squeezing of concrete is possible, especially in wet cohesionless soils. If the concrete is poured too rapidly, voids may be created causing the pile to become weaker than its assumed strength or carrying capacity. The process of drilling or boring almost guarantees a relaxation of the soil and a reduction in the soils internal angle of friction. This implies that cast in place piles rely primarily on end bearing support and minimally on frictional resistance.

Concrete piles are also susceptible to water action and chemicals within the water. Horizontal stresses such as waves in sea water act abrasively on the pile surface causing physical degradation. If the water contains high levels of sulphates (200 to 400 ppm) it can produce cracking within the pile (Fleming et al., 1992). Acidic groundwater caused by organic and inorganic acids, alkalis, plant and animal fats, and other organic matter can also damage concrete piles by causing rusting of the metal reinforcement.

Steel piles are used since they can withstand hard driving, support heavy loads, are easy to transport, handle, cut, and splice. H piles contain small cross sectional areas which allow them to penetrate hard layers; however this also allows the pile to be easily deflected or twisted when encountering boulders or other major obstructions.

Tube piles are larger, can support heavier loads, and are more rigid than H piles. This means that tube piles are less likely to bend or deflect. They have the advantage in that they are inspected after installation but prior to concreting to check for defects and damage; however, open end tube piles are routinely cleaned or the end may become plugged and act as a high displacement closed end pile rather than a low displacement pile. When driving, tube piles are hit on centre with the pile hammer or there is a risk that the pile undergoes accordion type damage. Only steel tubes specifically constructed for pile driving are used, if the carbon content is too low or high there is a risk that the pile may split or yield (Sowers, 1979).

Steel piles are more expensive than timber or concrete piles and also produce more noise during driving. Steel piles are vulnerable to corrosion by air, water, and soil if driven in areas with acidic soil, certain organic matter, or high level of salts (Sowers, 1979). In air, corrosion rates are measured at 2.5 to 10 and as high as 20 millimetres per 100 years in industrial areas (0.1 to 0.4 and 4 inches per 100 years). Rates of 5.0 to 25 millimetres (0.2 to 1 inch) in water environments, just below the water line and in marine environments corrosion rates are as much as 34 and 60 millimetres (1.36 and 2.4 inches) occur, respectively, per 100 years. Soil, although less damaging to steel than air or water can still corrode piles, rates range from 0.5 to 7.5 millimetres (0.02 to 0.3 inches) per 100 years (Fleming et al., 1992). Despite these rates, the rule of thumb for the amount of corrosion that steel undergoes is on average 1 to 2 millimetres (0.05 to 0.1 inches) before the rust acts as a protective barrier. Other methods of protecting piles are discussed below in section 2.4.

Composite piles are sensitive to the same limitations of driving stresses, maximum lengths permissible, loads supported, and susceptibility to physical, biological, and chemical damage as piles made from single materials. Additionally however, while joining two dissimilar piles, both with respect to composition and geometry, every measure is taken to ensure that the contact between piles is flush, properly aligned, and structurally sound (Sowers, 1979). Piles with poor splicing may become damaged relatively easily during driving, while undergoing tensile loads or from environmental action faster than piles composed from a single material.

2.4 Pile Protection

To safeguard timber piles, the wood is physically covered or undergoes chemical treatment. Physical protection includes measures such as engineering the soil (fill) around the pile, leaving a layer of bark on the outside of the pile, adding concrete sleeves to shield vulnerable areas against deterioration or applying a coating of coal, tar, creosote, or petroleum to the outside of the pile as well as maximizing the effectiveness of the protection by the addition of heating or pressurized treatments (Chellis, 1961; CITC, 1962). These provide protection by poisoning the food source of insects and minimizing the physical reaction from water action.

Concrete piles are susceptible to corrosion by air, water, and soil; especially by the air in industrial areas with certain pollutants, if the water is very saline or if the soil is acidic. To protect the reinforcement inside concrete piles, a protective layer with a minimum thickness of 50 millimetres (2 inches) is often suggested (Sowers, 1979). Other types of physical protection include painting the pile, impregnating the concrete with asphalt, coating the pile with shotcrete, epoxy, resin, bitumen or adding a wood jacket, making the concrete mix denser than usual during pouring, adding specialized cements or encasing the pile in steel or pvc liners (Chellis, 1961; Fleming et al., 1992).

Steel piles are susceptible to rusting and pitting which may cause premature failure and is somewhat preventable by painting, tarring, galvanizing or cathodic protection. Steel piles are also encased with concrete armour to aid in slowing the decay of metal (Sowers, 1979). Adding copper to the steel, approximately 0.2 percent, aids in protection against the pile reacting with air (Chellis, 1961). Other properties that are altered are that the thickness of the web or flange is increased to allow or compensate for the occurrence of some pile damage (Fleming et al., 1992).

When piles are made from a combination of materials it is to increase the service life of the foundation. For example, piles installed near open water that are subjected to the tides or those embedded into the earth where the groundwater level fluctuates seasonally has the lower half, beneath where the water level is constantly wet, composed of timber and in the swash zone above it they are composed of concrete. Guidelines also exist for other combinations of material such as for a concrete – steel pile which suggest that the steel portion is driven at least five metres into the concrete portion of the pile.

During hammer installation, shoes are often employed to protect the tip of the pile, especially during hard driving. These attachments are often made from hardened steel and pointed to allow the pile to break through hard soil strata, thin rock layers, cobbles, and boulders without damaging the pile. To prevent timber piles from brooming a steel cap or band is often placed at the top of the pile to protect it throughout the driving process (Das, 2004).

2.5 Pile Groups

The main purpose of piles is to support loads from superstructures above them and this is most often accomplished by distributing the weight of the load among multiple piles, known as a pile group, via a cap.

The majority of analysis on pile capacity is performed on single piles however the majority of piles in use are bundled together in groups of at least three. The amount and distribution of load transferred to an individual pile within a group compared to a single pile by itself is required. If known, this aids in counteracting eccentric loadings and in locating where individual piles are driven. Additionally, the pile groups' efficiency factor is also required. A pile group's efficiency is determined by comparing the ultimate load that the group can safely support to the summation of the individual loads that each pile within the group can support. For end bearing piles the group efficiency is taken as approximately one, for driven piles in sand the efficiency factor typically ranges from one to as high as two or three, for driven piles in clays the efficiency factor is less than one, and for bored piles the efficiency factor is taken as about two-thirds, to account for the lack of increased frictional resistance during installation and the decreased internal angle of friction (Sowers, 1979; Knight 2008).

End bearing piles support loads rather well since the majority of the stresses is carried down to the hard or impermeable stratum on which the tips rest; this results in negligible amounts of settlement and practically no foundational failure, unless the loading exceeds the strength of the pile material itself, as mentioned above. Frictional piles however are more susceptible to failure; either because of loading which may cause excessive amounts of settlement via punching or shear failure of the soil, or differential settlement due to eccentric loading (Chellis, 1961). Therefore the remainder of this section only discusses the behaviour of friction piles in groups.

As previously mentioned; single piles driven in clay behave and carry loads differently than single piles driven in sand. This is also true for groups of piles driven in clays compared to in sands. Pile groups in clay act as one large cohesive unit or block encompassing the outside area of the installed piles. This block develops shear failure planes like other structures in or on clay. The more piles in clay the easier it is for the block to fail. Since the efficiency factor is less than one, the total loads applied to the group of piles is one third to one half of the ultimate load which the pile group can support (Terzaghi and Peck, 1960) as a precaution or factor of safety against excessive settlement and punching failure.

Research shows that the usual sub surface area affected by pile driving is at least five pile diameters away from the driven pile. It is also shown that piles driven four or more pile diameters apart on centre have no practical effect on adjacent piles and cause the pile cap to cover too large an area to be economical. The exception to this is when driving piles in marine environments; in which case piles are spaced at least five diameters apart, centre to centre distance, to avoid abrasion and eddying (Chellis, 1961). Piles which are spaced closer than two pile diameters apart on center in cohesive soils can cause stress zones to overlap so much so that they can cause failure simply from the act of driving and preliminary loading if not the full load itself despite the results from predictive capacity analysis and pile load tests. As such the usual spacing of piles in groups is between two and half to four pile diameters apart centre to centre distance (Terzaghi and Peck, 1960). In cohesionless soils, driving piles beside other previously driven piles further increases the soils density and thus the internal angle of friction which then allows the piles able to support larger loads due to the developed overlapping stress bulbs. In rare cases, the bearing capacity of the soil decreases a short while after initial driving (Terzaghi and Peck, 1960). This is caused by the temporarily high developed stresses at the pile tip during driving, after the stresses dissipate the soil relaxes and as such the bearing capacity they can support is less than indicated during the original driving process. However, piles driven too close together can cause an unacceptable amount of soil heave and cause adjacent piles to move laterally or even vertically up from the ground possibly damaging the pile itself, separating spliced segments, squeezing freshly poured concrete, shearing pile joints or crushing pile casings in the process.

Formulae such as the one listed below are empirically derived to predict the efficiency of the pile group based upon spacing, both longitudinally and laterally rather than pile type, length, shape, soil properties, or method of installation.

In Ontario the most commonly used formulae is the Converse Labarre Method, as seen below:

$$Efficiency = 1 - \varphi \left[\frac{(n-1)m+(m-1)n}{90mn}\right] \tag{3}$$

where: φ is equal to the angle whose tangent is the quotient of the pile diameter to the pile spacing, m is the number of rows in the pile group, and n is the number of piles within each row.

Other methods to determine group efficiency are the Los Angeles Group action, Masters, Feld, Seiler and Keeney, Pressure Area, and Cylindrical Pier methods as well as direct measurements such as the Pretest method (Chellis, 1960). These methods take the pile lengths into account calculating stress from Boussinesq equations, are modified Converse Labarre methods, are based on rules of thumb, are a combination of the Converse Labarre and Masters methods, are based on the pile group area, and assume that the group acts as one large pier whose ultimate carrying capacity is the sum of the carrying capacity of an equivalently sized pier plus the product of the shearing resistance per unit area of the pile and the outside area of the pile group, respectively. Direct measurement methods either test pile capacities in groups or record individual pile reactions during driving via strain gauges. Whichever method is used, for a group with the same number of piles the efficiencies range from just under 20 and up to 75 percent, as seen in Figure 8 (Chellis, 1960).

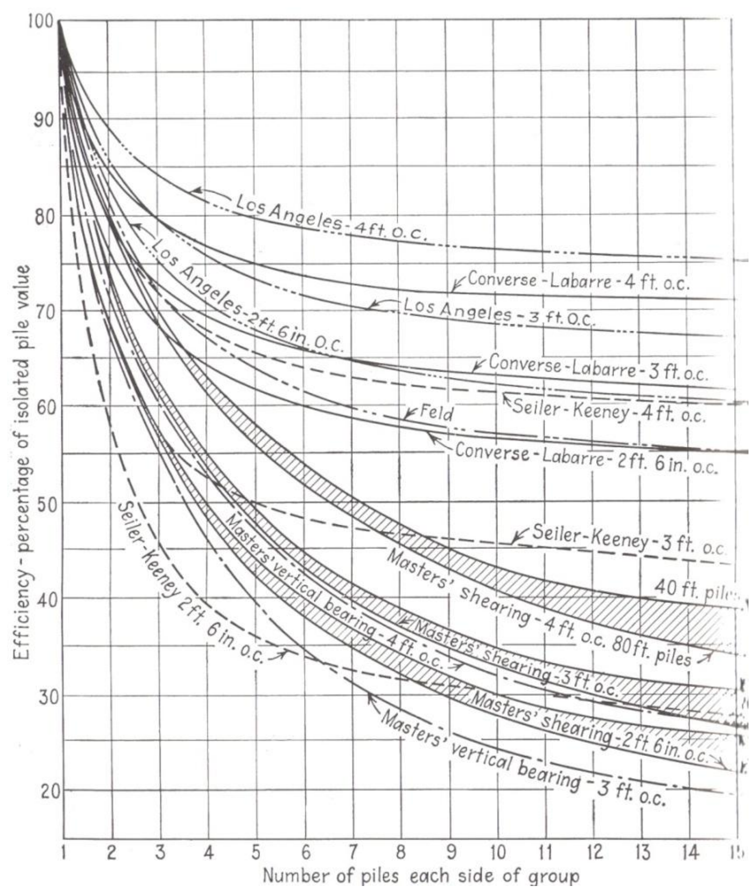


Figure 8: Comparison of Pile Efficiencies by Various Methods (Chellis, 1960).

From studying the behaviour of piles in groups some conclusions are determined by Chellis, 1960. He determined that friction bearing piles in rectangular and circular groups can support greater loads before succumbing to shear failure than piles in square groups. Additionally, that a group composed of a small number of long piles can support loads better than a pile group of equal area which consists of a greater number of piles driven to shallower depths spaced more closely together. This is because the generated stress bulbs of piles spaced further apart do not overlap as much as closely spaced piles and that the shearing value per pile length is reduced in long piles when compared to short piles.

2.6 Alternatives To Conventional Piles

For completeness, alternatives to pile driving are presented as potentially viable options depending upon the soil properties and bearing capacity required. They include chemical and physical modifications such as grouting, pneumatic, vibratory, and thermal methods as well as variations based on conventional pile technologies.

Grouting is a general term referring to a number of processes which are used to increase soil strength and decrease permeability. Ideally the effects are permanent while not being detrimental to the soil or further soil modifications. Methods include silt injection, cement, bituminous, and chemical grouting. Silt injection is the addition of a silt slurry into the subsurface to consolidate fine grained porous soils. Cement grouting consists of pumping pure cement, cement and clay, cement and sand or any combination thereof and is limited only to sand sized or larger soil particles. If the soil contains more than 0.5 percent sodium, calcium, magnesium or potassium sulphates, cement injected in the soil may be destroyed within 10 years (Chellis, 1961). Sulphides, organic acids, dissolved carbon dioxide, and pH values less than 6 and greater than 8 are also detrimental to cement addition. Cemented soils can support loads up to 120 kN/m^2 (2 500 lb/ft^2) without causing noticeable settlement (Chellis, 1961). In cemented cohesive and cohesionless soils, unconfined compressive strengths (UCS) of up to 1 400 kN/m^2 (200 lb/in^2) and 10 350 kN/m^2 (1 500 lb/in^2) are achieved, respectively (Sowers, 1979; Das, 2004). Bituminous grouting consists of adding emulsified asphalt mixed with cement to fine grained porous soils up to 2 millimetres (0.08 inches) in diameter and increase compressive strengths up to 960 kN/m^2 (10 $tons/ft^2$) and shearing resistance up to 350 kN/m^2 (50 lb/in^2) (Chellis, 1961).

Chemical grouting is the addition of two or more solutions which react and harden in order to increase the bearing capacity of soils. Usually a silicon solution and any heavy metal salt such as calcium chloride, hydrogen chloride, calcium hydroxide or aluminum sulphate combine to produce a gel like substance which binds soil grains together to form an artificial sandstone like mass (Sowers, 1979). Strengths in excess of 1 900 kN/m^2 (20 tons/ft^2) to 4 800 kN/m^2 (700 lb/in^2) and tensile strengths of 1 080 kN/m^2 (157 lb/in^2) are observed in fine sands and compressive strengths up to 8 300 kN/m^2 (1 200 lb/in^2) are reported in gravels (Chellis, 1961) in deposits up to 35 metres (115 feet) thick (Das, 2004).

Physical improvements to soils include compaction by, dynamic and vibratory (vibroflotation, vibro compaction and vibro replacement) forces, stone columns, sand compaction piles, in situ reinforcement, and modified piles.

Dynamic Compaction is accomplished by dropping weights from 4 to 36 Mg (5 to 40 tons) (Coduto, 2001) with 20 000 kg (44 000 pounds) force from a height of 6 to 30 metres (20 to 100 feet) which impacts an area from 2 to 4 square metres (22 to 43 ft^2) (Chellis, 1961). This soil is compacted by both vibrations and impact forces which propagate down 5 to 10 metres (16 to 32 feet) below ground surface. Vibroflotation, vibro compaction or vibro replacement are other methods which use vibration and jetting to form an opening 2 to 3 metres (8 to 10 feet) in diameter up to 15 metres (50 feet) deep (Sowers, 1979) and compacts soil particles by rearranging them increasing their overall density, compressive strength, and shear resistance.

Thermal methods such as heating or freezing may also be used. Freezing temporarily increases soil strength from; 6 to 14 MN/m^2 (125 to 290 kip/ft^2) in clean sands, 4 to 13 MN/m^2 (80 to 270 kip/ft^2) in sandy silts and 1.5 to 6 MN/m^2 (30 to 125 kip/ft^2) in clays, depending upon the temperature applied. Heating is another method which desiccates the soil by creating temperatures greater than 1 100^{o}C (2 000^{o}F); the process is permanent and can consolidate soils up to a depth of 10 metres (32 feet) (Chellis, 1961; Sowers, 1979).

Micro piles, also known as pin or root piles are typically 50 to 250 millimetres (2 to 10 inches) in diameter, installed to lengths of 20 to 50 metres (66 to 170 feet) (Sowers, 1979) below ground surface and are able to support loads from 20 to 500 kN (5 to 110 kips) (Fleming et al., 1992). Like customary piles, micro piles are installed via pneumatic, vibratory or augured methods. They can also be spliced or threaded together or formed by pouring concrete with or without a casing or reinforcement. They are typically used where overhead space is limited, near pre-existing structures which may be negatively affected by conventional installation methods or as a corrective measure for current foundations.

3.0 PILE DRIVING EQUIPMENT

Bearing capacities of various pile types due to frictional and tip resistance are discussed in Chapter 2. This chapter focuses on various pile driving rigs (see Figure 9), how they change and develop with time, their physical properties, and the mechanisms in which they install various piles.

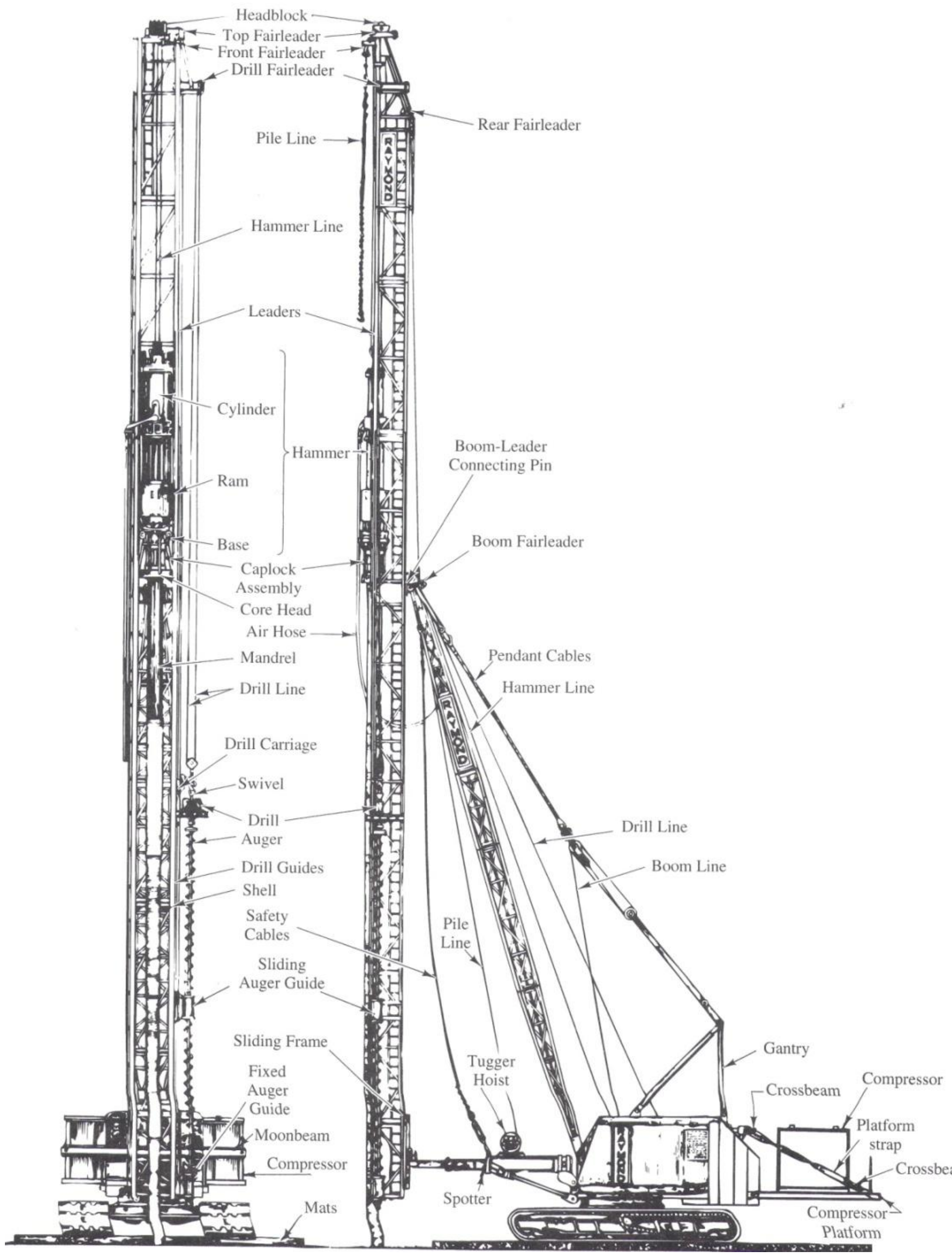


Figure 9: Pile Rig Components (Coduto, 2001)

Historically, pile drivers have been around since the act of piling itself. From 12 000 years ago until the early 1800s, the technology for installing piles as well as piles themselves remained relatively constant. As previously discussed with piles, pile drivers themselves have also evolved with time.

Since the MTO database used primarily for analysis employs drop and diesel hammers exclusively, only they are presented in detail. However, other types of pile drivers such as steam powered, gun powder, hydraulic, and vibratory drivers exist as well as pile installation by jacking, jetting, boring, excavating, and driving by electroosmosis are also discussed.

3.1 Drop Hammers

The earliest pile drivers are simple gravity pulley systems in which a weight free-falls onto the head of a pile via a trigger mechanism or actuator. The weight is guided by leads and raised by man power through a rope and winch. In the 1700s, drop hammers with ram weights from 1.2 to 3.5 kN (0.27 to 0.79 kips) achieved blow rates as high as 25 to 30 times per minute; however, since these are man powered a break of 1 to 2 minutes normally occurred effectively lowering the blow rates to 8 to 10 blows per minute, resulting in a relatively slow and cumbersome progress. In the early to mid-1800s, weights as much as 15 kN (3.4 kips) have been used and raised by man or horse power (Fleming et al., 1992). More recently, weights from 2 to 13.3 kN (0.5 to 3 kips) are in use with a drop height from 2 to 3 metres (6 to 10 feet) (Sowers, 1979). The energy imparted on the pile per hammer blow is the product of the ram weight and the height of the fall.

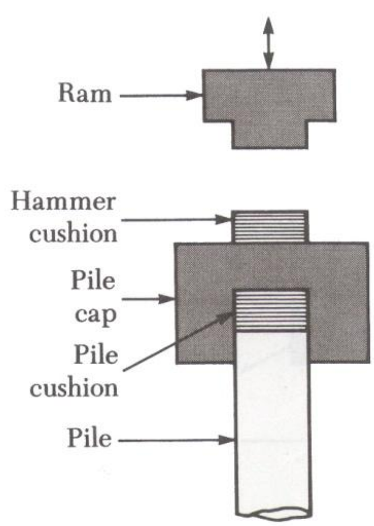


Figure 10: Drop Hammer Components (Das, 2004)

3.2 Single Acting Steam Hammers

With new innovations in technology pile drivers went from simple gravity systems with a large drop weight to single acting steam hammers developed in 1801 by John Rennie in Britain (Fleming et al., 1992). Powered by steam, steam or air enters the base of the chamber and uses an engine to raise a ram 0.6 to 1 metre (2 to 3 feet) high inside a cylinder and drop due to gravity in a manner similar to the drop hammer. Hammer weights range from 22 to 2 669 kN (5 to 600 kips) and result in producing hammer energies from 20 to 2 500 kN·m (15 to 1 670 kip·ft). The blows are relatively slow with most of the energy coming from the mass of the ram, however, rates of 50 to 80 hammer blows per minute are achieved (Sowers, 1979; Fleming et al., 1992).

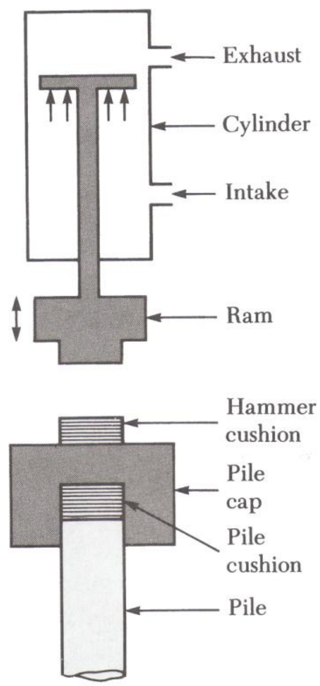


Figure 11: Single Acting Steam Hammer Components (Das, 2004)

After 40 years, the original single acting steam or air hammers are modified to a double acting steam hammers (Fleming et al., 1992).

3.3 Double Acting Steam Hammers

Double acting steam hammers lift the ram in the same method as single acting hammers; however, the top of the cylinder contains a valve which allows steam to force the ram down and was first developed in 1843 by Nasmyth (Fleming et al., 1992) and enables blows from 80 to 240 blows per minute. This creates more energy imparted to the pile than gravity can provide alone. Ram weights range from 22 to 62 kN (5 to 14 kips) and imparts energy from 22 to 49 kN·m (16 to 36 kip·ft) onto pile heads. The energy of the hammer is calculated by the following formula (Chellis, 1961; Fleming et al., 1992):

$$E_n = \frac{W_r v^2}{2g} \tag{4}$$

where: E_n is the energy imparted from the hammer to the pile (kN·m or kip·ft), W_r is the weight of the ram (kN or kips), v is the velocity of the ram fall (m/s or ft/s), and g is the gravitational constant of 9.81 m/s^2 or 32.2 ft/s^2.

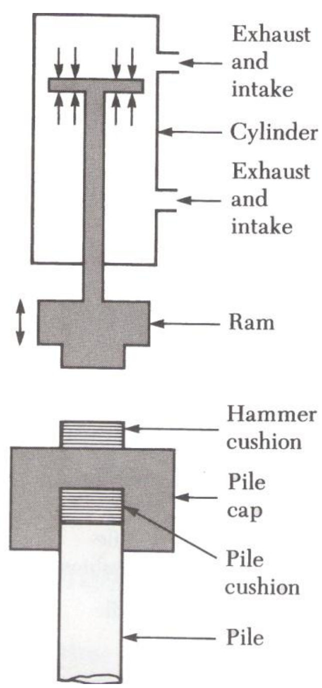


Figure 12: Double Acting Steam Hammer Components (Das, 2004)

3.4 Gun Powder Hammers

This type of hammer was in use for only a short period in the early 1870s by Shaw in Philadelphia (Fleming et al., 1992). It employs a mechanism similar to that of diesel hammers however instead of fuel being ignited it is gun powder. This has been stopped because it is dangerous for the operator and extremely loud. Blow rates of 15 to 20 blows per minute have been achieved and as many as thirty piles 1.8 to 2.5 metres in length could be driven per day (Fleming et al., 1992).

However, gun powder powered hammers are innovative in the sense that they only required two people to operate, drivers before this usually required a team of about four to eight men to operate. This style no doubt led to diesel powered drivers that are now well known and used throughout the world today.

3.5 Diesel Hammers

Diesel hammers became common use in 1946 (Fleming et al., 1992). The physical action of the hammer is similar to that of steam hammers. In single acting diesel hammers the ram is raised mechanically and allowed to fall initially due to gravity. In this case, diesel fuel is injected into the bottom of the cylinder as the ram reaches its apex. As the ram falls the compressed air and heat causes an explosion to occur which forces the cylinder down onto the pile and the ram upwards, where fuel is injected again causing the cycle to repeat. In double acting diesel hammers; fuel is also injected into the top of the cylinder and ignited to cause the ram to travel down the cylinder. Diesel rigs utilize faster hammer blows and produce more energy per blow, 13 to 267 kN·m (9 to 197 kip·ft) than earlier models, use smaller ram weights from 8 to 78 kN (1.8 to 17.6 kips), and in hard driving rates from 39 to 84 blows per minute are possible (Sowers, 1979; Fleming et al., 1992). A disadvantage of diesel hammers is that the energy transferred to the pile depends largely upon driving resistance and is therefore very variable. During hard driving conditions, most of the theoretical maximum produced energy impacts the pile head, however in easy driving; the hammer produces less energy per blow.

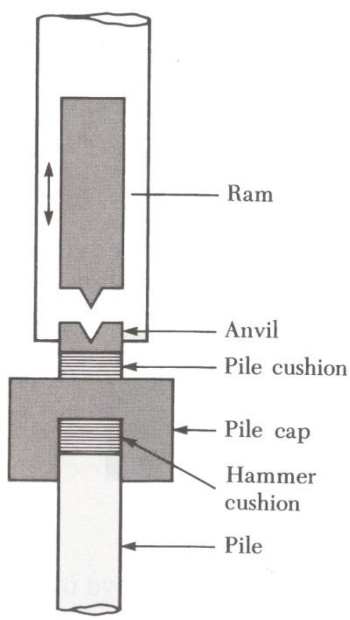


Figure 13: Diesel Hammer Components (Das, 2004)

3.6 Hydraulic Hammers

One of the most recent types of hammers developed, in the 1960s, use hydraulic lines to add and decrease pressure to raise and lower the hammer weight and have gained more and more popularity since the 1990s (Fleming et al., 1992). The mechanisms involved are the same as the double acting hammers, however since hydraulic fluid is used in place of steam or diesel, the driving operation is faster, quieter, and environmentally cleaner. The high pressures produced allows the same energy to be transmitted to the pile during driving as steam hammers but with smaller ram weights; unfortunately, they also cause more mechanical wear of the pile rig and driving equipment. Ram weights typically used range from 29 to 290 kN (6.5 to 65 kips), fall distances of 0.2 to 1.2 metres (0.7 to 3.9 feet) and result in striking energies of 26 to 350 kN·m (19.5 to 262.5 kip·ft) with blow counts of 28 to 130 times per minute (Sowers, 1979).

3.7 Vibratory Hammers

Not as common as percussion style hammers, vibratory hammers install piles by applying a constant downward force imparted by two counter rotating weights. These hammers have the advantage of being much quieter and thus less intrusive to local areas than percussion hammers. The vibrations cause the soil beneath the pile to loosen thus advancing the pile under its own weight. Vibrators normally operate at frequencies from 12 to 40 Hz (Fleming et al., 1992) but may operate at frequencies as high as 150 Hz and weigh between 6.7 and 169 kN (150 and 3 800 pounds) (Sowers, 1979 and Coduto, 2001). As the driver weights rotate, the eccentricities cause the horizontal components of the generated force to cancel with each other resulting in only a net sinusoidal vertical force that allows the pile to travel downward. When the vibrators operate at a frequency above 100 Hz the pile and soil may act in resonance causing the pile to advance easily. Rates of 5 and up to 20 metres per minute are observed for dense granular and loose to moderately dense soil, respectively (Fleming et al., 1992). When vibrators are operated at these relatively high frequencies the pile advances quickly; however, the dynamic force created often damages the vibrator thus advancing at resonance is not common.

This method is effective in cohesionless but less so in cohesive soils; the disadvantage to the developed bearing capacity is that in sands and gravels the driving action fluidizes the soil, greatly reducing the friction between the pile and the soil, thus vibratory installed piles rely primarily on end bearing support.

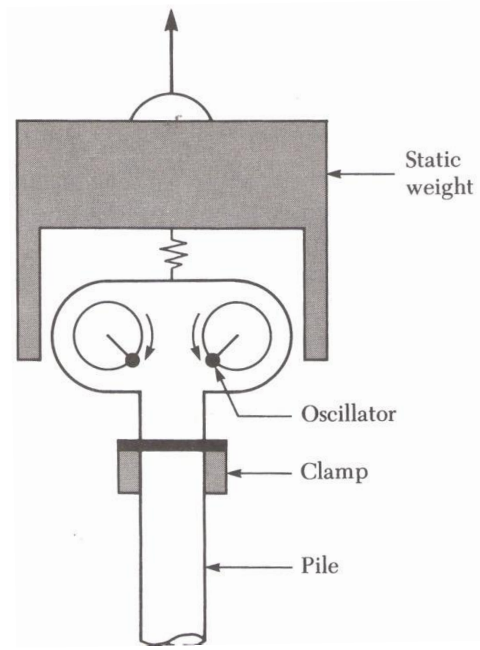


Figure 14: Vibratory Hammer Components (Das, 2004)

3.8 Excavating And Boring Methods

Excavated or bored piles are developed as an alternative to conventional pile driving since they are constructed to any diameter or length required, depending upon machine limitations and can support much larger loads than driven steel piles alone. Concrete cast in Place piles, as they are commonly constructed now, first came in use in 1897 by A.A. Raymond. In 1903 R.J. Beale invented a technique to drive hollow tube piles and fill them in with concrete as the tube is withdrawn (Fleming et al., 1992).

Bored piles came into common use in the United Kingdom since the early 1930s, installed by percussion equipment and later in the 1950s by rotary auguring machines.

3.8.1 Screw Piles

First used in 1838 by Alexander Mitchell (Fleming et al., 1992), screw or helical piles densify the soil rather than remove it as is done by boring or excavating methods. The auger head is advanced downwards and the soil underneath it becomes compacted, as the head is retreated, a hollow stem concrete is filled into the opening to form the pile. The process is shown below in Figure 16:

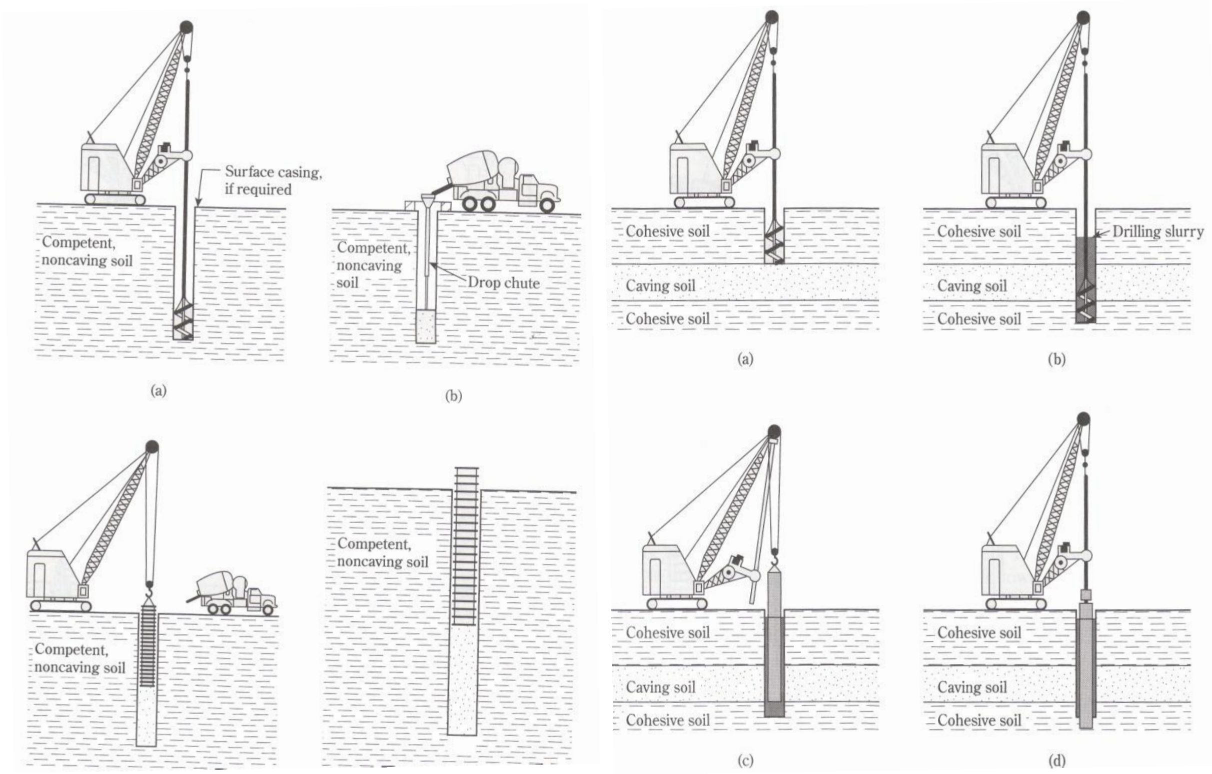


Figure 15: Bored Hammer Components (after Das, 2004)

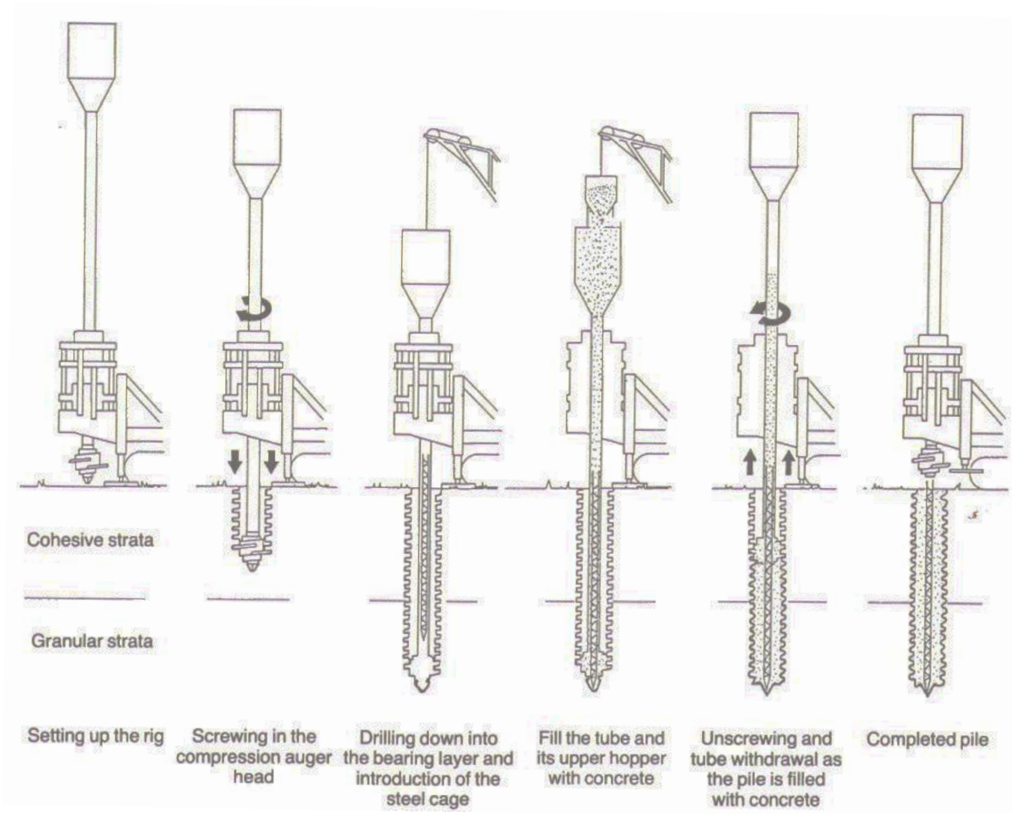


Figure 16: Atlas pile installed by screw method (Fleming et al., 1992)

3.8.2 Pressure Injected Footings

Pressure injected footings (PIFs) were developed pre-World War I (Coduto, 2001). The process consists of driving a tube 300 to 600 millimeters (12 to 24 inches) in diameter by placing a hollow tube with either a plate on top or a concrete plug in the bottom and then hammering the tube into place. Once specified depth is reached, the tube is held in place as concrete is poured into it and compacted via hammer blows. This compaction causes the injected concrete to form a bulb at the bottom of the tube which increases end bearing resistance for compressive loads and tensile resistance for uplift loads. E. Frankignoul from Belgium, in 1908, invented an early Franki driven tube pile system (Fleming et al., 1992). Once the specified amount of concrete is placed or a specific concrete is achieved, as indicated by the number of hammers blows, the shaft of the pile is constructed. This is done by either raising the tube while hammering additional amounts of concrete into the hole or inserting a shell and filling it with concrete.

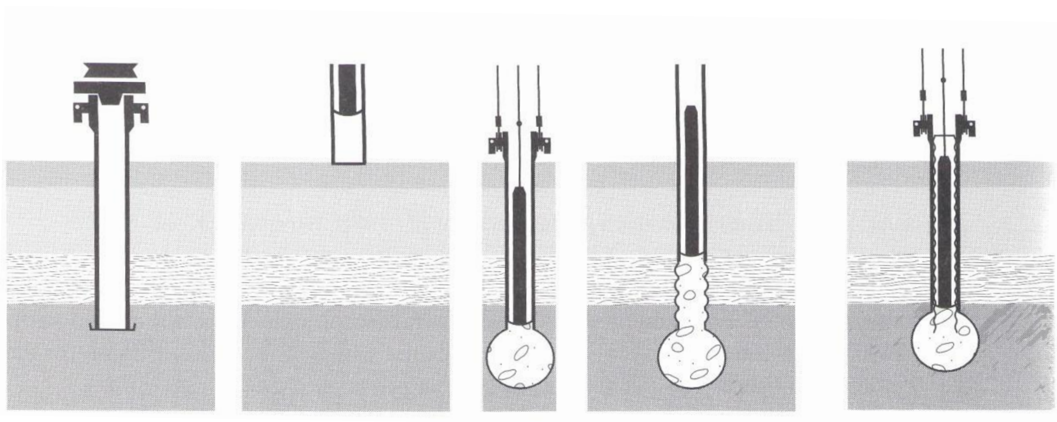


Figure 17: PIF Hammer Components (Coduto, 2001)

3.8.3 Spudding

Spudding involves attaching advancing hardened steel points. A point is advanced until it reaches a specified depth, usually past a dense layer of sand, gravel or rock. Once through, the metal point is extracted and the pile is driven through the formed hole.

3.9 Jetting

Similar to excavating and boring methods, jetting uses a fluid in advance of the drill bit to aid in the drilling of soil.

Jetting is the process where a nozzle which pumps high pressurized water is attached to the tip of the pile. The process loosens the soil below the pile tip so that little or no hammer blows are required to advance the pile. This procedure is effective when piling into sands and gravels but not when clays are encountered.

3.10 Pile Jacking

Pile jacking is performed by driving piles against a reaction, usually a structure already in place, and used for underpinning. It is most commonly used to drive micro and sheet piles where overhead spacing is limited and therefore traditional pile driving rigs are employed. It has the advantage of being quieter and causing fewer vibrations than percussion pile drivers.

3.11 Electroosmosis

Electroosmosis originally developed as a method of strengthening soil and has been in use since the 1930s and 1940s (Chellis, 1961). The process involves placing anodes and cathodes into the subsurface and applying a current through them. Typical currents and voltages used range between 20 to 30 amps and 30 to 180 volts direct current, respectively. This causes the pore water to travel from the positive electrode to the negative one as well as the soil to dry out near the anode. The water near the cathode is then pumped out via extraction wells. Records show pumping rates to improve by a factor of nine to ten when electroosmosis is used in cohesionless soils. In cohesive soils the drying effect is known to increase shear resistance by 500 percent (Chellis, 1961). The drying effect is temporary and as the moisture returns a reduction in the soil shear strength is expected.

In 1953 H.K.S. Begemann began experiments using electroosmosis in conjunction with percussion pile driving to aid pile advancing piles in clays (Nikolaev, 1962). A cathode is attached to the tips of the piles being driven; this causes the groundwater to develop around them creating a film, which aides in pile driving. The excess water reduces the frictional resistance of the soil and allows the pile to achieve greater depths than by conventional methods with less hammer blows. Once the pile reaches its specified depth the electrodes are turned off and the soil regains its original strength; to accelerate the process the poles are reversed, turning the cathode into the anode, causing the soil at the pile tip to dry thus increasing its frictional resistance and shear strength.

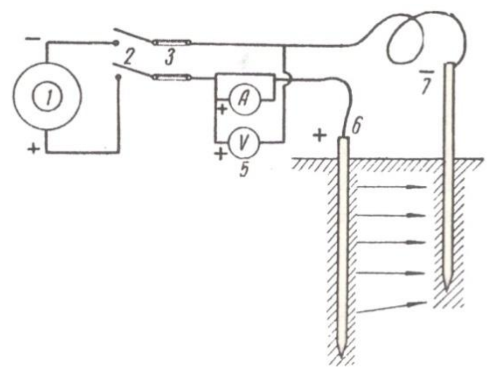


Figure 18: Electroosmosis Pile Setup (Nikolaev, 1962)

4.0 PREDICTIVE BEARING CAPACITIES

The maximum ultimate load which a pile can support is determined by the lower value of either the compressional strength of the pile itself or the bearing capacity of the soil. Pile material strengths and maximum allowable loading values are previously discussed in sections 2.1.4 and guidelines for end bearing capacities and frictional resistance values based primarily on experience and engineering judgement are given in section 2.2. The ultimate load depends upon the pile composition and its cross sectional area; whereas, the ultimate bearing capacity derived from the soil depends upon the soil properties, the method of installation, and the embedded pile length. To safely support any structure, knowing the theoretical maximum load a pile can sustain is not sufficient for design purposes. The corresponding amount of acceptable settlement that occurs is also determined for design acceptability. There are many techniques available to predict the soil – pile bearing capacity and the resulting settlement.

These predictive methods range from general rules of thumb as stated above to theoretical static formulae based on soil properties derived from field and laboratory testing to semi empirical dynamic formulae based on site observations made during pile driving to pile load tests based on the reactions of fully installed piles.

Often times, all methods are used in conjunction to assure that the piles installed are able to support their designed capacity without allowing excessive settlement to occur. Theoretical formulae are used as a first step to determine approximate pile capacities based upon pile type, length, and soil conditions. They are regularly used as a design tool varying pile lengths, widths, material types, and installation methods until a combination is found which can withstand the required load with an appropriate factor of safety as well as being the most economically viable option.

Methods based on site observations such as energy or momentum equations with appropriate factors of safety are used as a guide to aid in determining required embedded pile lengths needed to support design loads or developed pile bearing capacities depending upon pile resistance measurements made through hammer blow counts.

In situ techniques such as pile load tests are performed on certain test piles, depending upon the number of piles installed or the number and confidence of geotechnical tests performed during site investigations prior to pile design, as a check to confirm pile capacity and to determine expected settlement values.

All of these techniques are discussed below for various soil and pile types.

4.1 Theoretical Predictive Formulae

Static formulae are used to calculate pile capacities based on the results of field and laboratory tests such as vane shear, standard penetration test (SPT), cone penetration test (CPT) triaxial, oedometer, hydrometer, grain size analysis, and moisture content tests to determine soil properties such as undrained shear strength (c_u), effective internal angle of friction (φ'), effective cohesion (c'), water table levels, soils density (D_r), effective soil unit weight (γ'), preconsolidation pressure (σ_c'), and overconsolidation ratio (OCR).

Various formulae are developed to predict pile reaction to being driven in sand, clay or rock as well as being driven by percussion or boring methods. With relevant site conditions known, the required pile capacity is computed by modifying the length of the pile; increasing it to add more frictional resistance or having the end terminate into various competent soil layers to determine which length results in the required capacity. The pile composition as well as installation method are varied to determine the effect that occurs on the frictional and end bearing resistance established.

The ultimate capacity of a pile (Q_u) is the summation of two factors; the end or point bearing capacity of the pile (Q_p) and the frictional or skin resistance of the pile – soil system (Q_s).

The end bearing resistance is given by a modified version of Terzaghi's equation used for determining the bearing capacity of shallow foundations, as presented in Das, 2004 (Equation 5). The principle differences are that the effective vertical stress is used during the calculation to account for the length of pile installed below the water table and the term involving the width of the foundation is not accounted for because it is negligible compared to the overall foundation length.

$$Q_p = A_p\left(c'N_c^* + q'N_q^*\right) \tag{5}$$

where: Q_p is the end or point bearing capacity of the soil at the pile tip, A_p is the cross sectional area of the pile tip, c′ is the effective cohesion of the soil supporting the pile tip, q′ is the effective vertical stress at the pile tip, and N_c^* and N_q^* are bearing capacity factors.

The frictional resistance is given by Equation 6 below (Das, 2004)

$$Q_s = \sum p\Delta Lf \tag{6}$$

where: Q_s is the frictional resistance of the pile (kN or lb) which is the sum of; *p*, the perimeter of the pile (m or ft), ΔL, the incremental length (m or ft) over which the p and *f* are taken as constant, and *f*, the unit frictional resistance (kN/m^2 or lb/ft^2) at the vertical depth, z (m or ft), being examined.

Described below are various techniques for calculating Q_p and Q_s, depending upon if the pile is embedded in sand, clay or rock for several methods of pile installation.

4.1.1 Piles In Cohesionless Soils

For piles in sand three basic techniques are used to determine pile capacity; the Meyerhof, standard penetration test (SPT), and cone penetration test (CPT) equations. However, many other methods such as those of Vesic, Janbu and Coyle, and Castello are in use as well to predict static pile capacity (Das, 2004).

4.1.1.1 Meyerhof Method

The cohesion value of clean free draining sands is taken as zero; as such Meyerhof's end bearing formula is modified from Equation 5 to:

$$Q_p = A_p q' N_q^* \tag{7}$$

where: Q_p is the end bearing capacity (kN or lb), A_p is the cross sectional area of the pile (m^2 or ft^2), N_q^* is a bearing capacity factor, and q′ is the effective stress at the pile tip (kN/m^2 or lb/ft^2) which is given by:

$$q' = (\gamma')z \tag{8}$$

where: γ' is the effective unit weight of the soil (kN/m^3 or lb/ft^3) and z is the depth from ground surface to the pile tip (metres or feet).

For the portion of the pile below the water table, the effective unit weight is given by:

$$\gamma' = (\gamma_T - \gamma_w) \tag{9}$$

where: γ_T is the total unit weight of the soil and γ_w is the unit weight of water, taken as 9.81 kN/m^3 or 62.4 lb/ft^3.

Meyerhof's modified values of N_q^* for deep foundations are given in Figure 19 below:

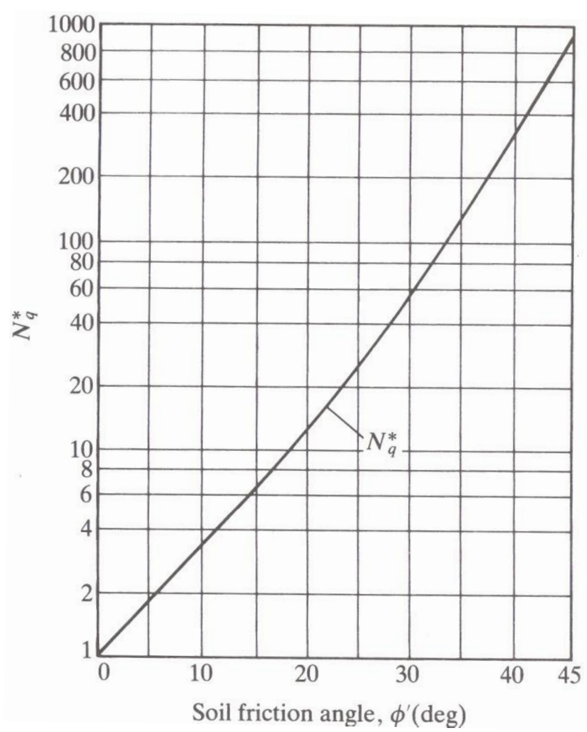


Figure 19: Bearing Capacity Factor N_q^* (Das, 2004)

The unit end bearing capacity of the sand layer is not infinite and Meyerhof suggested limiting it to no more than a value of q_l, given by:

$$q_l = 0.5 p_a N_q^* tan\varphi' \quad (10)$$

where: q_l is the limiting point bearing resistance, p_a is atmospheric pressure, taken as 100 kN/m^2 or 2 000 lb/ft^2, ϕ′ is the effective friction angle of the soil that the pile tip rests in (degrees), and N_q^* is the same bearing factor as given above in Figure 19.

Meyerhof, in 1961, presented an equation to estimate frictional resistance of piles in sand. To account for the changes in soil conditions during driving, Meyerhof created certain assumptions based on observations and test results. These lead to Equation 11:

$$f = K\sigma_o' tan\delta \quad (11)$$

where: *f* is the unit frictional resistance (kN/m^2 or lb/ft^2), K is the effective earth coefficient, σ_o' is the effective vertical stress (kN/m^2 or lb/ft^2) at the depth being analyzed, and δ is the soil – pile friction angle (degrees).

Similarly to Meyerhof's end bearing equation, the frictional resistance developed from pile driving is also bounded by an upper limit. This is based on observations that the frictional resistance of sand increases with depth until the pile reaches a critical depth of 15 to 20 pile diameters. To be conservative *f* is taken to increase with depth until 15 pile diameters and thereafter is taken as constant for the remainder of the pile length.

The effective earth coefficient (K) is approximately equal to the at rest earth coefficient (K_o) and changes depending upon pile installation method as given in Table 8.

Table 8: Effective Earth Coefficients Depending on Installation Method (after Das, 2004)

Pile Type	Effective Earth Coefficient (K)
Bored or Jetted Pile	1-sinφ′
Low Displacement Driven Pile	1.4(1-sinφ′)
High Displacement Driven Pile	1.8(1-sinφ′)

The effective vertical stress, σ_o', is calculated in the same manner as q′ for end bearing capacity and δ, the soil-pile friction angle, ranges from 0.5φ′ to 0.8φ′ degrees. From investigation it appears that for high displacement piles, the value of δ is equal to 0.8φ′ and for non-displacement or bored piles, the value of δ is equal to 0.5φ′.

Another suggestion is that K is set as 1.0 and 2.0 for concrete piles driven in loose and dense sand, respectively, and 0.5 and 1.0 for steel H piles driven in loose and dense sand, respectively. The value of δ is set equal to 0.75φ′ for concrete piles and 20 degrees for steel piles (Craig, 2002).

4.1.1.2 Beta Method

Due to the rapid dissipation of excess pore water pressure in sands and gravels, the bearing capacity of cohesionless soils is calculated by the effective stress or beta method. The method is used to calculate the frictional and end bearing resistance. The frictional resistance is given below as:

$$f_s = \beta \sigma_z' \tag{12}$$

where: f_s is the unit frictional resistance, σ_z' is the effective vertical stress at the pile section being examined, and β is a function of the at rest and effective earth coefficients, the shaft – soil interface angle, and the effective internal angle of friction as given in Equation 13:

$$\beta = K_o \left(\frac{K}{K_o}\right) tan\left[\varphi' \left(\frac{\delta}{\varphi'}\right)\right] \tag{13}$$

The ratio of the effective to at rest earth coefficients and shaft – soil to effective internal friction angles are given in Tables 9 and 10 below.

Table 9: Ratio of Soil Pile to Effective Soil Internal Friction Angles (after Coduto, 2001)

Interface Materials	Typical Field Analogy	δ/φ'
Sand/rough concrete	Cast-in-place	1
Sand/smooth concrete	Precast	0.8 to 1.0
Sand/rough steel	Corrugated	0.7 to 0.9
Sand smooth steel	Coated	0.5 to 0.7
Sand/timber	Pressure-treated	0.8 to 0.9

Table 10: Ratio of Effective to At Rest Earth Coefficients (after Coduto, 2001)

Foundation Type & Installation Method	K/K_o
Driven Pile, small displacement	0.75 – 1.25
Driven Pile, large displacement	1 – 2

For large displacement piles, beta is simplified to Equation 14 as given by Coduto (2001).

$$\beta = 0.18 + 0.65D_r \tag{14}$$

where: D_r is the relative density of the sand layers being examined, in decimal form.

4.1.1.3 SPT Method

Many correlations exist for calculating the unit point resistance (q_p) for various soil types and installation methods, as shown in Table 11 below.

Table 11: Unit Point Resistance Correlated from Field SPT Values (after Das, 2004)

Soil Type	Unit Point Resistance Equation
Sand	$q_p = 19.7p_a(N_{60})^{0.36}$
Cast in Place Pile in Sand	$q_p = 3p_a$
Bored Pile in Sand	$q_p = 0.1p_aN_{60}$
Bored Pile in Gravelly Sand	$q_p = 0.15p_aN_{60}$
Driven Piles, all soils	$q_p = 0.3p_aN_{60}$

In the above formulae q_p is the unit point resistance (kN/m^2 or lb/ft^2), pa is atmospheric pressure (100 kN/m^2 or 2 000 lb/ft^2), and N_{60} is the SPT number obtained in the field. The N_{60} value is derived from an average number of blows near the pile tip. The area near the pile tip is defined as a vertical distance from 10 pile diameters above to 4 pile diameters below the tip (Das, 2004).

The unit point resistance is used to calculate the end bearing capacity with Equation 15 below:

$$Q_p = A_p q_p \tag{15}$$

where: Q_p is the point bearing capacity of the pile (kN or lb), A_p is the area of the pile (m^2 or ft^2), and q_p is the unit point resistance (kN/m^2 or lb/ft^2).

For homogenous granular soils, defined as those with less than 35% of samples passing through the number 200 sieve, Meyerhof advises limiting the unit point resistance (q_p), as defined in Das, 2004 to:

$$q_l = 0.4p_a(N_1)_{60}\frac{L}{D} \leq 4p_a(N_1)_{60} \tag{16}$$

where: q_l is the limiting unit point resistance (kN/m^2 or lb/ft^2), p_a is atmospheric pressure (100 kN/m^2 or 2 000 lb/ft^2), $(N_1)_{60}$ is the SPT value obtained from averaging those from 10 pile diameters above to 4 pile diameters below the pile tip, L is the embedded length of the pile (metres or feet), and D is the pile diameter or width (metres or feet). For bored piles unit point resistance is taken as one third the value given by Equation 16 (Craig, 2002).

To determine the frictional resistance through SPT counts Meyerhof correlates them with the average unit frictional resistance for low and high displacement piles as seen in Equations 17 and 18, respectively (Das, 2004).

$$f_{av} = 0.01p_a(\overline{N_1})_{60} \tag{17}$$

$$f_{av} = 0.02p_a(\overline{N_1})_{60} \tag{18}$$

where: f_{av} is the average unit frictional resistance (kN/m^2 or lb/ft^2) over the length which the corrected SPT count is constant, p_a is atmospheric pressure (100 kN/m^2 or 2 000 lb/ft^2), and $(\overline{N_1})_{60}$ is the average corrected SPT value. For bored piles the frictional unit resistance is taken as one half of the value obtained by Equations 17 and 18.

The average corrected SPT value$(\overline{N_1})_{60}$ is calculated by multiplying the field obtained SPT value (N_{60}) by a correction factor C_N. The corrected value is to account for the variation of N_{60} due to the changing effective overburden pressure (σ_o') with depth. Das provides several methods for calculating C_N, the most cited of those being the formulae developed by Skempton and Liao and Whitman (2004).

Table 12: Correction Factor for Field Obtained SPT values (after Das, 2004)

Author's Relationship	SPT Correction Factor
Peck et al. (1974)	$C_N = 0.77 log \left[\frac{20}{\left(\frac{\sigma_o'}{P_a}\right)}\right]$
Seed et al. (1975)	$C_N = 1 - 1.25 log \left(\frac{\sigma_o'}{p_a}\right)$
Skempton (1986)	$C_N = \frac{2}{1 + \left(\frac{\sigma_o'}{p_a}\right)}$
Liao and Whitman (1986)	$C_N = \sqrt{\frac{p_a}{\sigma_o'}}$

Where σ_o' is the vertical effective stress at the elevation being considered (kN/m^2 or lb/ft^2) and p_a is atmospheric pressure (100 kN/m^2 or 2 000 lb/ft^2).

The pile capacity due to frictional resistance then becomes:

$$Q_s = \sum p\Delta L f_{av} \tag{19}$$

where: Q_s is the skin friction developed (kN or lb), p is the perimeter of the pile (metre or feet), ΔL is the incremental length over which the unit frictional resistance is constant (metre or feet), and f_{av} is the unit frictional resistance over which the corrected SPT count $(\overline{N_1})_{60}$ is constant (kN/m^2 or lb/ft^2).

4.1.1.4 CPT Method

The Laboratoire Central des Ponts at Chausseées (LCPC) and Dutch methods are the two most commonly used techniques to estimate pile end bearing capacities through CPT results (Das, 2004).

The LCPC method involves calculating the average cone penetration resistance ($q_{c(av)}$) value by taking the mean cone resistance values from 1.5 pile widths above the pile tip to 1.5 pile widths below the tip. Then calculating the equivalent cone resistance ($q_{c(eq)}$) by eliminating the values that are greater than 1.3 times $q_{c(av)}$ and less than 0.7 times $q_{c(av)}$, and recalculating the mean of the remaining values, as seen in Figure 20.

The unit point resistance is then calculated with the following equation:

$$q_p = q_{c(eq)} k_b \tag{20}$$

where: q_p is the unit point resistance (kN/m^2 or lb/ft^2), $q_{c(eq)}$ is the equivalent average cone penetration resistance value (kN/m^2 or lb/ft^2), and k_b is an empirical bearing capacity factor. Briaud and Miran suggest a k_b value of 0.375 for sands and gravels (Das, 2004).

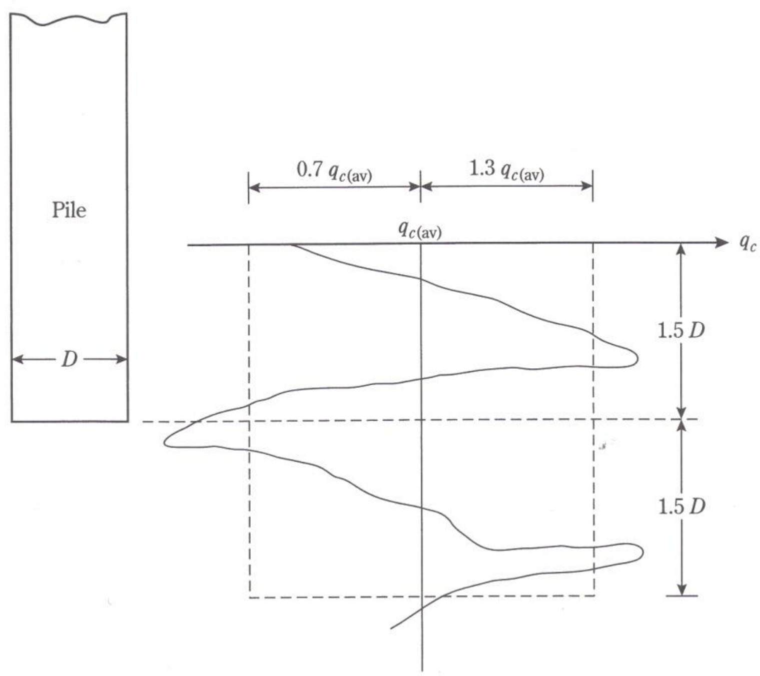


Figure 20: LCPC method (Das, 2004)

The Dutch method is similar to the LCPC method but more complicated. The first step involves summing the cone resistance values (q_c) along the actual CPT path from the elevation of the pile tip down to the minimum CPT value recorded between 0.7 and 4 pile diameters below the pile tip. The second step is then to sum the qc values from the straight line path back up to the pile tip, see Figure 21. With these paths determined, q_{c1} is calculated, which is the average cone resistance value within the outlined area.

The next step is to calculate q_{c2}, which is the average q_c value from the pile tip to 8 pile diameters above the pile tip, while ignoring any minor depressions. With q_{c1} and q_{c2} known the unit point resistance is calculated as follows:

$$q_p = \frac{(q_{c1}+q_{c2})}{2} k_b' \leq 150p_a \tag{21}$$

where: q_p is the unit point resistance (kN/m^2 or lb/ft^2), q_{c1} and q_{c2} are the average cone penetration resistance value (kN/m^2 or lb/ft^2) below and above the pile tip, respectively, p_a is atmospheric pressure (100 kN/m^2 or 2 000 lb/ft^2), and k_b' is an empirical bearing capacity factor. DeRuiter and Beringen, 1979, recommended a k_b' value of 1 for normally consolidated sands and 0.67 for sands with an overconsolidation ratio between two and four (Das, 2004). The overconsolidation ratio is a measure of the maximum effective normal stress (preconsolidation pressure) that the soil profile has experienced in the past compared to the current vertical stress, at any elevation under investigation.

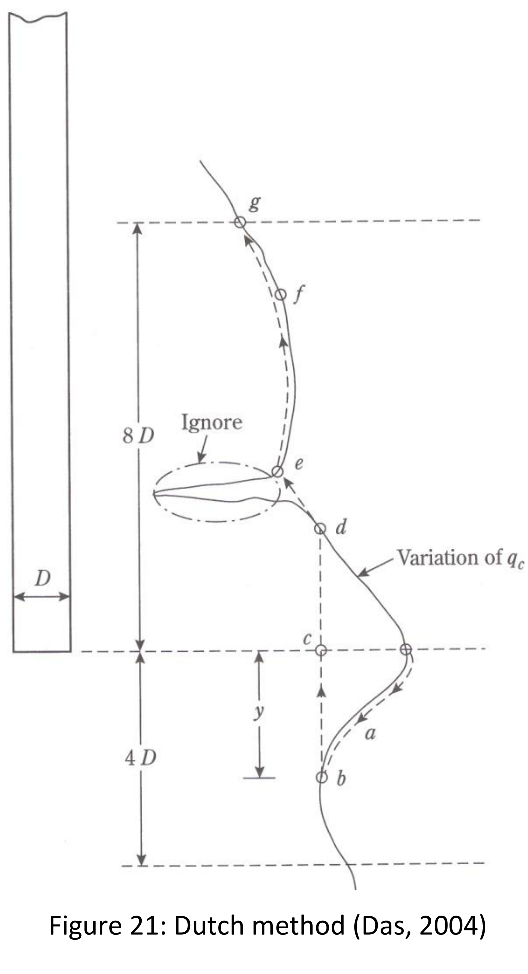


Figure 21: Dutch method (Das, 2004)

Relationships to correlate the frictional resistance of piles with CPT results which are developed by Nottingham and Schmertmann (1975), and Schmertmann (1978) are given in the formula below (Das, 2004):

$$f = \alpha' f_c \tag{22}$$

where: f is the unit frictional resistance (kN/m^2 or lb/ft^2), α' is a skin friction coefficient of proportionality, and f_c is the cone frictional resistance from CPT data (kN/m^2 or lb/ft^2). The value of α' depends upon the embedded pile length (z) to width (D) ratio and the type of cone penetrometer; electric or mechanical, as shown in the figures below.

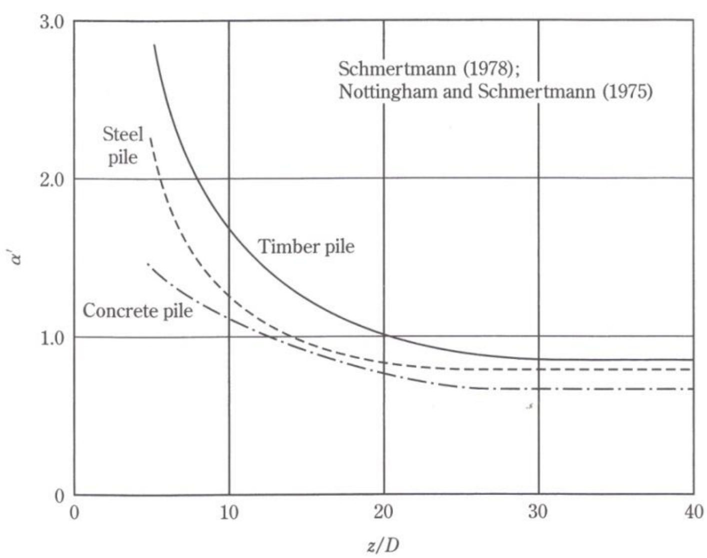


Figure 22: Embedded Pile Length to Width Ratio vs α' for Electric Cone Penetrometer (Das, 2004)

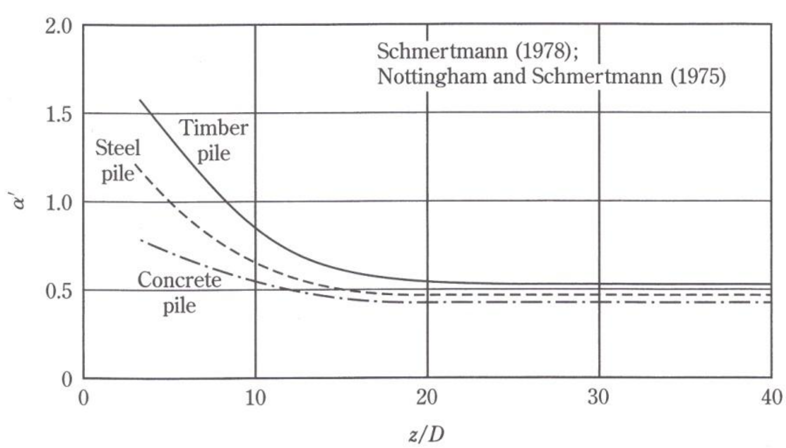


Figure 23: Embedded Pile Length to Width vs α' for Mechanical Cone Penetrometer (Das, 2004)

4.1.2 Piles In Cohesive Soils

For cohesive soils such as clays and silts pile bearing capacity is primarily developed through skin friction. However, there may be limited tip or end bearing capacity developed as well, depending upon installation method and pile type.

End bearing capacity is derived from the same formula in section 4.1:

$$Q_p = A_p\left(c'N_c^* + q'N_q^*\right) \tag{5}$$

4.1.2.1 Meyerhof Method

The effective angle of internal friction (φ ') in cohesive soils is taken as being equal to zero. Meyerhof suggests that since N_q^* is dependent on the soils friction angle, it is neglected and N_c^* is set equal to nine, thus the formula is reduced to:

$$Q_p = 9c_u A_p \tag{23}$$

where: Q_p is the unit bearing resistance (kN or lb), c_u is the undrained shear strength of the soil (kN/m^2 or lb/ft^2), and A_p is the pile area (m^2 or ft^2). For shear strengths determined by laboratory testing a reduction factor of 0.75 is advised (Craig, 2002).

The undrained shear strength is determined from triaxial tests or in situ field vane shear tests. Equations of undrained shear strength are also correlated to other parameters such as vertical effective stress (σ_o'), plasticity index (PI), and preconsolidation pressure (σ_c') as shown in Das, 2004.

Table 13: Empirical Correlations of Undrained Shear Strength (after Das, 2004)

Author	Relationship	Applicability
Skempton (1957)	$c_{u(vst)} = (0.11 + 0.0037PI)\sigma_o'$	$c_{u(vst)}$ – c_u from shear vane test For normally consolidated clays
Jamiolkowski et al. (1985)	$c_u = (0.23 \pm 0.04)\sigma_c'$	For lightly overconsolidated clays
Chandler (1988)	$c_{u(vst)} = (0.11 + 0.0037PI)\sigma_c'$	For over and normally consolidated clays, not fissured or sensitive clays Accuracy ± 25%
Mesri (1989)	$c_u = 0.22\sigma_o'$	

To determine the developed frictional resistance of piles in clay, four basic methods are used to determine the piles bearing capacity; the alpha (α), beta (β), lambda (λ), and CPT method.

4.1.2.2 Alpha Method

The alpha or total stress method involves determining the skin friction acting on a pile due to the undrained shear strength of clay and an adhesion factor, which is a function of the ratio of undrained shear strength to effective overburden pressure (see Figure 24, below).

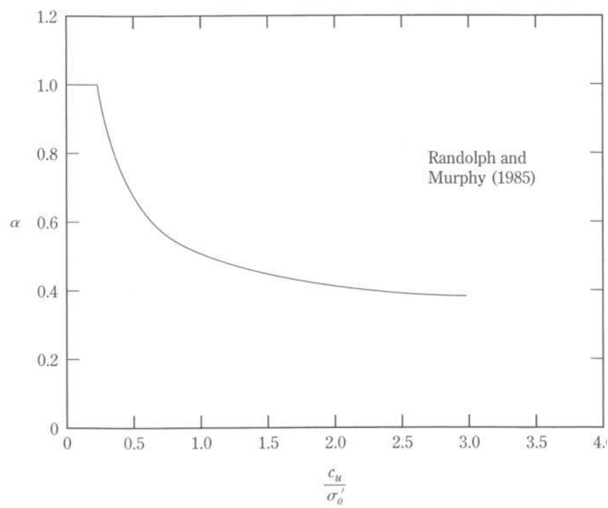


Figure 24: Adhesion Factor for Determining Frictional Capacity (Das, 2004)

The unit skin friction is given as:

$$f = \alpha c_u \tag{24}$$

where: f (kN/m^2 or lb/ft^2) is the unit skin friction, α is the adhesion factor, and c_u is the undrained shear strength (kN/m^2 or lb/ft^2). In general the value of α is between 0.3 and 1 (Craig, 2002).

4.1.2.3 Beta Method

The beta or effective stress method is used to account for the temporary creation and subsequent dissipation of excessive pore water pressure in clays. The frictional resistance is by Equation 25:

$$f = \beta \sigma_o' \quad (25)$$

where: f (kN/m^2 or lb/ft^2) is the unit skin friction, σ_o' is the vertical effective overburden pressure (kN/m^2 or lb/ft^2), and β is a function of the earth pressure coefficient and clay friction angle (see Equation 26 below). The average value of β is usually between 0.25 and 0.40 for normally consolidated clays and significantly higher with a wider range for overconsolidated clays (Craig, 2002).

$$\beta = K \tan \varphi_R' \quad (26)$$

$$K = 1 - \sin \varphi_R' \quad (27)$$

$$K = (1 - \sin \varphi_R')\sqrt{OCR} \quad (28)$$

where: K is the at rest earth pressure coefficient for normally and overconsolidated clays, respectively, φ_R' is the drained friction angle of remoulded clay (degrees) and OCR is the overconsolidation ratio of clay.

4.1.2.4 Lambda Method

The lambda method is based on the supposition that the installed pile induces passive lateral pressures in the soil during driving and that the frictional resistance developed is described by Equation 29 below:

$$f_{av} = \lambda\left(\overline{\sigma_o'} + 2c_u\right) \quad (29)$$

where: f_{av} is the unit frictional resistance of the pile (kN/m^2 or lb/ft^2), $\overline{\sigma_o'}$ is the mean effective vertical strength for the entire pile embedment length (kN/m^2 or lb/ft^2), c_u is the undrained shear strength of clay (kN/m^2 or lb/ft^2), and λ is a coefficient which varies with pile length as shown below:

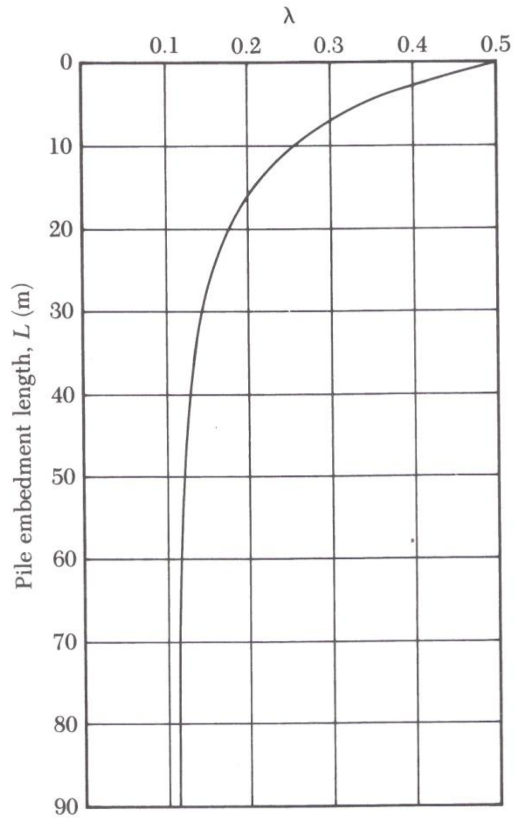


Figure 25: Variation of Lambda with Pile Embedment Length (Das, 2004)

The equation to calculate the frictional capacity is then modified to:

$$Q_s = pLf_{av} \tag{30}$$

where: Q_s is frictional resistance of the pile (kN or lb), p is the pile perimeter (metres of feet), L is the total embedded pile length (metres or feet), and f_{av} is the average unit frictional resistance caused by pile driving (kN/m^2 or lb/ft^2).

4.1.2.5 CPT Method

The LCPC method to determine the end bearing capacity of piles is cohesive soils is similar to the end bearing capacity of piles in cohesionless soils. The equation is the same as in section 4.1.1.3; however the empirical bearing capacity factor (k_b) changes from 0.375 for sands and gravels to 0.6 for silts and clays.

To calculate the end bearing capacity using the Dutch method, the unit point is resistance is given as:

$$q_p = R_1 R_2 \frac{(q_{c1}+q_{c2})}{2} k_b' \leq 150 p_a \tag{31}$$

where: q_p is the unit point resistance (kN/m^2 or lb/ft^2), p_a is atmospheric pressure (100 kN/m^2 or 2 000 lb/ft^2), q_{c1}, q_{c2}, and k_b' are calculated in the same methods as described in section 4.1.1.3, R_2 is set to a value of 1 when an electric cone penetrometer is used and 0.6 when a mechanical cone penetrometer used, and R_1 is a function of the undrained shear strength of the soil and atmospheric pressure as given in the table below:

Table 14: R_1 Values for Cohesive Soils Used With the Dutch Method (Das, 2004)

$\frac{c_u}{p_a}$	R_1
≤ 0.50	1.00
0.75	0.64
1.00	0.53
1.25	0.42
1.50	0.36
1.75	0.33
2.00	0.30

The method to calculate the unit frictional resistance of piles in clay using CPT results is the same for piles in sand, however the α' coefficient is a function of the cone penetration resistance and atmospheric pressure rather than pile embedded depth and width, as shown below:

The frictional resistance is given by the same formula as in section 4.1.1.3:

$$f = \alpha' f_c \tag{19}$$

where: f is the unit frictional resistance (kN/m^2 or lb/ft^2), α' is the CPT skin friction coefficient, and f_c is the cone frictional resistance (kN/m^2 or lb/ft^2).

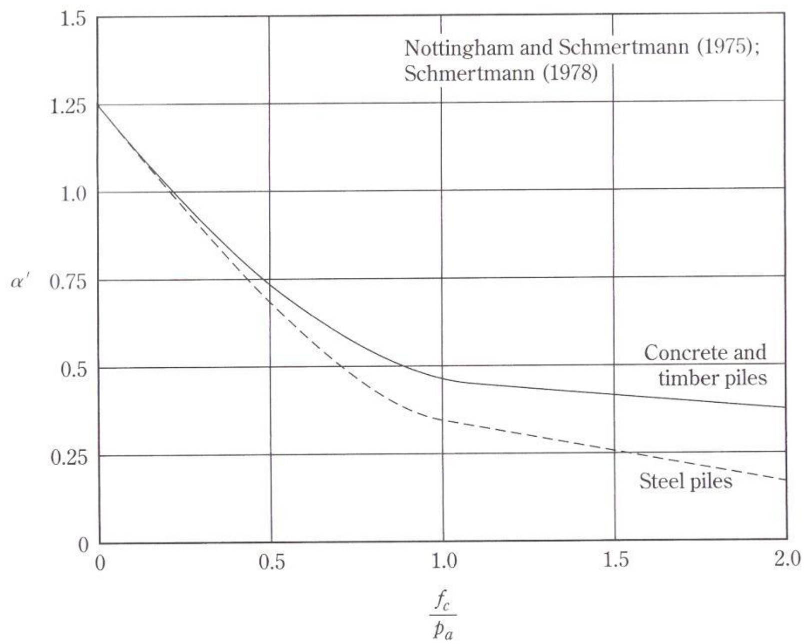


Figure 26: CPT Coefficient Versus Cone Resistance and Atmospheric Pressure (Das, 2004)

4.1.3 Piles In Rock

A method to determine the bearing capacity of piles driven to and into rocks is given by Goodman, 1980 (Das, 2004). The ultimate bearing capacity is:

$$Q_{p(ult)} = \left[q_{u(design)}\left(N_{\varphi} + 1\right)\right]A_p \tag{32}$$

where: $Q_{p(ult)}$ is the ultimate end bearing capacity of the pile on rock (kN or lb), $q_{u(design)}$ is the designed unconfined compressive strength (UCS) of the rock (kN/m^2 or lb/ft^2), A_p is the area of the pile (m^2 or ft^2), and N_{φ} is a function of the friction angle of the rock (as shown below).

$$N_{\varphi} = \tan^2\left(45 + \frac{\varphi'}{2}\right) \tag{33}$$

where: φ′ is the effective friction angle of the rock mass (degrees). Typical friction angles according to rock type are displayed in Table 15.

Table 15: Effective Internal Angle Classified by Rock Type (Das, 2004)

Rock Type	Friction Angle, φ′ (degrees)
Sandstone	27 – 45
Limestone	30 – 40
Shale	10 – 20
Granite	40 – 50
Marble	25 – 30

The design UCS of rock ($q_{u(design)}$) is typically one fifth of the UCS values determined from cores during laboratory testing. A factor of safety is used to account for scaling effects; tested cores are often intact whereas the rock mass itself frequently contains many discontinuities, such as joints, fractures, fissures, seams, and faults which decrease the strength of the stratum and may be missed during sampling. Laboratory determined UCS of rocks is given in Table 5 in section 2.2. The allowable end bearing capacity of the pile ($Q_{p(all)}$) is customarily taken as one third of the calculated ultimate end bearing capacity ($Q_{p(ult)}$). A factor of safety of three is used to account for unseen qualities in the rock, the uncertainty of the length of pile is driven into the rock, if the pile end is damaged due to driving or not and the extent of the damage, the fact that the pile may not develop full end bearing capacity since the settlement of the pile is likely less than the 10 to 25% of the pile width required, and the possibility that the end of the pile does not make full contact with the rock layer, either due to pile damage or rock and soil debris at the bottom of the socket in the rock layer.

The limiting factors for estimating the bearing capacity of piles on the basis of theoretical formulae is that in general these methods are based upon test results which are carried out in-situ or in a laboratory where the samples are as undisturbed as possible, thus they represent the site conditions prior to pile driving and the method of installation is only taken into account for some formulae and not others. However, during driving the sub surface conditions change; i.e. in loose cohesionless soil the internal angle of friction increases, in very dense cohesionless soils the density decreases, in cohesive soils the remoulded zone loses strength and large pore water pressures are generated, and the soil beneath and around the pile increases in density. As such the predictions based on static formulae are inherently erroneous from the actual soil conditions and pile capacity.

To predict the created pile bearing capacity with more accuracy, other methods are developed which take into account changes in soil properties as well as the method of installation. Some of these methods include pre-construction analysis such as computer modeling, analysis during driving such as by dynamic formulae, and post driving analysis which physically loads the pile until failure to determine the true ultimate pile bearing capacity.

From the 77 piles designated for comparisons of predicted to measured bearing capacities it is determined that none could be analysed for all of the theoretical formulae described above. The borehole records from the MTO publication are examined and none of the soil layers from any boreholes have the internal angles of friction or relative densities recorded. Ten do not contain the ground water elevations, only 17 have unit weights for all soil layers given, 53 piles have SPT counts recorded, but only nine also have unit weights for all soil layers given. Only 8 pile sites have CPT data recorded and only 19 have shear strength values for each soil layer recorded.

For a complete list of theoretical static formulae including applicable soil types, general rules of thumb for parameters such as active earth coefficients, relative densities, a, b, β coefficients, friction angles, and digital borehole records from the MTO publication see Appendix A.

4.2 Dynamic Predictive Formula

Dynamic formulae are one of the oldest and most controversial methods used to predict pile capacity. They are formed on the basis of the principals of conservation of energy and momentum. They have the advantage of measuring conditions during driving and as such there is no need to estimate or assume soil properties at one location from information at another. Another possible advantage is that they are simple and straight forward, relating the ultimate pile capacity to common field measurements such as pile driver properties and blow counts and are independent of soil types and properties, thus similar blow counts in any soil should allow the piles to support similar ultimate loads.

The disadvantages of dynamic formulae are that they are used to predict static pile bearing capacities of soils through observations from when the pile is advancing and the resulting soil reaction; but it is well known that soils behave very differently under dynamic and static conditions. This effect may explain the extreme variability in results when comparing predicted capacities to pile load tests which measure the actual ultimate load that a pile can support. Historically dynamic formulae under and over predict pile capacity by as much as a half to 30 times, respectively. Another failing is that dynamic formulae in no way convey the amount or rate of settlement that a pile may undergo after loading. Since most dynamic formulae do not take pile composition into account, there is no method which stipulates how much of the developed capacity is due to frictional resistance and how much is due to end bearing resistance when they are used alone.

Dynamic formulae are essentially energy balance equations where the energy generated by the hammer is equal to the energy imparted on the pile with the exception of any losses. The losses may be due to friction created by the ram weight during its fall, compression of the hammer weight, the pile, the pile cushion, and the soil during striking. Other areas of energy loss are due to hammers not performing at peak efficiencies, formulae which do not consider freeze effects, piles flexing during striking, and possible differences in pile driver properties used to calculate predictive capacities and those used during subsequent pile driving operations (Coduto, 2001).

There are several assumptions made by basing the pile bearing capacity on dynamic methods, such as the behaviour of a pile during static loading is the same as its behaviour during dynamic loading and that the pile acts as a rigid rod which experiences an instantaneous compressive wave as the pile is struck. This assumes that when the pile is struck it experiences a compressional wave over its entire length and moves downward as a single unit; however, in reality only sections of a pile experience compressional forces at a time and if the pile is terminated in a very hard or dense stratum, the reflected stress wave may cause part of the pile to undergo tensile forces. Implicit in these formulae is the assumption that the ram weight used to strike the pile is significantly heavier than the pile itself. Once the ram weights are similar to the pile weights, as may occur in the case of long piles or pile made from heavy material such as concrete or concrete filled steel tubes then the assumption is no longer valid and the dynamic predictive formulae become less accurate.

Dynamic formulae are used in two ways; to determine the specifications required during driving such as minimum hammer energy, blow counts, energy loses permitted and pile lengths or as a field aid in determining when the required pile capacity is achieved and when to terminate pile driving. This is done by inputting the required capacity, rated hammer energy and back calculating the number of blow counts required to create this capacity. This blow count is then used in the field as a guide to determine when a pile is driven to a satisfactory depth or bearing capacity. For greater confidence, the usual practice is to obtain a specified number of hammer blows per feet and repeat this for two to three feet sequentially.

Although hundreds of equations exist for estimating pile capacity through dynamic means, the four most commonly used ones in North America are the Engineering News Record (ENR), Gates, FHWA

Modified Gates, and Hiley formulae. The MTO uses a customized version of the Hiley formula (hereafter referred to as the MTO modified Hiley formula), which is also used to analyse pile capacity from driving records and compared to the others as well as results from pile load tests.

Dynamic formulae are developed according to results observed for driven piles only, thus piles installed by other means (boring, excavating, augering, vibratory, etc.) are not used during the analysis.

4.2.1 ENR Formula

The Engineering News Record or ENR formula is the oldest of the five studied, developed in 1888 by A. M. Wellington (Coduto, 2001) for timber piles installed by drop hammer (Fragaszy et al., 1985) and named after the civil engineering journal in which it first appeared which is given below as:

$$R = \frac{2E_n}{s+c} \tag{34}$$

where: R is the allowable pile capacity (lb) and E_n is the hammer energy (ft·lb). For drop hammers the energy is calculated as the product of the ram weight (lb) and the fall height (ft). For all other hammers, the energy is the rated energy of the pile driver and s is the final set of the pile (inches). The final set is defined as the amount of settlement the pile undergoes per one hammer blow at the end of driving, though it is typically averaged over the last 20 blows. The coefficient c represents the energy loss during the driving process by the means described above, in inches. From a unit check the numerator is in terms of feet and the denominator in inches, taking into account the 2 the units are still out of agreement by a factor of 6. This difference is taken as the safety factor to compensate for the heterogeneity within the subsurface, the variability in the energy developed, and measurements of pile set.

The simplest of the formulae examined, it assumes that the energy created by the hammer is equal to the energy imparted on the pile minus any losses, given by the parameter c. The loss term is to account for the inefficiencies during the driving process. The amount of energy lost, given in inches is set equal to 1; later the c coefficient is modified to 0.1 inches for all other pile drivers including diesel and hydraulic hammers.

As illustrated in Section 3, as hammer technology improved, the force striking the pile as well as the blow rates generally increased; however, ram weights and fall heights decreased, thus some of the assumptions mentioned above lose validity and as a result the equations inherently become less accurate. Published findings consistently show that the ENR formula is under conservative, probably for this very reason.

One of the objectives of this study is to determine if the loss (c) value should be modified to better reflect energy losses for modern hammers which are commonly used during pile driving.

4.2.2 Gates Formula

The Gates formula (Equation 35) named after Marvin Gates and developed in 1957 is based on his results from comparing the results from 130 pile load tests to the hammer energy used and the set developed for each pile driven (Gates, 1957). The piles examined are composed of steel, timber, precast concrete, thin shell cast in place concrete, pipe, and one composite pile. The predictive formula is given below as:

$$R = \frac{1}{7}\sqrt{E_n}(1 - \log s) \tag{35}$$

where: R is the allowable bearing capacity of the pile (tons), E_n is the developed hammer energy calculated in the same manner as in the ENR formula (ft·lb), and s is the final pile set (inches). The $^1/_7$ parameter is derived from solving for the constants of inequality which relate the driving resistance to the hammer energy and pile set as well as applying a factor of safety of three.

Unlike other formulae, the Gates formula uses the square of the developed energy based on Redenbacher's analysis. Mathematically, the maximum set is most 10 inches; larger sets cause the value of R to become negative; which does not make physical sense. A pile advancing 10 inches per blow indicates very easy driving. Although rare, in the 77 piles examined for this thesis one such case occurred. The predicted pile capacities are correspondingly low; however, pile load tests indicate a safe bearing capacity of 89 kN or approximately 10 tons.

4.2.3 FHWA Modified Gates Formula

Many modifications are developed using the original Gates formula as a basis, to better predict pile capacity by taking into account novel technologies such as longer, heavier piles and innovative driving equipment. To account for these changes in driving techniques, governmental groups as well as private companies altered the Gates formula to better fit their own experiences during the construction processes.

The Federal Highway Administration (FHWA) suggests using the following formula since 1997, first developed by Hannigan et al. (Allen, 2005):

$$R = 1.75\sqrt{E_n}\log(10N) - 100 \tag{36}$$

where: R is the allowable bearing capacity (kip), En is the energy of the hammer calculates in the same manner as in the ENR formula (ft·lb), N is the inverse of the pile set (blows/in).

The FHWA modified Gates formula also contains a built in factor of safety of three. The modified formula also takes the efficiency of the pile driver used into account. For drop hammers the E_n term is multiplied by 0.75 and for all others it is multiplied by 0.85; this is meant to account for all the loses in energy transferred from the driver to the pile. The 1.75 coefficient at the beginning of the formula is an empirical factor to aid in giving the resulting capacity a closer estimate to field observations of true pile capacity as well as a fudge factor to give the resulting answer the correct units.

There is a mathematical limitation on the logarithmic term. In order to keep the calculated capacity greater than zero, the N term must be greater than 0.1 or the calculated value does not make physical sense. Purely to aid in better correlating the predicted pile capacity to the measured pile capacity, a term of 100 is added to the end of the equation. This also makes no physical sense, since it implies that zero energy imparted onto a pile or that a pile that is not driven at all is able to support a load of negative 100 kip.

4.2.4 Hiley Formula

The Hiley formula is often described as the most elegant or comprehensive dynamic formula developed. This is due to the fact that it takes into account the pile driver efficiency, the weight of the pile, the length of the pile, and quantifies the loss of transferred energy due to the compression of the pile cap, pile, and soil explicitly.

One of the most common versions of Hiley formula used is presented in Chellis (1961) for drop and single acting steam hammers as:

$$R_u = \frac{e_f W_r h}{s + {}^1\!/_2 (C_1 + C_2 + C_3)} \times \frac{W_r + e^2 W_p}{W_r + W_p} \tag{37}$$

and for double acting, differential acting, and diesel hammers as:

$$R_u = \frac{12 e_f E_n}{s + {}^1\!/_2 (C_1 + C_2 + C_3)} \times \frac{W_r + e^2 W_p}{W_r + W_p} \tag{38}$$

where: R_u is the ultimate bearing capacity of the pile (lb), e_f is the efficiency factor of the pile driver, which ranges from 65 to 100 percent, depending upon hammer type and manufacturer. During this study, the efficiency factor is taken as 75% for drop hammers and 100% for diesel hammers. W_r is the ram weight (lb), h is the fall height (in), E_n is the rated hammer energy (ft·lb), s is the final set of the pile (in), W_p is the pile weight including any soil attached to the outside, inside or within the flanges of the pile (lb), and e is the coefficient of restitution of the pile, which ranges from 0 to 0.8 depending upon cap material, physical condition and pile material. During this study the coefficient of restitution is taken as 0.5. C_1, C_2, and C_3 are the amount of compression of the pile cap and head, pile, and soil, respectively (in). Compression values from Chellis (1961) are given below as Tables 16 to 18.

Table 16: Temporary Compression Values of C_1 - Pile Cap and Head (after Chellis, 1961)

Material to Which Blow is Applied	Easy Driving	Medium Driving	Hard Driving	Very Hard Driving
Head of Timber Pile no Cap	0.05	0.10	0.15	0.20
Cap on Concrete Pile	0.12	0.25	0.37	0.50
Steel Cap on Steel Pile	0.04	0.08	0.12	0.16
Steel Pile no Cap	0.00	0.00	0.00	0.00

Table 17: Temporary Compression Values of C_2 - Pile (after Chellis, 1961)

Pile Type	Easy Driving	Medium Driving	Hard Driving	Very Hard Driving
Timber	0.004 x L	0.008 x L	0.012 x L	0.016 x L
Precast Concrete	0.002 x L	0.004 x L	0.006 x L	0.008 x L
Steel	0.003 x L	0.006 x L	0.009 x L	0.012 x L

Table 18: Temporary Compression Values of C_3 – Soil (after Chellis, 1961)

	Easy Driving	Medium Driving	Hard Driving	Very Hard Driving
Pile of Constant Cross Section	0 – 0.10	0.10	0.10	0.10

The Hiley formula assumes that the pile acts as a rigid rod and that the compression it undergoes during striking is instantaneous throughout the entire pile length. This assumption is incorrect and causes predicted pile capacities to be much lower than they actually are for relatively long or heavy piles. Some authors suggest calculating the predicted capacity with half the pile length for hard driving conditions or using an equivalent pile length based on the time between hammer blows.

4.2.5 MTO Modified Hiley Formula

MTO engineers (Tse, 2010) use the following modified version of the Hiley Formula for drop and single acting steam hammers:

$$R_u = \frac{n e_f W_r g H}{s + C/2} \tag{39}$$

and for diesel, double acting, and differential acting steam hammers:

$$R_u = \frac{n e_f E_n}{s + C/2} \tag{40}$$

where: R_u is the ultimate pile resistance (kN), s is the measured pile penetration per hammer blow (mm), C is the measured rebound of the pile per hammer blow (mm), E_n is the rated hammer energy (Joules), W_r is the mass of the pile ram or piston (kg), H is the free fall height of the mass (metres), g is the gravitational constant (9.81 m/s^2), e_f is the hammer efficiency based on the manufacturers' gross rated energy. For drop hammers e_f is set to 0.75, for steam hammers e_f is taken from 0.6 to 0.8, and for diesel hammers e_f is set to 1.0, and *n* is the efficiency of the hammer blow, given as:

$$n = \frac{W_r + W_p e^2}{W_r + W_p} \tag{41}$$

where: W_r is the same as defined above, W_p is the mass of the pile and anvil or helmet (kg), and e is the coefficient of restitution set to 0.25 for timber piles driven with a pile cushion, 0.32 for steel piles driven with a cushion, and 0.55 for steel piles driven without a cushion.

Substituting equation 38 into 36 or 37 gives very similar equations to that of the original Hiley formulae; with the exception of the units, the set values, the coefficient of restitution (e), and the compression values (C). In the original Hiley Formulae C_1, C_2, and C_3 are assumed from published works; however, in the MTO modified Hiley formulae C is measured from the individual pile being driven. Since the MTO modified Hiley formulae uses site data, it is assumed to be more accurate than the earlier Hiley formulae, nevertheless the results may still be similar depending on the sensitivity of the parameters e and C and the accuracy of the published data compared to real world observations.

Since the MTO modified Hiley Formula is based on the same assumptions as the original it experiences the same limitations as the original.

4.3 Computer Modeling

The concept of relating developed pile capacity to measured site observations is appealing since it is intuitive, straight forward, and simple. Unfortunately, semi empirical dynamic formulae historically result in large variations and often skewed predictions of pile capacity so computer models are created to better correlate pile capacity with hammer energies, impact forces, and blow counts taking into account soil and pile properties.

As mentioned in Section 4.2, pile prediction methods assumed that piles acts as a rigid body when struck by a hammer; however, this assumption is false and in reality the pile acts as a slender rod when experiencing stress waves from hammer impacts. In 1931, Isaacs suggested analyzing piles based on propagating stress waves. He described pile movement by the following one dimensional double differential equation (Coduto, 2001):

$$\frac{\partial^2 u}{\partial t^2} = \frac{E}{\rho} \frac{\partial^2 u}{\partial z^2} \tag{42}$$

where: z is the depth of the pile section being analyzed (metres or feet), u is the displacement of the pile at the depth z (metres of feet), t is time (seconds), E is the modulus of elasticity of the pile (kN/m^2 or lb/in^2), and ρ is the density of the pile (kg/m^3 or lb/ft^3).

When considering the boundary conditions of pile – soil system, the solution to the differential equation becomes non trivial and numerical methods are used to approximate the solution. In 1951 E.A.L. Smith began experimenting with computer programs and by 1960 developed the first numerical model used to predict pile bearing capacity via wave analysis (Coduto, 2001). He simulated the soil – pile system as a series of weightless springs and dashpots as shown below.

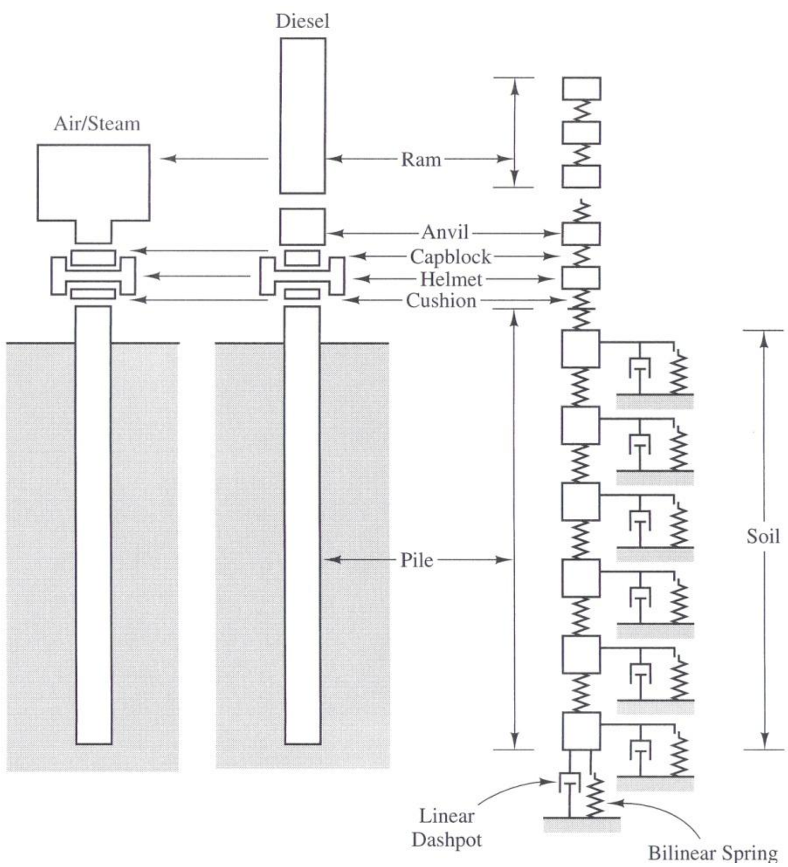


Figure 27: Smith's Numerical Model of pile soil system (Coduto, 2001)

Each element of the system represents the properties of the pile or soil at that location in reality thus the stiffness of the spring corresponds to the stiffness of the pile and soil at the same depth being analysed. The springs and dashpots on the side of the pile represent the frictional resistance of the soil whereas the springs and dashpots on the bottom of the pile represent the point or end bearing resistance of the pile.

The springs are modeled as bilinear components which behave elastically until they reach their ultimate resistance (R_u) at a displacement *q* (or quake) at which point they are assumed to act completely plastically. The dashpots are set to behave linearly and react to the velocity of the pile being driven whose resistance is given by the Smith damping factor J_s (s/m or s/ft) (Coduto, 2001).

The factors R_u, *q*, and J_s are calibrated to actual site values determined from comparisons to pile load tests and used as a site specific tool to estimate bearing capacities of future piles driven at the same site assuming the same pile driving equipment and same pile types are used. The wave analysis is used to produce multiple charts relating various blow counts to ultimate bearing pile capacities. These charts are then used on site to determine which blow count is necessary to obtain the desired pile capacity or, inversely, to indicate the current pile capacity developed at the current blow count and conclude if further driving is required or not. The wave analysis also has the advantage of calculating the stresses acting on the pile and determines if the pile is able to withstand the driving forces necessary for particular sites or applications.

Examples of commercial wave analysis programs are TTI developed at Texas A&M University and WEAP (Wave Equation Analysis Program) developed at the Case Western Reserve University, both developed in 1976. WEAP has been updated several times; in 1981, as CUWEAP in 1983 and completely rewritten in 1986 (Goble and Rausche, 1986). In 1988 by GRL Engineers Inc. developed GRLWEAP which is still in use today with the most recent update in 2010. As programs evolve they are updated to include various pile driver properties such as different hammer models, energy, and velocity imparted to the pile and ram, cushion, cap and equipment properties such as weights, conditions, sizes, and elastic modulus. Many other finite element models are developed specifically for civil engineering and piling purposes such as midasGTS, FOXTA, DEFPIG, PIES, SCARP, ZSOIL, Plaxis 3D Foundation, IMAGINE, AFENA, and AllPile.

The newest computer programs which are used for pile capacity analysis are specially written finite difference geotechnical code such as Fast Lagrangian Analysis of Continua in two or three dimensions (FLAC and FLAC3D) and Universal Distinct Element Code in two or three dimensions (UDEC and 3DEC), both created by Itasca International Inc. first released in 1986 and 1988 which uses various soil models such as Mohr Coulomb, Cam Clay, Hoek Brown, Drucker Prager, elastic, and bilinear plastic to simulate the reaction to various forces acting in the subsurface using soil parameters such as internal angle of friction, shear strength, cohesion, water table level, soil unit weight, shear modulus, bulk modulus, and Young’s modulus for different soil layers and structures.

The limitations of using computer models are similar to those of static theoretical formulae in that the basis for model meshes are most often from boreholes which are not always at the exact location of the pile being installed. Typically few boreholes are used to describe a relatively large area and as such the computer model is a simplified version of reality; additionally some subsurface features such as thin clay seams, vertical faults, fissures etc. may be missed by drilling samples. As well geologic conditions can change relatively abruptly from one geographic location to the other, thus the accuracy is limited from the beginning of the process. As with all techniques the accuracy of the prediction lays with the precision of the input parameters therefore the utmost care is required when measuring soil properties in the field or laboratory.

Next specific programs are designed which record observations from hammer strikes during driving and are used to calculate pile capacity. Since these are methods used during driving they are discussed further in the following subsection.

Unfortunately for this thesis, vital parameters of soil properties such as soil stiffness, damping factors, internal angles of friction, cohesion, shear and young's moduli, unit weights, and occasionally groundwater conditions and shear strength are not recorded for pile driving sites thus computer modelling results would yield limited benefits into assessing its accuracy as a tool to predict pile load capacity.

4.4 Signal Matching Programs

Developed relatively recently, signal matching is a high strain dynamic curve fitting predictive method which uses the measured force and acceleration acting on the pile during driving to predict pile bearing capacity by a correlation factor J_c (Coduto, 2001). This correlation factor is determined from comparing the measured values to static load tests at the site of driving.

The earliest methodology developed at Case Western Reserve University from the 1960s to the early 1970s is called CASE. The CASE method involves attaching strain gauges and accelerometers to the upper most portion of a pile and recording, via a pile driver analyzer (PDA), strain and acceleration data which the pile undergoes when struck. This data is transmitted to a personal computer in real time and the information is used to calculate forces acting on the pile, displacements achieved, and is then used to infer hammer efficiencies, stresses induced within the pile, pile integrity, and developed pile bearing capacity.

In 1972, Case Western Reserve University also developed the Case Pile Wave Analysis Program or CAPWAP. This methodology combines the numerical modeling of wave equation analysis programs such as WEAP or GRLWEAP with the onsite wave trace data from PDAs used in CASE type analysis. The basis of this approach is to compensate for the weaknesses of each method, namely that the WEAP method uses assumptions about the hammer energies and efficiencies achieved when driving piles, and the CASE method correlates the compressional force and velocity data to static load tests by the factor j_c to predict pile capacity, which due to the variability of subsurface properties and soil stratigraphy may change from one pile to another even on the same site. CAPWAP takes the data measured during striking and uses that as the source for numerical modeling which is based on detailed soil properties measured in situ or from laboratory testing rather than empirical derived comparative values. The result is much more accurate predictions of pile bearing capacities.

Research shows that wave analysis and wave trace programs drastically improves the predictive capacity of a pile compared to theoretical formulae, dynamic formulae, and computer modeling alone. The limitations of computer modeling programs are the assumptions that the programs are based upon as well as the simplification of the geology in which the pile is being driven. Wave trace programs suffer in the sense that it is not economical to instrument and record PDA measurements for every pile advanced on a site, thus only a select few piles can benefit from this analysis while the remainder may over or under predict pile capacities, the extent of which is determined by the estimates, assumptions, and uncertainties in pile driving equipment and subsurface conditions.

Unfortunately for the analysis performed on the chosen sites, information required to execute wave trace programs such as driving measurements of force, velocity, and strain acceleration is not available in the pile records studied thus no comparison to dynamic formulae are established.

4.5 Pile Load Tests

Often considered the true measure of pile capacity are the results derived from static load tests typically performed a few days after driving, to allow time for soil set up or freeze to occur. Set up is the increase in soil strength as the excess pore water pressure created during driving dissipates for cohesionless soils or the partial strength that is regained after remoulding effects for cohesive soils dissipated. The average time for excess pore water pressure to dissipate for cohesive soils is in the range of days to weeks depending upon hydraulic conductivity, thus static load tests performed on piles whose majority of resistance is from clay or silt layers may give conservative values of pile capacity subject to the time between driving and load testing.

Many types of pile load tests exist such as, compression, extraction, tensile or pull, and lateral load tests. Seven types of compressive tests are commonly performed in North America; quick, maintained, loading in excess of maintained, constant time interval, constant rate of penetration, constant movement increment, and cyclic loading tests, all of which are standardized in ASTM D1143: Standard Test Methods for Deep Foundations Under Static Axial Compressive Load. Only the quick test is mandatory when testing piles according to the ASTM standard, the remaining six are optional.

Standard D1143 allows for pile load tests to be conducted between 3 and 30 days or longer after driving to allow for soil setup or relaxation thus the measured capacity is as close to the long term pile capacity as possible. For more detailed information on the various types of pile load tests consult ASTM D1143 directly. During the quick test, the pile is loaded at increments of 5% of the total estimated failure load, as calculated from one of the methods described in sections 4.1 or 4.2. The load is kept constant for a minimum of 4 to a maximum of 15 minutes during which time the pile displacement is recorded. The pile is similarly loaded until failure; however, the total load added should not be greater than the structural capacity of the pile. During unloading, the same time intervals are used and complete unloading is accomplished in five to ten equal decrements. Pile displacements are recorded at 0.5, 1, 2, 4, 8, and 15 minute intervals after each load increment, at 1, 4, 8, and 15 minute intervals after each unloading decrement, and after all loads are removed.

The maintained test involves loading the pile up to 200% of the design load or until failure, whichever occurs first. Each load is applied in increments of 25% of the design load and held until the rate of pile movement is less than 0.25 millimetres (0.01 inches) per hour. If failure occurs before the total 200% load weight is added, the failure load is maintained until the pile movement equals 15% of the pile width or diameter, after which the load is taken off the pile in 25% decrements with a maximum of 1 hour between decrements. The overall test should take at least 12 hours. While performing the maintained test as well as the loading in excess of maintained, constant time interval loading, and cyclic loading tests the pile displacements are recorded before and after each loading increment at the 5, 10, and 20 minute mark as well as every 20 minutes thereafter. After the total load is applied, readings are taken in the same manner for the first two hours of the test and then every hour after that until the 12th hour and then every 2 hours until the 24th hour of the test. If failure occurs, the pile displacement is recorded immediately prior to removing the first load. During unloading, pile movements are recorded at 20 minute intervals for each unloading decrement as well as the 12th hour after all loads are removed.

Loading in excess of the maintained test is performed after the pile is unloaded from a previous test by loading it to its preceding maximum capacity in 50% increments within a maximum of 20 minutes per increment. After which load increments decrease to 10% of the design load until failure or the maximum required load is obtained. If failure occurs the load is increased until the pile displacement is 15% of the pile width or diameter. Otherwise the full load is held for 2 hours then the load is taken off in four equal decrements of 20 minutes each.

The methodology of the constant time interval loading test is done in a manner similar to the other tests however the increments are 20% of the design load and are completed in 1 hour stages. The unloading decrements are also completed in 1 hour intervals.

The constant rate of penetration test employs any device which can vary the load smoothly to ensure that the rate of pile displacement is 0.25 to 1.25 millimetres (0.01 to 0.05 inches) per minute in cohesive soils or 0.75 to 2.5 millimetres (0.03 to 0.1 inches) per hour in cohesionless soils. The load is increased until the pile penetration is 15% of the pile width or until pile advancement cannot continue. Because of the constant rates of penetration readings are taken every 30 seconds during loading and immediately after unloading as well as an hour after the final load is removed.

The constant movement increment test is performed by applying loads which cause the top of the pile to advance downward at a rate of 1% of the pile width, additional loads are only introduced when the variation in load required to keep the movement increment constant is less than 1% of the total load applied per hour. The pile is loaded in this fashion until the total displacement is equal to 15% of the pile width or diameter. The pile is unloaded in four equal decrements, each one taken off when the rate of load variation is less 1% of the total load applied per hour. Pile displacements are recorded immediately prior to and after each loading increment as well as each unloading decrement and as frequently as necessary to ensure that the rate of load variation is properly calculated to maintain each incremental settlement as well as measure the pile rebound due to unloading. Additionally, a final reading is recorded 12 hours after the final load is removed.

When completing the cyclic loading test, the pile is loaded to 50, then 100, then 150% of the design load holding the total test load of each case for an hour. While unloading, the decrements are equal to the loading ones but are completed in 20 min intervals. After removing the maximum load the following loads are applied in 50% increments of the design load allowing 20 minutes between load increments.

Despite the method(s) chosen, the results are a set of graphs which charts the applied loads and test times against pile displacements. ASTM D1143 classifies failure during a test as when the total applied load causes the pile displacement to be 15% of the average pile width or diameter; however the safe working load or capacity of the pile is determined by the engineer according to the test results.

The Canadian Foundation Manual (CFM, 1985) suggests performing the quick or constant rate of penetration tests because they are less time consuming, more cost effective, and the results, in terms of estimating bearing capacities are easier to interpret than those from the other methods described above. In Canada there are three main methods are used for determining the safe bearing capacity of the pile, listed below. This is done by measuring the amount of deflection as loading is increased and comparing it to the settlement that the pile undergoes.

4.5.1 Brinch Hansen 80% Failure Criterion

First proposed in 1963, the Brinch Hansen 80% failure criterion assumes that the mechanism governing pile failure is plunging, thus the derived bearing capacity value it gives is considered the true pile failure value. To determine the ultimate bearing capacity of the pile, a chart of the root of settlement over load ($\sqrt{\Delta}/Q$) against settlement (Δ) is graphed, as seen in Figure 28.

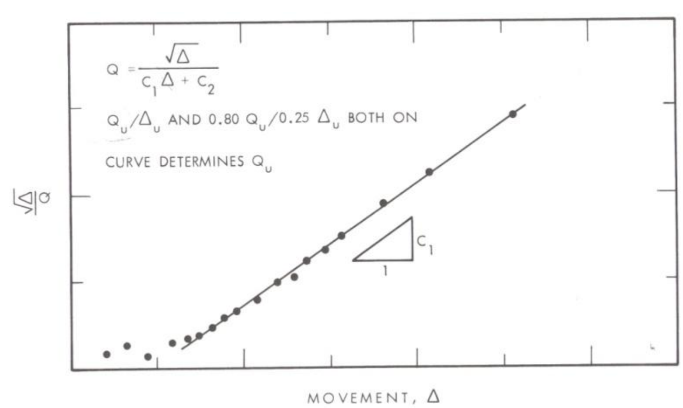


Figure 28: Brinch Hansen 80% Failure Criterion Example (CFM, 1985)

The slope of the line in Figure 28 is denoted as C_1 and the y-intercept as C_2. The ultimate load is then given by Equation 43:

$$Q_u = \frac{1}{2\sqrt{C_1 C_2}} \quad (43)$$

where: Q_u is the ultimate load (kN or tons), C_1 is the slope of the graphed line ($1/\mathrm{kN}\cdot\sqrt{mm}$ or $1/\mathrm{ton}\cdot\sqrt{in}$), and C_2 is the y-intercept ($\sqrt{mm}$/kN or $\sqrt{in}$/tons).

The calculation of Q_u is only valid if the following criteria are met; namely that the points plot as a straight line and the load test causes a plunging failure of the pile to occur. Confirmation that the second criteria is done by plotting the point ($0.25\Delta_u$, $0.8Q_u$) on the load settlement graph, if it lies on or near the curve then the pile has been loaded to failure. The value of Δ_u (mm or in) is taken as the y intercept divided by the slope of the Brinch Hansen graph (C_2/C_1).

If plunging failure is not reached then the standard practice is to assume that the maximum applied load is the load which causes failure. This is obviously erroneous and data from pile load tests should not be extrapolated to obtain higher working load values than tested, for safety reasons.

4.5.2 Chin Failure Criterion

Published in 1970, the Chin failure criterion results in a straight line graph, indicating if a pile reaches failure during the load test, similar to that of the Brinch Hansen method having settlement (Δ) on the x axis but with the settlement over load (Δ/Q) on the y axis, not the root of settlement over load.

The inverse of the slope of the straight line is taken as the ultimate pile bearing capacity; however, the value is always greater than the maximum load applied during the static pile test. Thus extreme caution must be used if this value is used as the basis of design procedures. The Chin failure criterion does have the advantage of being able to show if the pile is damaged or not during the load test. If so this shows up as sharp changes or curves in the graphed line.

4.5.3 Davisson Offset Limit Load

Developed in 1972, the Davisson offset limit load is a method which is used to predict the safe bearing capacity of piles by considering the elastic shortening that the pile undergoes when having various loads placed on it during pile load tests. To be effective, the resulting load settlement chart should be linear on both axes and have a scale such that when graphing the elastic compression of the pile the line it forms is at an angle of approximately 20 degrees between it and the load axis of the chart. Calculating the safe bearing capacity of the pile involves determining the load (Q_L) which corresponds to a specified pile head displacement (Δ) as given by the formula below (CFM, 1985):

$$\Delta = \delta + (4 + 8\mathrm{b})\mathrm{x}10^{-3} \tag{44}$$

where: Δ is the movement of the pile head (mm), b is the width or diameter at the pile toe or expanded pile base (mm), and δ is the elastic shortening of the pile (mm); as given below:

$$\delta = \frac{QL}{A_p E} \tag{45}$$

where: Q is the applied load (kN or ton), L is the pile length (metre or feet), A_p is the cross sectional area of the pile (m^2 or ft^2), and E is the elastic or Young's modulus of the pile material (kN/m^2 or ton/ft^2).

Once Δ is known, it is input into Equation 46.

$$Q = \frac{AE}{L}\Delta \tag{46}$$

where: all parameters are the same as defined above. Equation 43 gives a straight line which is the theoretical elastic compression or deformation that the pile undergoes during testing. A parallel line is then drawn with an offset given by:

$$x = (4 + 8\mathrm{b})\mathrm{x}10^{-3} \tag{47}$$

The point where the parallel line intersects the load settlement curve is taken as the offset limit load (Q_L) as seen in Figure 29.

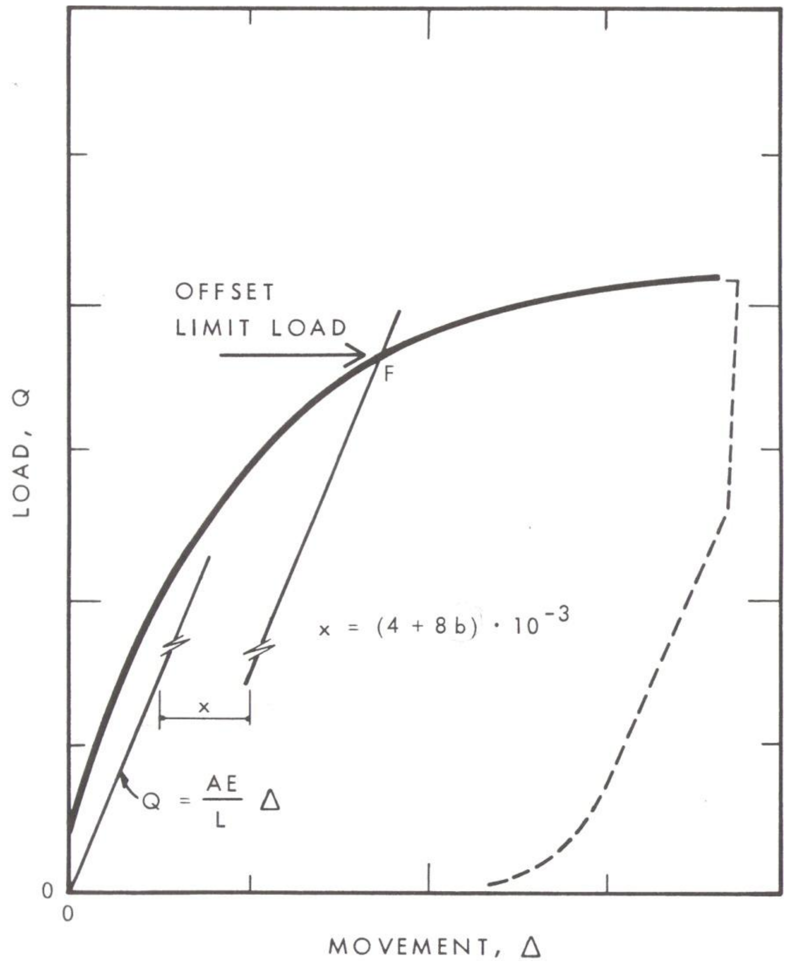


Figure 29: Davisson Offset Limit Load Determination Example (CFM, 1985)

The offset limit load is developed for end bearing driven piles assuming that little to no resistance is developed by skin friction. Other assumptions are that the pile head is free and the end is fixed. These assumptions are not always valid and as such the calculated allowable load are overly conservative.

Some limitations of the offset limit load are that if it is used for bored piles the estimated pile capacity is overly conservative, it is very susceptible to errors in recording amounts of loads, pile movements, and estimating Young's modulus, especially for pre-cast and cast in place concrete piles and if used for friction piles the elastic deformation is less than the theoretically calculated value and thus the calculation of safe bearing capacity of the pile is overestimated. Since the methodology is developed for driven piles, the offset distance or required pile displacement, x, may not be sufficient for cast in place piles to fully develop toe bearing resistance. This is due to the fact that in situ piles develop a dense soil bulb during placement thus the amount of settlement needed to undergo is greater than that of displacement piles.

4.6 Load And Resistance Factor Design

All the methods discussed thus far estimate pile capacity on the basis of an allowable stress design (ASD). The ASD method involves determining the ultimate capacity or strength of a pile by various methods which calculate the ultimate load that a pile can support through theoretical, empirical or semi-empirical formulae, and physical tests. The allowable load is then calculated as the ultimate pile strength, resistance or capacity divided by a factor of safety. The factor of safety is somewhat arbitrary and for pile design can range anywhere from slightly greater than 1 to 6, although a value of 1.5 to 3 is typical.

The general rule of thumb is that the larger the uncertainty in design, the larger the factor of safety used. In certain instances the factor of safety is predetermined such as built in ones within certain dynamic formulae or prescribed as in specific building codes. In other cases the factor of safety is dependent upon the judgement and experience of the engineer. Uncertainty in design is with respect to the variability or unknowns in applied pile loads, estimated and measured pile capacities, assessed and assumed soil properties, and rated and actual installation equipment parameters. Another consideration for determining safety factors is the consequence of failure; the greater the magnitude of failure, monetary loss or loss of life, the greater the factor of safety employed. Unfortunately as the factor of safety increases so does the cost of the construction project, thus from an economical viewpoint, the less efficient the design becomes.

Due to the subjective nature of assigning the actual value to the factor of safety, projects which use the ASD method are occasionally criticized as being overdesigned, wasteful, and redundant. This is because during typical pile design, factors of safety are used at many sections, not just at the end calculation when the ultimate pile capacity is determined. During the initial design piles are never intended to be loaded to their full structural capacity, predictive formulae use various factors of safety to estimate bearing capacities, and static load test derived capacities are from investigations which generally only load piles enough to cause a set amount of settlement. When various piles are tested the assumed bearing capacity for all piles is set equal to the results of the weakest pile and in the case of differential loadings the load that every pile undergoes during the foundation lifespan is taken as the maximum possible load which is most likely to occur. Another point of contention is that the determined safe pile load comes from a mixture of different techniques, namely theoretical, dynamic, and static methods. Theoretical techniques assume that parameters are exact, constant, and every aspect of the foundation soil system acts ideally which is erroneous. Combining static and dynamic methods are equally faulty since it assumes that piles behave the same during static loading as they do during dynamic loading which is also incorrect.

One method used which attempts to overcome these limitations is to perform a reliability based design which uses probabilities to estimate the actual or maximum pile capacities developed based on examining critical design parameters such as pile strengths, capacities, moments, deflections, settlements, etc. through numerous samples of previously installed pile datasets, therefore these designs are referred to as ultimate stress designs or USDs. The three most popular reliability techniques include stochastic, first order second moment (FOSM), and load and resistance factor design methods (LRFD). The manufacturing industry utilizes reliability methods which are proven to save money and aid in increasing the efficiency and reliability of designs (Coduto, 2001). Despite the method being used the aim of each is the same; to quantify the probability that a specific design fails or succeeds based upon a specified reliability while ensuring that the project is still as economical as possible. Corollary effects are that the results obtained may be used to form construction guidelines to aid in the overall design or specific components of deep foundations. LRFD is the simplest of the reliability methods discussed and the most similar to traditional ASD methods, thus it is likely to be the most readily adopted by civil and geotechnical engineers. Designs are based on performing a probability analysis on a statistically sufficient number of samples to determine how they are distributed and allowing parameters such as means, standard deviations and coefficients of variation to be calculated and probability density functions to be created. For deep foundations, the probability density functions typically describe the likelihood of a pile achieving a certain bearing capacity or having a specific load applied to it, as seen in Figure 30.

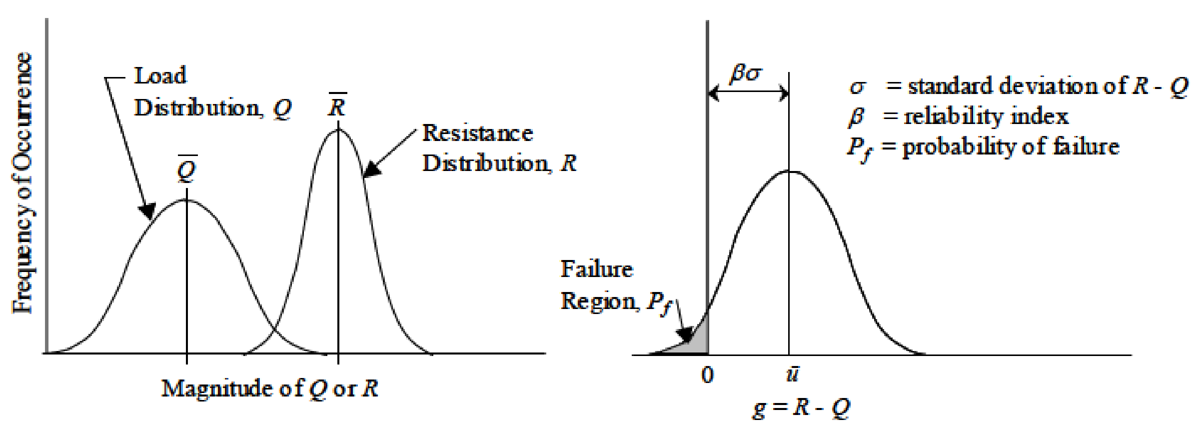


Figure 30: PDF of deep foundation bearing capacity (Allen, 2005)

The overlapping sections of the density functions above indicate the probability that the applied load exceeds the pile resistance for a specific design. The design is then stipulated to ensure that the possibility of pile failure during its expected lifespan is as low as feasibly possible. To account for the variability and uncertainties in field conditions, pile behaviour, developed capacities and applied loads, load and resistance factors are calculated to ensure that a certain level of reliability in the design is consistently achieved. Resistance factors, also known as reduction factors, are values less than unity while the load factors are greater than unity. This is similar to factors of safety used in ASD methods; however, whereas safety factors are based upon uncertainties in design and are somewhat subjective, LRFD factors are based upon examination of piles by type, installation method, soil conditions, and their behaviour via measured resistance. These factors are then applied to current or newly developed formulae to determine if a particular pile design is adequate to support the desired load or indicate if another type of pile or installation method is required.

Relatively new to the field of geotechnical engineering; LRFD has been used in concrete design and steel construction since 1960 and 1986, respectively, and is currently in development for wood and masonry applications (Coduto, 2001). Unlike steel or concrete, soil is not a manmade substance thus properties such as strength are not constant but vary considerably from site to site, soil layer to layer, within a single layer laterally and even vertically within a soil profile. This variability may cause difficulty in the determination of the factors and may result in a range of values rather than a single value.

In the United States, departments of transportation (DOT) such as those of Washington State (Allen, 2005) commissioned studies to better predict pile capacities from dynamic formulae and wave analysis programs such as ENR, Hiley, Gates, WEAP, GRLWEAP, CAPWAP equations. LRFD is used to calibrate these formulae to improve their pile predictive capabilities. The results from the studies concluded that LRFD analyses along with Monte Carlo simulations most improved predications of CAPWAP formulae with resistance, or reduction, factors of 0.7 to 0.8.

Ideally, LRFD allows for the creation of formulae which better predicts pile behaviour consistently to a predetermined degree of reliably based upon objective measurements; however, the current state of load and resistance factors is to modify existing dynamic formulae rather than develop new ones based entirely upon results from LRFD studies and as such are still affected by the same limitations previously discussed such as using dynamic measurements to predict the static behaviour of piles.

5.0 RESULTS

In order to determine which dynamic formulae best predicts pile load bearing capacities, the MTO publication Pile Load and Extraction Tests 1954 – 1992 and given Excel files; PD_Records10s.xls, PD_Records20s.xls, and PD_Records30s.xls are used as the basis to create a database to compare pile load test results to the calculated pile capacities derived from the five formulae discussed in section 4.2.

The results from each dynamic formula are compared to the pile load tests as well as each other and subdivided into groups by hammer type and pile material to determine which factors affect dynamic predictive accuracy. Simple statistics are carried out to quantify the variability of each predictive method through R – squared, correlation, factor of safety, and standard deviation values.

5.1 MTO Database

The MTO publications contain a total of 371 pile installation records from 41 different sites across northern and southern Ontario. All the piles are load tested, however out of 98 compression tests taken to failure only 77 are determined as adequate for pile analysis, 48 load tests are pull or extraction tests, 118 tests are repeated compression and extraction tests, 24 piles are lateral load tests, 9 piles are installed by boring or coring methods, 2 piles are installed at orientations other than vertical, 40 compression static load tests are not taken to failure or their maximum permissible load and the remainder are missing pertinent information required for analysis such as static load test results, final set or blow count and hammer information including type, weight, and rated energy output. If this information is found and added, the database can potentially expand to 148 piles, almost double the current size and potentially add more reliability to the derived results. Tables 19 through 22 goes into detail with regards to site number, pile number, material type, driven lengths, and load tests performed. Table 23 presents the piles which are not used during the analysis and states which missing parameters are required.

For a complete record of all MTO piles which have undergone compression load tests please see Appendix B.

Table 19: Compression Load Tested Piles used for Analysis

Site Number	Pile Number	Pile Type	Embedded Length (m)
1	C1	Size 32 Timber – Untreated	7.19
2	5	Steel Tube Concrete Filled	5.79
4	2	Steel Tube Concrete Filled	35.97
7	2	HP 310 x 79	22.25
14	2	Steel Tube Concrete Filled	18.29
15	1	Franki Displacement Pile	7.32
15	2	Size 36 Timber – Untreated	8.99
16	3	Size 36 Timber- Untreated	12.19
17	1	HP 310 x 110	25.72
17	2	HP 310 x 110	26.47
22	3	Steel Tube Concrete Filled	15.30
22	4	Steel Tube Concrete Filled	30.15
22	5	Steel Tube Concrete Filled	15.28
22	9	Size 36 Timber – Untreated	14.46
23	1	Size 36 Timber – Untreated	3.11
23	2	Steel Tube Concrete Filled	3.02
23	3	HP 310 x 110	3.05
24	1	Size 36 Timber - Untreated	14.25
24	2	Steel Tube Concrete Filled	15.39
24	3	Steel Tube Concrete Filled	22.40
24	4	HP 310 x 79	22.40
24	5	HP 310 x 79	15.39
25	1	Steel Tube Concrete Filled	5.64
25	4	HP 310 x 79	18.44
25	5	Steel Tube Concrete Filled	18.35
25	6	Steel Tube Concrete Filled	9.27
25	9	HP 310 x 79	9.39
26	1	Steel Tube Concrete Filled	12.19
26	4	Steel Tube Concrete Filled	30.48
26	5	Steel Tube Concrete Filled	42.67
26	9	Size 36 Timber – Untreated	21.95
28	1	HP 310 x 79	6.10
28	2	HP 310 x 79	18.29
28	3	HP 310 x 79	12.19
28	4	Precast Concrete Pile	11.89
28	5	Precast Concrete Pile	17.98
28	6	Precast Concrete Pile	5.79
28	7	Steel Tube Concrete Filled	6.10
28	8	Steel Tube Concrete Filled	18.29
28	9	Steel Tube Concrete Filled	12.04
29	1	Size 33 Timber – Untreated	13.72
29	2	Size 33 Timber – Untreated	13.72
31	1	Size 30 Timber – Treated	6.55
31	2	Size 30 Timber – Treated	4.72
31	3	Size 36 Timber – Treated	3.51
32	4	Size 36 Timber – Treated	13.48

Site Number	Pile Number	Pile Type	Embedded Length (m)
32	5	Size 33 Timber – Treated	9.14
32	6	Size 30 Timber – Treated	7.58
33	2	Steel Tube Concrete Filled	32.67
33	3	Precast Concrete Pile	34.85
33	4	Precast Concrete Pile	16.61
33	5	Size 36 Timber – Treated	8.69
35	1	HP 310 x 110	14.78
35	4	Steel Tube Concrete Filled	14.69
35	5	HP 310 x 110	27.58
35	6	Steel Tube Concrete Filled	27.42
35	7	Size 36 Timber – Treated	12.67
35	10	Precast Concrete Pile	14.63
36	389	Steel Tube Concrete Filled	29.57
36	391	Steel Tube Concrete Filled	31.39
37	3	HP 310 x 79	14.48
37	5	HP 310 x 79	31.24
37	6	HP 310 x 110	14.48
37	8	HP 310 x 110	30.92
37	9	Size 36 Timber – Untreated	9.55
37	10	Size 36 Timber – Treated	10.36
38	2	Size 36 Timber – Treated	3.30
38	3A	Size 34 Timber – Treated	5.00
38	4	Steel Tube Concrete Filled	11.90
38	5	Steel Tube Concrete Filled	16.10
39	1	Size 36 Timber – Treated	17.13
39	2	HP 310 x 110	25.50
39	3	Steel Tube Concrete Filled	25.40
40	1	Size 36 Timber – Treated	14.70
40	2	HP 310 x 110	24.50
40	3	Steel Tube Concrete Filled	17.20
41	1	Size 36 Timber – Treated	8.00

Table 20: Extraction Load Tested Piles

Site Number	Pile Number	Pile Type	Embedded Length (m)
22	3	Steel Tube Concrete Filled	15.30
22	4	Steel Tube Concrete Filled	30.15
22	5	Steel Tube Concrete Filled	15.28
23	1	Size 36 Timber – Untreated	3.11
23	2	Steel Tube Concrete Filled	3.02
23	3	HP 310 x 110	3.05
24	1	Size 36 Timber - Untreated	14.25
24	2	Steel Tube Concrete Filled	15.39
24	3	Steel Tube Concrete Filled	22.40
24	4	HP 310 x 79	22.40
24	5	HP 310 x 79	15.39
25	1	Steel Tube Concrete Filled	5.64
25	4	HP 310 x 79	18.44
25	5	Steel Tube Concrete Filled	18.35
25	6	Steel Tube Concrete Filled	9.27
25	9	HP 310 x 79	9.39
26	1	Steel Tube Concrete Filled	12.19
26	4	Steel Tube Concrete Filled	30.48
26	5	Steel Tube Concrete Filled	42.67
26	9	Size 36 Timber – Untreated	21.95
28	2	HP 310 x 79	18.29
28	5	Precast Concrete Pile	17.98
28	7	Steel Tube Concrete Filled	6.10
28	8	Steel Tube Concrete Filled	18.29
28	9	Steel Tube Concrete Filled	12.04
29	1	Size 33 Timber – Untreated	13.72
29	2	Size 33 Timber – Untreated	13.72
35	1	HP 310 x 110	14.78
35	4	Steel Tube Concrete Filled	14.69
35	5	HP 310 x 110	27.58
35	6	Steel Tube Concrete Filled	27.42
35	7	Size 36 Timber – Treated	12.67
35	10	Precast Concrete Pile	14.63
37	3	HP 310 x 79	14.48
37	4	HP 310 x 79	38.94
37	5	HP 310 x 79	31.24
37	6	HP 310 x 110	14.48
37	7	HP 310 x 110	45.29
37	8	HP 310 x 110	30.92
39	1	Size 36 Timber – Treated	17.13
39	2	HP 310 x 110	25.50
39	3	Steel Tube Concrete Filled	25.40
40	1	Size 36 Timber – Treated	14.70
40	2	HP 310 x 110	24.50
40	3	Steel Tube Concrete Filled	17.20
41	1	Size 36 Timber – Treated	8.00
41	2	HP 310 x 110	19.50
41	3	Steel Tube Concrete Filled	16.00

Table 21: Lateral Load Tested Piles

Site Number	Pile Number	Pile Type	Embedded Length (m)
21	2	Cast In Situ Concrete Pile	18.59
21	6	HP 370 x 108	22.99
35	5	HP 310 x 110	27.58
35	6	Steel Tube Concrete Filled	27.42
37	3	HP 310 x 79	14.48
37	4	HP 310 x 79	38.94
37	5	HP 310 x 79	31.24
37	6	HP 310 x 110	14.48
37	7	HP 310 x 110	45.29
37	8	HP 310 x 110	30.92
38	1	HP 310 x 110	16.20
38	2	Size 36 Timber – Treated	3.30
38	3A	Size 34 Timber – Treated	5.00
38	4	Steel Tube Concrete Filled	11.90
38	5	Steel Tube Concrete Filled	16.10
39	1	Size 36 Timber – Treated	17.13
39	2	HP 310 x 110	25.50
39	3	Steel Tube Concrete Filled	25.40
40	1	Size 36 Timber – Treated	14.70
40	2	HP 310 x 110	24.50
40	3	Steel Tube Concrete Filled	17.20
41	1	Size 36 Timber – Treated	8.00
41	2	HP 310 x 110	19.50
41	3	Steel Tube Concrete Filled	16.00

Table 22: Piles Installed by Coring/Boring Methods

Site Number	Pile Number	Pile Type	Embedded Length (m)
3	A	Franki Displacement Pile	8.00
3	B	Franki Displacement Pile	12.31
15	1	Franki Displacement Pile	7.32
18	C1	Cast In Situ Concrete Pile	9.45
20	SA4	Franki Displacement Pile	16.46
21	2	Cast In Situ Concrete Pile	18.59
27	1	Cast In Situ Concrete Pile	5.79
27	2	Cast In Situ Concrete Pile	6.25
27	3	Cast In Situ Concrete Pile	5.92

Table 23: Piles Missing Essential Information for Analysis

Site Number	Pile Number	Pile Type	Embedded Length (m)	Missing Data
4	1	Not Recorded	Not Recorded	Pile Load Test Results
5	1	HP 310 x 79	33.53	Pile Load Test Results
5	37	HP 310 x 79	50.60	Hammer Model, Rated Energy, Final Set/Blow Count
5	43	HP 310 x 79	20.73	Hammer Model, Rated Energy, Final Set/Blow Count
5	F10	HP 310 x 79	16.76	Hammer Model, Rated Energy, Final Set/Blow Count
5	G5	HP 310 x 79	16.76	Hammer Model, Rated Energy, Final Set/Blow Count
6	3	Not Recorded	Not Recorded	Pile Load Test Results
6	5	Not Recorded	Not Recorded	Pile Load Test Results
6	8	Not Recorded	Not Recorded	Pile Load Test Results
7	1	Not Recorded	Not Recorded	Pile Load Test Results
7	3	Not Recorded	Not Recorded	Pile Load Test Results
8	1	Size 36 Timber – Treated	9.85	Hammer Model/Ram Weight
8	2	Size 36 Timber – Treated	10.06	Hammer Model/Ram Weight
10	A	Size 32 Timber – Treated	15.06	Hammer Model, Rated Energy, Final Set/Blow Count
10	D	Size 36 Timber – Treated	15.51	Hammer Model, Rated Energy, Final Set/Blow Count
11	1	HP 310 x 79	26.82	Hammer Model, Rated Energy, Final Set/Blow Count
18	T1	Size 30 Timber – Treated	12.50	Hammer Model, Rated Energy, Final Set/Blow Count
18	T2	Size 36 Timber – Treated	12.34	Hammer Model, Rated Energy, Final Set/Blow Count
18	T3	Size 32 Timber – Treated	12.38	Hammer Model, Rated Energy, Final Set/Blow Count
18	C1	Cast In Situ Concrete Pile	9.45	Hammer Model, Rated Energy, Final Set/Blow Count
19	3	Size 36 Timber – Untreated	13.72	Hammer Model, Rated Energy, Final Set/Blow Count
19	4	Size 36 Timber – Untreated	8.84	Hammer Model, Rated Energy, Final Set/Blow Count
21	7	HP 14 x 73	23.04	Pile Load Test Results
22	8	Steel Tube Concrete Filled	22.56	Pile Load Test Results
24	6	Steel Tube Concrete Filled	22.56	Pile Load Test Results
24	7	Steel Tube Concrete Filled	24.09	Pile Load Test Results

Site Number	Pile Number	Pile Type	Embedded Length (m)	Missing Data
24	8	Steel Tube Concrete Filled	25.91	Pile Load Test Results
24	9	Steel Tube Concrete Filled	22.25	Pile Load Test Results
25	2	Steel Tube Concrete Filled	18.29	Pile Load Test Results
25	3	Steel Tube Concrete Filled	18.29	Pile Load Test Results
25	7	Steel Tube Concrete Filled	18.29	Pile Load Test Results
25	8	Steel Tube Concrete Filled	18.29	Pile Load Test Results
26	2	Steel Tube Concrete Filled	36.58	Pile Load Test Results
26	3	Steel Tube Concrete Filled	36.58	Pile Load Test Results
26	7	Steel Tube Concrete Filled	36.58	Pile Load Test Results
26	8	Steel Tube Concrete Filled	36.58	Pile Load Test Results
29	35	Not Recorded	12.50	Hammer Model, Rated Energy, Final Set/Blow Count
29	65	Not Recorded	12.50	Hammer Model, Rated Energy, Final Set/Blow Count
29	95	Not Recorded	21.03	Hammer Model, Rated Energy, Final Set/Blow Count
30	3-1	Steel Tube Concrete Filled	44.50	Pile Load Test Results
30	3-2	Steel Tube Concrete Filled	39.01	Pile Load Test Results
30	4	Steel Tube Concrete Filled	40.39	Pile Load Test Results
30	5-1	Steel Tube Concrete Filled	39.93	Pile Load Test Results
30	5-2	Steel Tube Concrete Filled	40.54	Pile Load Test Results
30	9	Steel Tube Concrete Filled	40.84	Pile Load Test Results
30	10-1	Steel Tube Concrete Filled	39.01	Pile Load Test Results
30	10-2	Steel Tube Concrete Filled	40.23	Pile Load Test Results
30	12	Steel Tube Concrete Filled	39.93	Pile Load Test Results
31	41	Size 14 Timber – Treated	6.40	Pile Load Test Results
31	43	Size 14 Timber – Treated	5.18	Pile Load Test Results

Site Number	Pile Number	Pile Type	Embedded Length (m)	Missing Data
31	51	Size 14 Timber – Treated	5.49	Pile Load Test Results
34	15	Steel Pile Concrete Filled	18.59	Hammer Model, Rated Energy, Final Set/Blow Count
34	18	Steel Pile Concrete Filled	18.59	Hammer Model, Rated Energy, Final Set/Blow Count
34	19	Steel Pile Concrete Filled	18.59	Hammer Model, Rated Energy, Final Set/Blow Count
34	23	Steel Pile Concrete Filled	18.59	Hammer Model, Rated Energy, Final Set/Blow Count
34	25	Steel Pile Concrete Filled	18.59	Hammer Model, Rated Energy, Final Set/Blow Count
34	27	Steel Pile Concrete Filled	18.59	Hammer Model, Rated Energy, Final Set/Blow Count
35	2-1	Steel Pile Concrete Filled	27.13	Pile Load Test Results
35	2-2	HP 12 x 74	24.99	Pile Load Test Results
35	3	Steel Pile Concrete Filled	25.30	Pile Load Test Results
35	8	Steel Pile Concrete Filled	27.43	Pile Load Test Results
35	9	Steel Pile Concrete Filled	28.48	Pile Load Test Results
35	12	HP 12 x 74	26.52	Pile Load Test Results
35	14	HP 12 x 74	17.37	Pile Load Test Results
35	15	HP 12 x 74	25.30	Pile Load Test Results
35	16	HP 12 x 74	27.74	Pile Load Test Results
35	18-1	HP 12 x 74	17.37	Pile Load Test Results
35	18-2	HP 12 x 74	24.38	Pile Load Test Results
35	19	HP 12 x 74	24.69	Pile Load Test Results
35	21	HP 12 x 74	24.38	Pile Load Test Results
35	22	HP 12 x 74	27.13	Pile Load Test Results

In summary from the 77 piles analysed; 27 piles are composed of concrete filled steel tubes, 24 are composed of timber, 19 are composed of steel H sections, and 7 are composed of precast concrete. Examining piles based on installation method, 32 are installed by drop hammer and 45 are installed by diesel hammer. Of the drop hammered installed piles, 10 are composed of concrete filled steel tubes, 12 are timber, 6 are steel H section, and 4 are precast concrete piles. Of the diesel hammered installed piles, 17 are composed of concrete filled steel tubes, 12 are timber, 13 are steel H sections, and 3 are precast concrete piles.

When comparing the Excel sheets to the MTO book, several pile records repeated between the two sources; however some discrepancies exist with respect to pile types, driven lengths, hammer types, rated energies or blow counts. In the case of conflicting information, the MTO book is taken as having the correct information and values from it are used as the source of all calculations. Other pile records which are missing pertinent information such as hammer energies, fall heights, and ram weights are taken from literature published about the specific hammer make and model used during driving. Those records which are missing information which is essential to perform predictive bearing capacity calculations or the information could not be found in published literature are omitted from the study database.

5.2 Dynamic Predictions

To calculate bearing capacities according to Equations 31 to 38, only the rated energy and set or blow count is required; however, for the Hiley and MTO modified Hiley formulae the weight of the pile including any soil around or within the pile is also required.

In Ontario, the two most common species of trees grown which are used for piling are jack pines and red pines, however throughout Canada other species commonly used include Douglas fir, western red cedar, western larch, spruces, oaks, maples, and tamarack (CITC, 1962). Since it is not known which species are used in the MTO database, bearing capacity calculations are performed using both the low and high end range of timber densities ranging from 390 to 810 kg/m^3 (24 to 51 lb/ft^3) for tree species based upon values presented in ASTM standard 2555-06. The percent difference in bearing capacities predicted from using these two values range from 1.3% to 20.9% for calculations derived from the Hiley formula and 1.6% to 30.9% for calculations derived from the MTO modified Hiley formula. Although the percent difference of the latter numbers appears significant, the average percent difference derived from the Hiley and MTO modified Hiley formulae are 6.8% and 9.4%, respectively, therefore they are considered an insignificant source of variation.

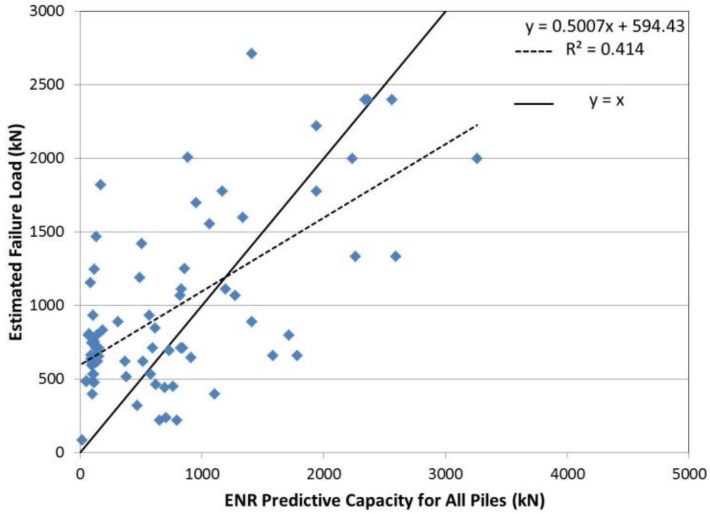
Since only the material density of H piles and steel tubes are given in the database, the density of all other materials is estimated based on typical values found in literature. For H piles, the total pile weight is assumed to include the weight of the steel pile itself plus the soil displaced between the flanges. For the calculation two assumptions are made; that the soil displaced within the flange extends for the entire length of the pile and the density of the soil is 2 650 kg/m^3 (165 lb/ft^3).

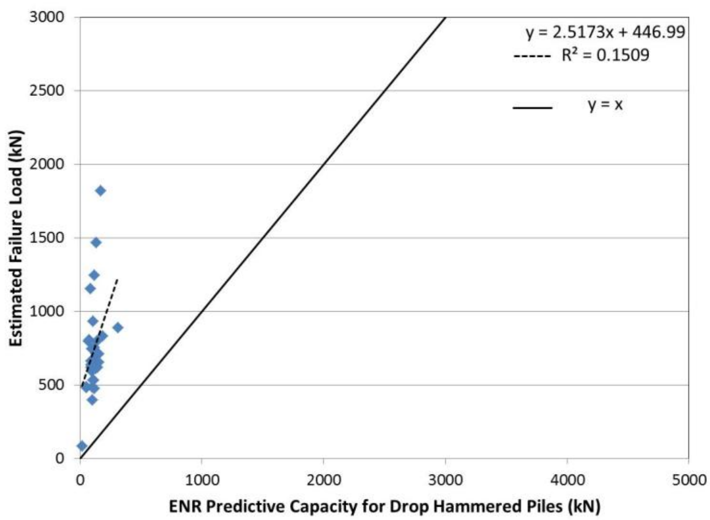
Similarly as for H piles, the linear weight of steel tubes is given in units of kg/m or lb/ft; however the density of concrete is not. From literature values, the density of concrete is assumed to range from 2 082 to 2 243 kg/m^3 (130 to 140 lb/ft^3). Bearing capacities are determined using both densities and have a percent difference ranging from 1.0% to 1.9% from Hiley formula calculations and 1.1% to 3.0% from MTO modified Hiley formula calculations for concrete filled steel tube piles and 1.0% to 2.2% from Hiley formula calculations and 1.3% to 3.6% from MTO modified Hiley formula calculations for precast concrete piles.

On average, the percent difference in predicted bearing capacities from using the high and low end density values of wood and concrete is determined as 3.7% from the Hiley formula and 5.3% from the MTO modified Hiley formula. Since this is a relatively small difference in bearing capacities, for consistency, the larger values are arbitrarily chosen and always used for analysis purposes.

For further detail on pile driving records including blow counts, pile types, hammer types, rated energies, and dynamic bearing capacity calculations see Appendix C. To determine any trends in the predicted bearing capacities derived from each dynamic formula, the piles are analyzed by first dividing them into three categories; all installed piles, those installed by drop hammer, and those installed by diesel hammers. The piles are further subdivided by pile type; timber, H, concrete filled steel tube, and precast concrete piles to determine if pile type contributes to any bias in the predicted capacities, either under or overestimating pile capacity. The following graphs, 31 to 35, show the predicated bearing capacities compared to the measured static capacities derived from pile load test results.

In the below graphs, predicted pile loads (in kN) calculated from dynamic formulae are presented on the x axis whereas the estimated failure loads (in kN) from static pile load tests are presented on the y axis. Ideally, if the predicted values match the load tested values exactly, all the data points lie on the black 1:1 (45^{o}) line giving the equation of the best fit line as $y = 1x + 0$ and a coefficient of determination of 1. A summary of equations of best fit lines and coefficients of determinations are presented in table 24 below.





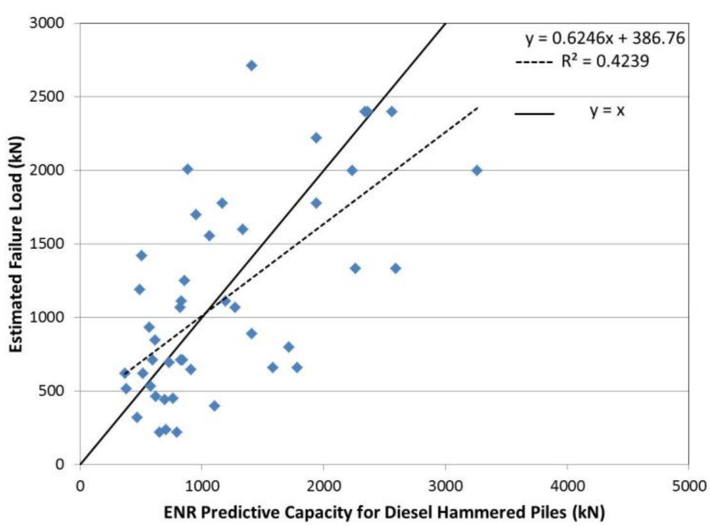
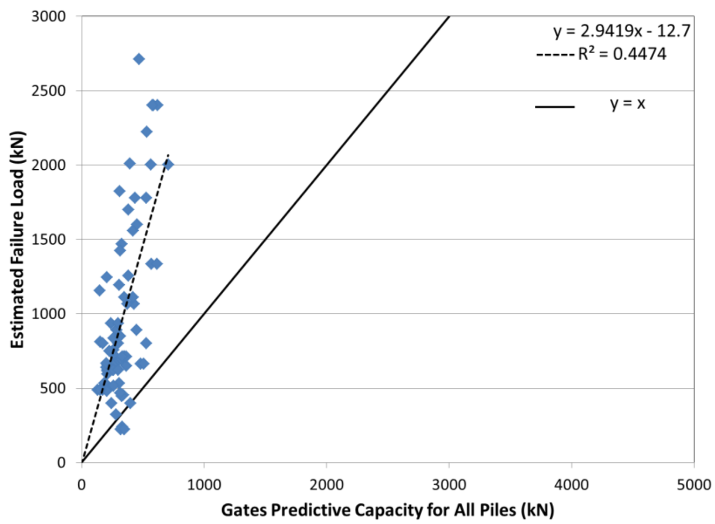
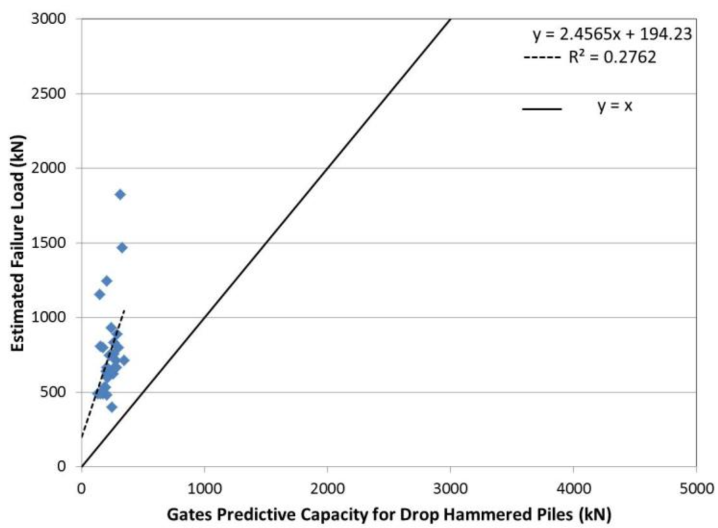


Figure 31: ENR formula predicted capacities vs. pile test failure loads





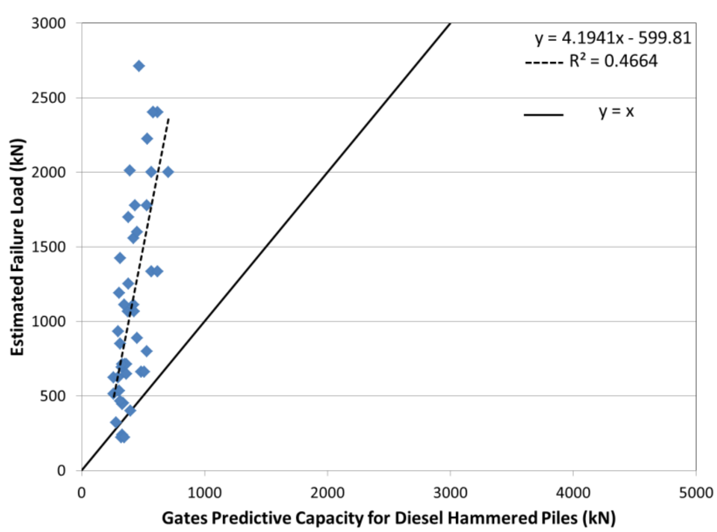


Figure 32: Gates formula predicted capacities vs. pile test failure loads

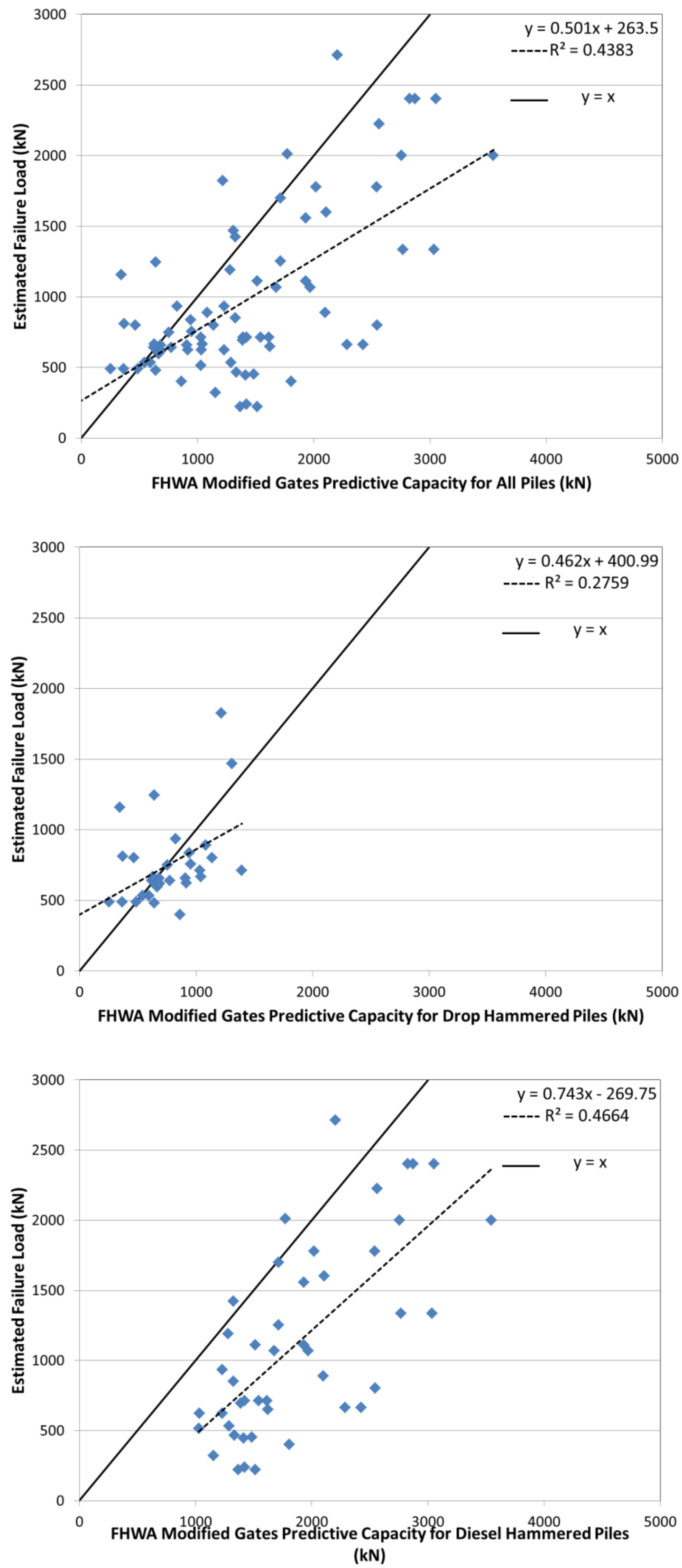


Figure 33: FHWA modified Gates formula predicted capacities vs. pile test failure loads

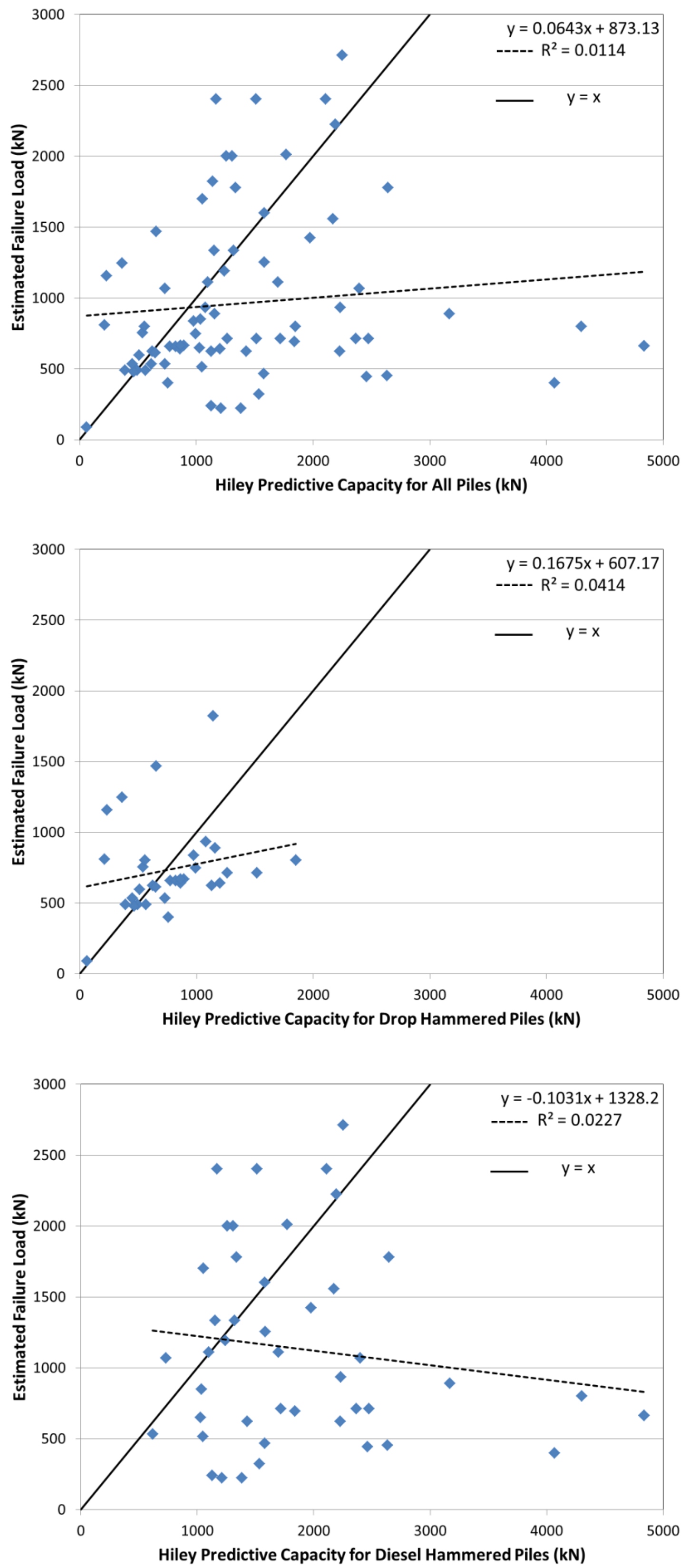


Figure 34: Hiley formula predicted capacities vs. pile test failure loads

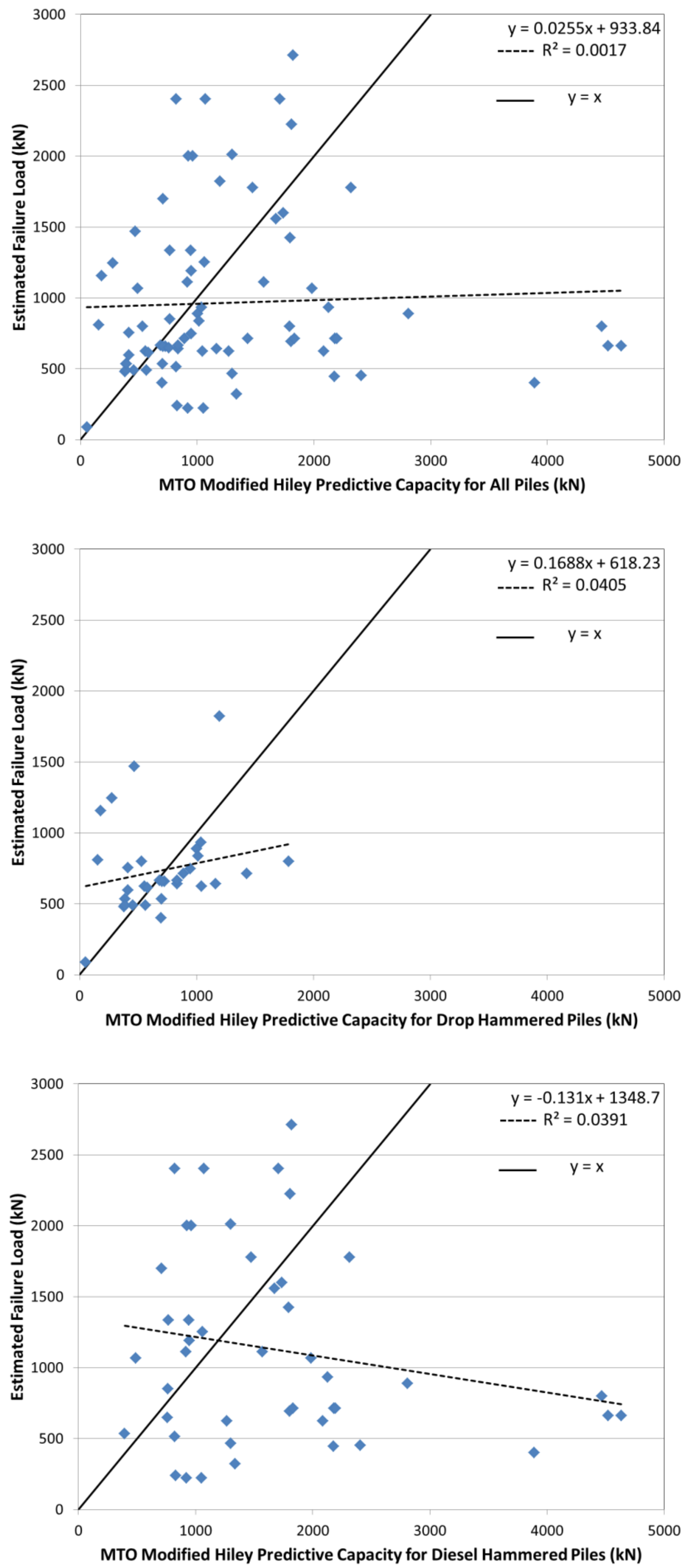


Figure 35: MTO modified Hiley formula predicted capacities vs. pile test failure loads

Table 24: Summary of Best Fit Lines and Coefficient of Determinations of Graphed Pile Capacities

Hammer	ENR Formula	Gates Formula	FHWA Modified Gates Formula	Hiley Formula	MTO Modified Formula
All Piles	y = 0.5007x + 594.43	y = 2.9419x – 12.7	y = 0.501x + 263.5	y = 0.0643x + 873.13	y = 0.0255x + 933.84
	R^2 = 0.414	R^2 = 0.4474	R^2 = 0.4383	R^2 = 0.0114	R^2 = 0.0017
Drop	y = 2.5173x + 446.99	y = 2.4565x + 194.23	y = 0.462x + 400.99	y = 0.1675x + 607.17	y = 0.1688x + 618.23
	R^2 = 0.1509	R^2 = 0.2762	R^2 = 0.2759	R^2 = 0.0414	R^2 = 0.0405
Diesel	y = 0.6246x + 386.76	y = 4.1941x - 599.81	y = 0.743x - 269.75	y = -0.1031x + 1328.2	y = -0.131x + 1348.7
	R^2 = 0.4239	R^2 = 0.4664	R^2 = 0.4664	R^2 = 0.0227	R^2 = 0.0391

Since the ideal situation does not occur, the equations for the best fit lines are useful in determining, qualitatively, the bias created from each formula used; whether the specific formula tends to under or over predict pile capacity. If the slope is greater than unity, on average, the dynamic formula under predicts the pile capacity and conversely if it is less than unity the formula over predicts the capacity. The y intercept gives an indication of how closely the trend line matches the 1:1 slope, the further away from an intercept of zero, the worse the match. Similarly the R^2 value gives an indication of the amount of variation within the data set; the smaller the value the less the correlation and the larger the amount of variation. Conversely, if all the data points fell on a straight line, the R^2 value would reach its upper limit of unity and the variation of the data would be minimal, therefore the equation for the linear best fit line and R^2 value for each graph is given in the top right hand corner.

From the above graphs, it is seen that in general the ENR formula over predicts the pile bearing capacity and results in a relatively low coefficient of determination of approximately 42% when examining all piles and piles installed by diesel hammers. While considering piles installed by drop hammers solely, the ENR formula under predicts the pile capacity and results in a significantly lower coefficient of determination, approximately 11%. Since the number of drop hammered piles to diesel hammered piles is similar, 32 compared to 45, it appears that the higher failure loads and coefficients of determination in the data points control the best fit line equations and coefficients of determination rather than the number of piles driven per installation method. The difference in trends between the drop and diesel hammered piles may be due to the fact that the ENR formula is based on semi empirical observations of pile driving in which ram weights are considerably less than the weight of the pile. This is still true for diesel hammers today; however, as progressively longer and larger capacity piles are required for modern day construction projects, the pile weights almost match the ram weights of drop hammers. The alternative is to increase the fall height; however increased velocities are more likely to damage the pile top. Another possible explanation is that the energy loss coefficient (c) of 1 inch is too large a value for modern drop hammers and thus the resulting bearing capacities are grossly under predicted.

The Gates derived predicative values, in general, under predict the pile bearing capacities despite the installation method used. However, in general, piles installed by drop hammers solely show a tendency to under predict pile capacities by a much smaller amount than those installed by diesel hammer. This is shown in the graphs as the value of the slope of 1.07 and 4.19 for drop and diesel hammered piles, respectively. This may be due to the fact that diesel hammered piles develop maximum estimated failure load approximately twice that of drop hammers, thus the slope is subsequently higher. Similarly with the ENR predicted capacities, the coefficient of determination of the Gates predicted capacities for the drop hammered piles is significantly lower than those from diesel hammered piles, 10% and 47%, respectively. This may also be due to the higher estimated failure loads reached by the diesel installed piles. A possible explanation of the nearly vertical trend in the data points of the Gates formula is that it is derived from piles which reached a maximum safe resistance of 600 kN (60 tons) and installed by drop and steam hammered piles only (Gates, 1957). Since piles within the MTO database exceed these bearing capacities and are also installed by diesel hammers it may not be adequate to extrapolate the formula to piles and installation methods past its original parameters.

In 1997, the FHWA suggested that U.S. DOTs start using the Modified Gates formula to better estimate the bearing capacity of piles. From the graphs it is seen that the data points seem to centre about the 1:1 line more than with the ENR and Gates methods even though the modified Gates formula does slightly over predict the pile bearing capacity when compared to the results of the pile load tests. The coefficient of determination is slightly higher than for the Gates formula when examining all the piles installed and those installed solely from diesel hammers; while those installed by drop hammers increased by approximately 2.5 times to 28 percent.

The Hiley formula also seems to centre on the 1:1 line for drop hammered piles and with a few noticeable data points lying to the extreme right of the graph, however the average from the best fit line indicates that on all three graphs the predicted capacities are considerably over estimated. The coefficients of determination are also very small at 1% to 4% which indicates a very large amount of scatter within the dataset. This is somewhat counter intuitive since the Hiley formula seems to account for most if not all the energy losses during driving by inputting values based on field observations or published literature to account for hammer efficiencies, coefficients of restitutions, temporary compression of the pile, pile cap material, and soil during driving. The large range in data may be caused by the semi arbitrarily used compressional values, since they are based on published literature rather than field observations; if a record of pile compressions caused by hammer strikes are recorded and used in the calculations more realistic predictive capacities may be obtained. Another cause of potential error is that the Hiley formula is based on specific assumptions, one being that the pile acts as a long slender rod and when it undergoes compression from pile driving the entire pile is compressed instantaneously over its entire length. Wave mechanics and published literature shows that this assumption does not hold true for very long piles and since the C_2 value is dependent upon pile length, it may be the source of some of the error in the predicted capacities.

Correspondingly to the Hiley predicted values, the capacities from the MTO modified Hiley formula are equally poor with similar over predicted tendencies and very low coefficient of determination values. This is due to the fact that the formulae are similar, with the exception for the values of the coefficients of restitution, and the fact that the same values of compression are used in both formulae since there are no recorded values of compression available in the MTO pile driving records.

Piles are also subdivided into groups according to material type; concrete filled steel tube, timber, steel H section, and precast concrete to determine if composition affects bearing capacity prediction accuracy. Graphs 36 to 45 show the predictive capacities from each dynamic formula for each pile type according to installation method used, drop and diesel hammer.

Examining the predicited values of the ENR formula, it appears that when piles are installed via drop hammer, the predictive formula severly underpredicts the bearing capacity to a maximum of approximately 300 kN, despite the type of pile used. For most material typs installed by diesel hammers, the data points seem to centre around the 1:1 line consistently, however for timber piles there is still a high amount of variation and the predicted values appear to spread out horizontally rather then along the 1:1 slope. For steel tube, H, and precast concrete piles, the piles tend to follow the 1:1 line much closer than drop hammered piles with a smaller amount of variation

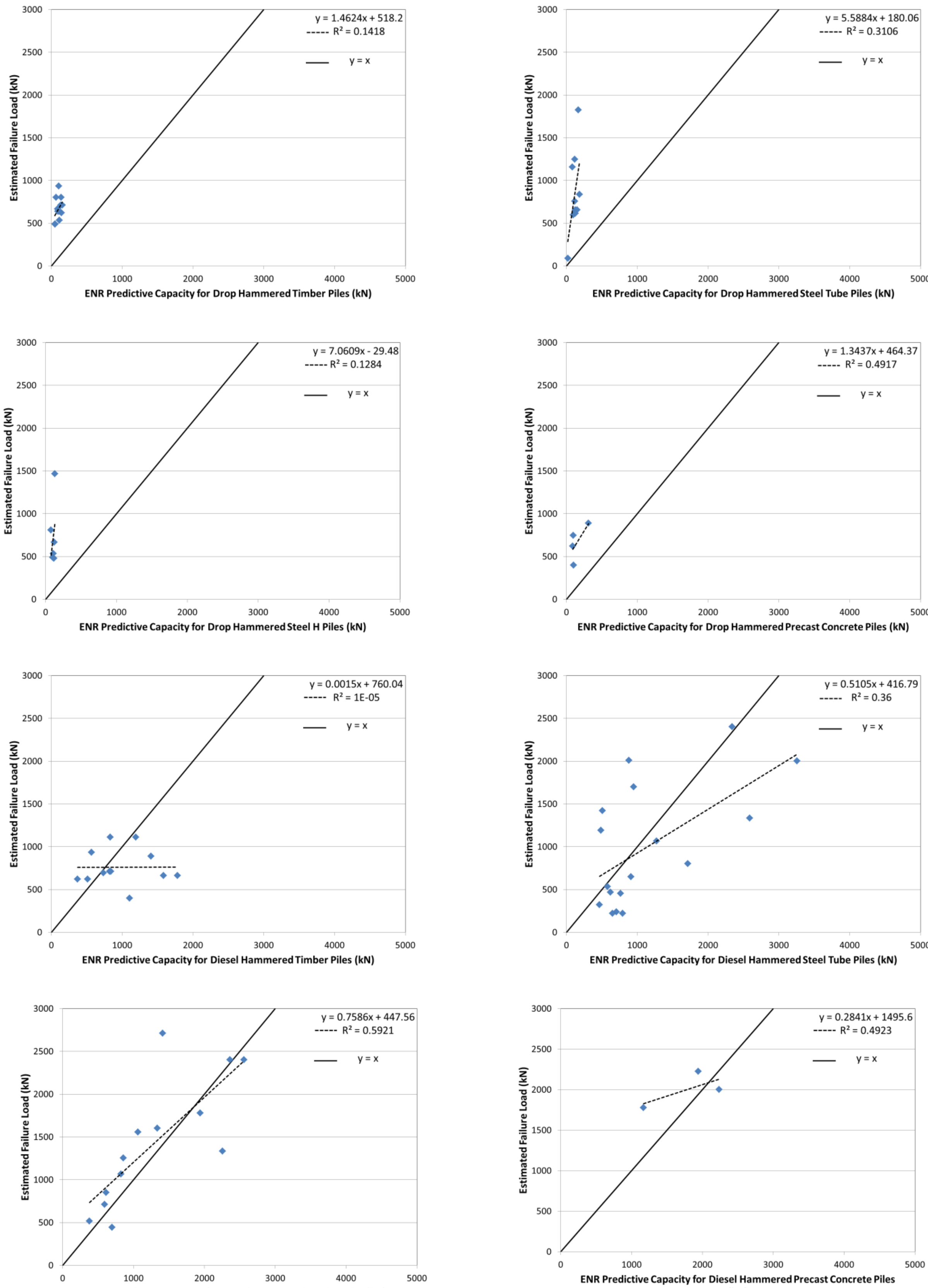


Figure 36: ENR formula predicted capacities vs. pile test failure loads divided by hammer and pile type

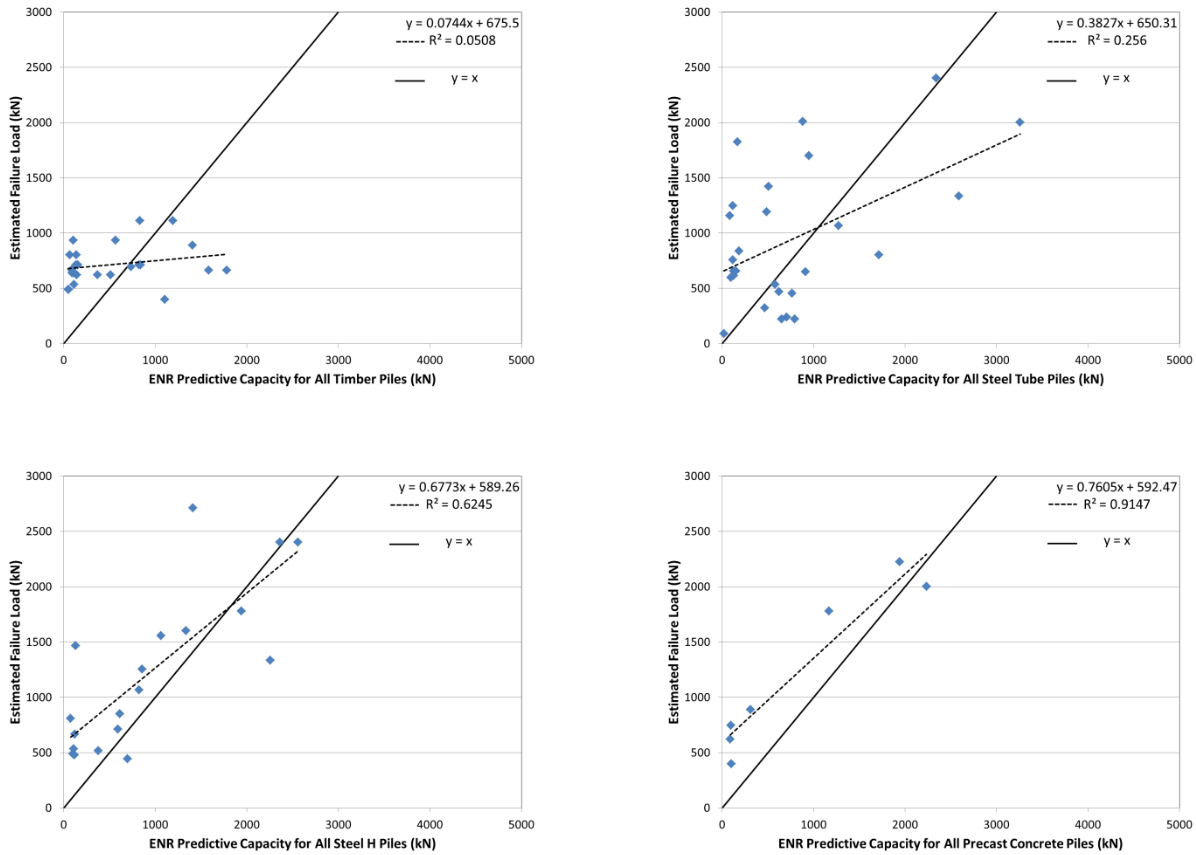


Figure 37: ENR formula predicted capacities vs. pile test failure loads divided by pile type

When looking at specific pile types; the predictive accuracy of the dynamic formula does not not improve by combining installation methods except for precast concrete piles. This may be due to the limited number of samples available for that particular pile type and by simply adding to the data set increases the derived reliability. Therefore the ENR formula may provide a good estimate for pile bearing capacity for steel tube, H, and precast concrete piles but not timber piles and only when intsalled by diesel hammers, not drop hammers.

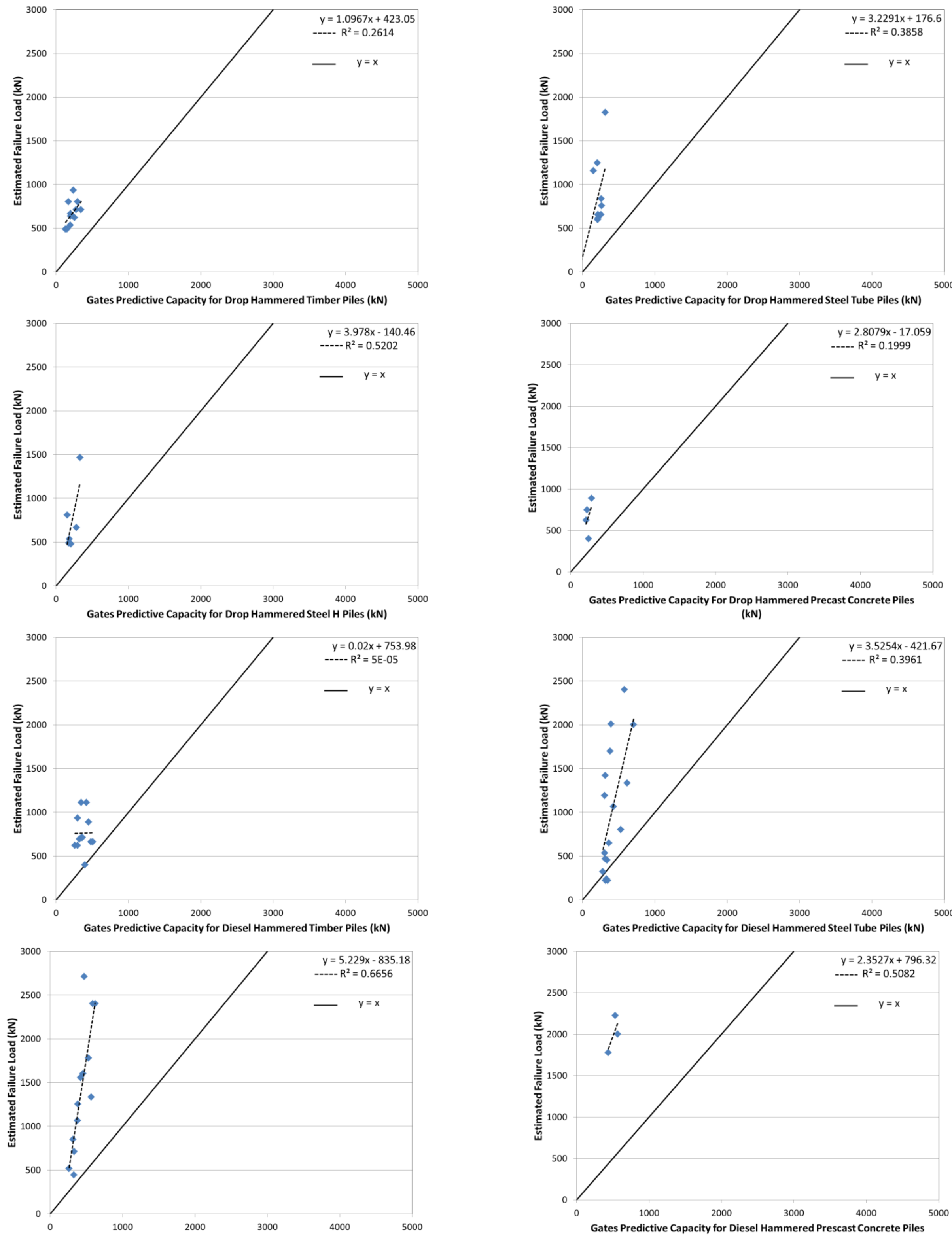


Figure 38: Gates formula predicted capacities vs. pile test failure loads divided by hammer and pile type

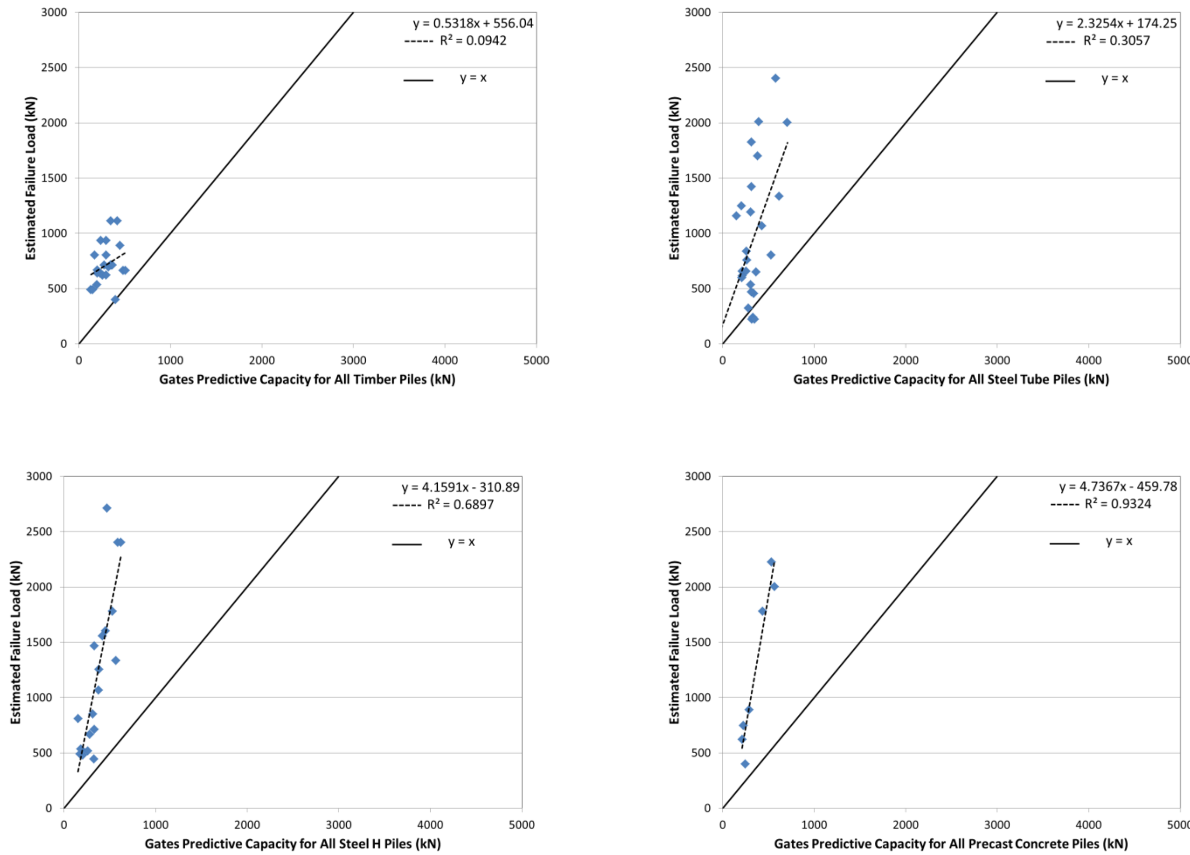


Figure 39: Gates formula predicted capacities vs. pile test failure loads divided by pile type

While examining the predicted capacity calculated from the Gates formula, according to installation method, pile type, and pile type and installation method together it is determined that the Gates formula underpredicts pile capacity for each combination studied. However the variation within the data for diesel hammer installed piles is on average smaller than that for drop hammer installed piles, with the exception of timber piles which result in some of the lowest coefficient of determination values of all pile types and installation methods.

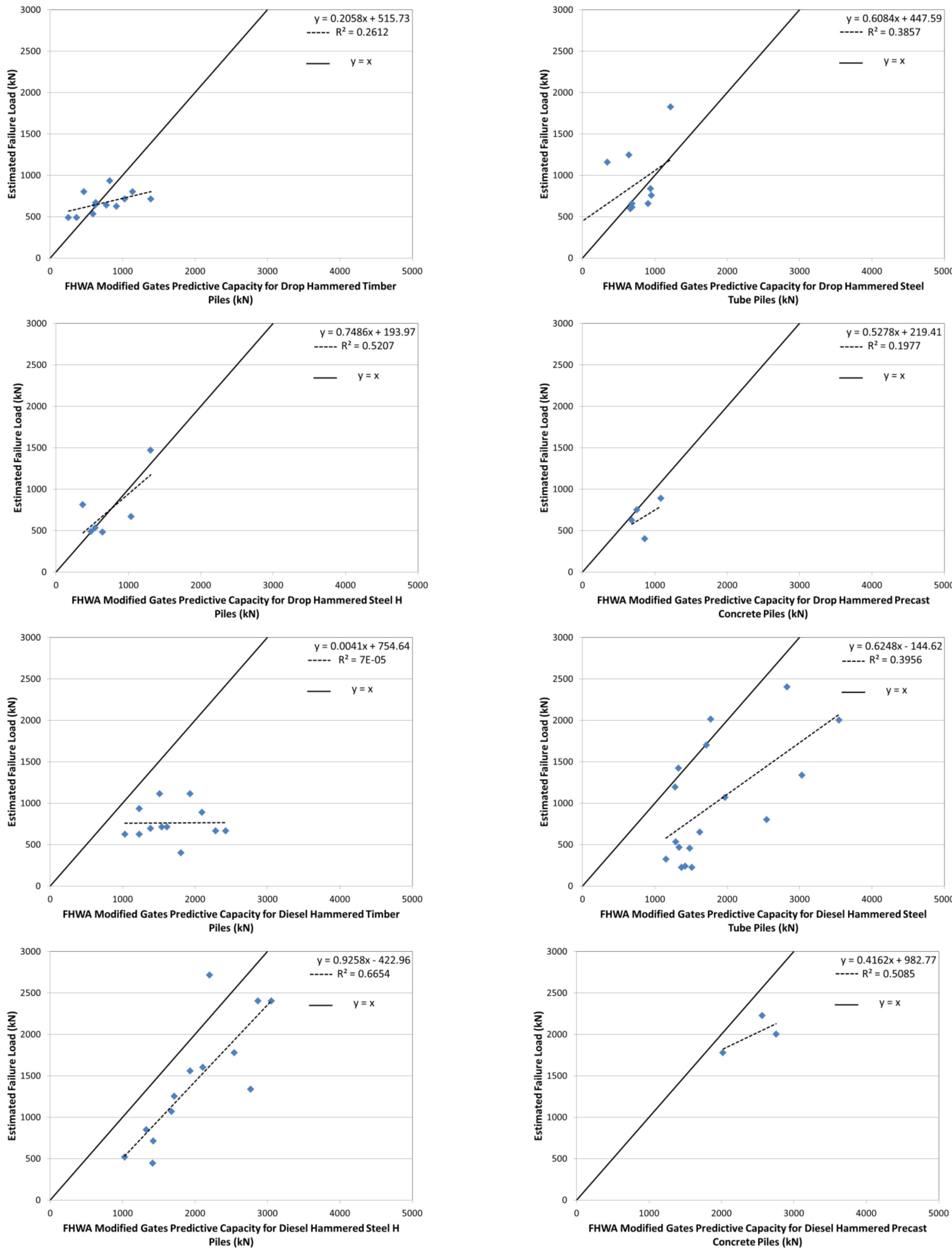


Figure 40: FHWA modified Gates formula predicted capacities vs. pile test failure loads divided by hammer and pile type

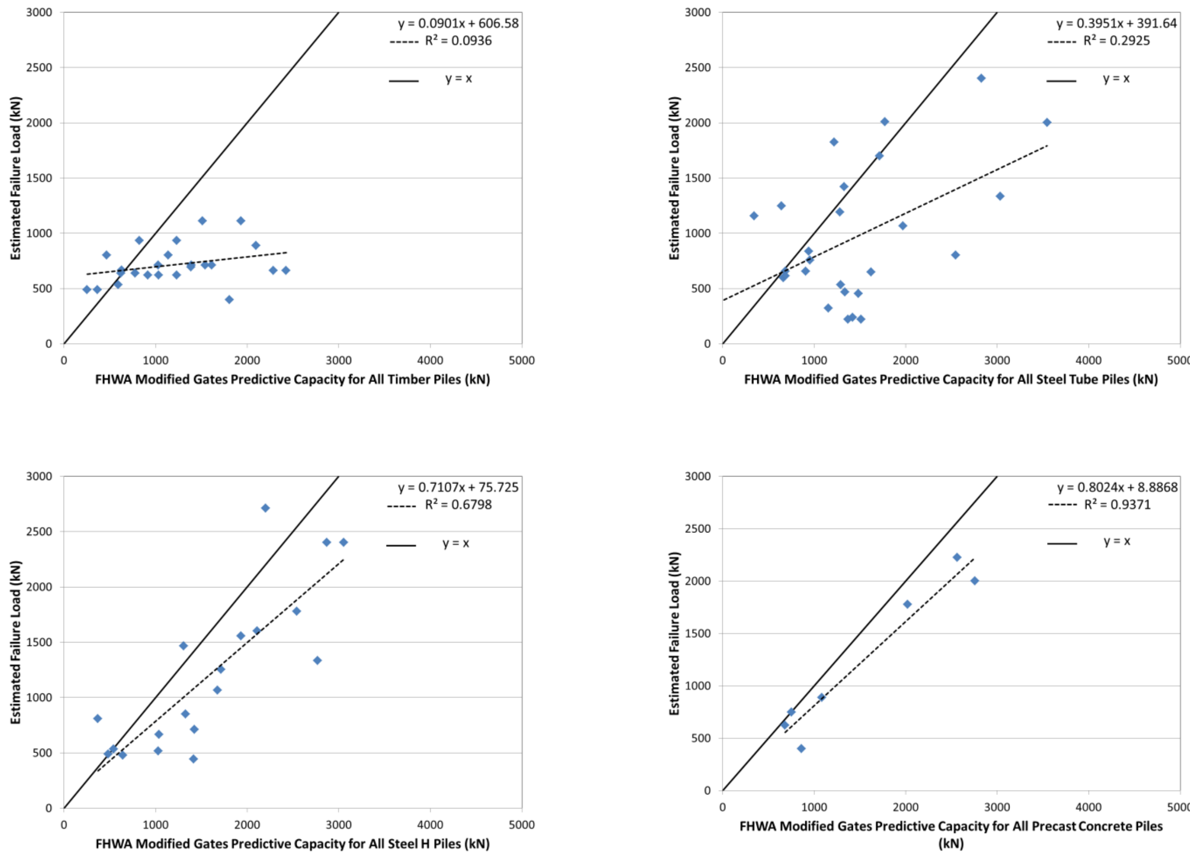


Figure 41: FHWA modified Gates formula predicted capacities vs. pile test failure loads divided by pile type

The FHWA modified Gates formula improves on the predicted values of the Gates formula as is seen by the data points centered around the 1:1 line rather than extending vertically near the y axis. However, the data seems to be biased towards being over predicted by a maximum factor of roughly four but more commonly by a factor of approximately two.

The FHWA modified Gates formula seems to provide a good estimate for drop and diesel hammer installed piles regardless of pile type except for timber piles which once again result in a low coefficient of determination value and whose capacity is grossly over predicted, more so than for other pile types.

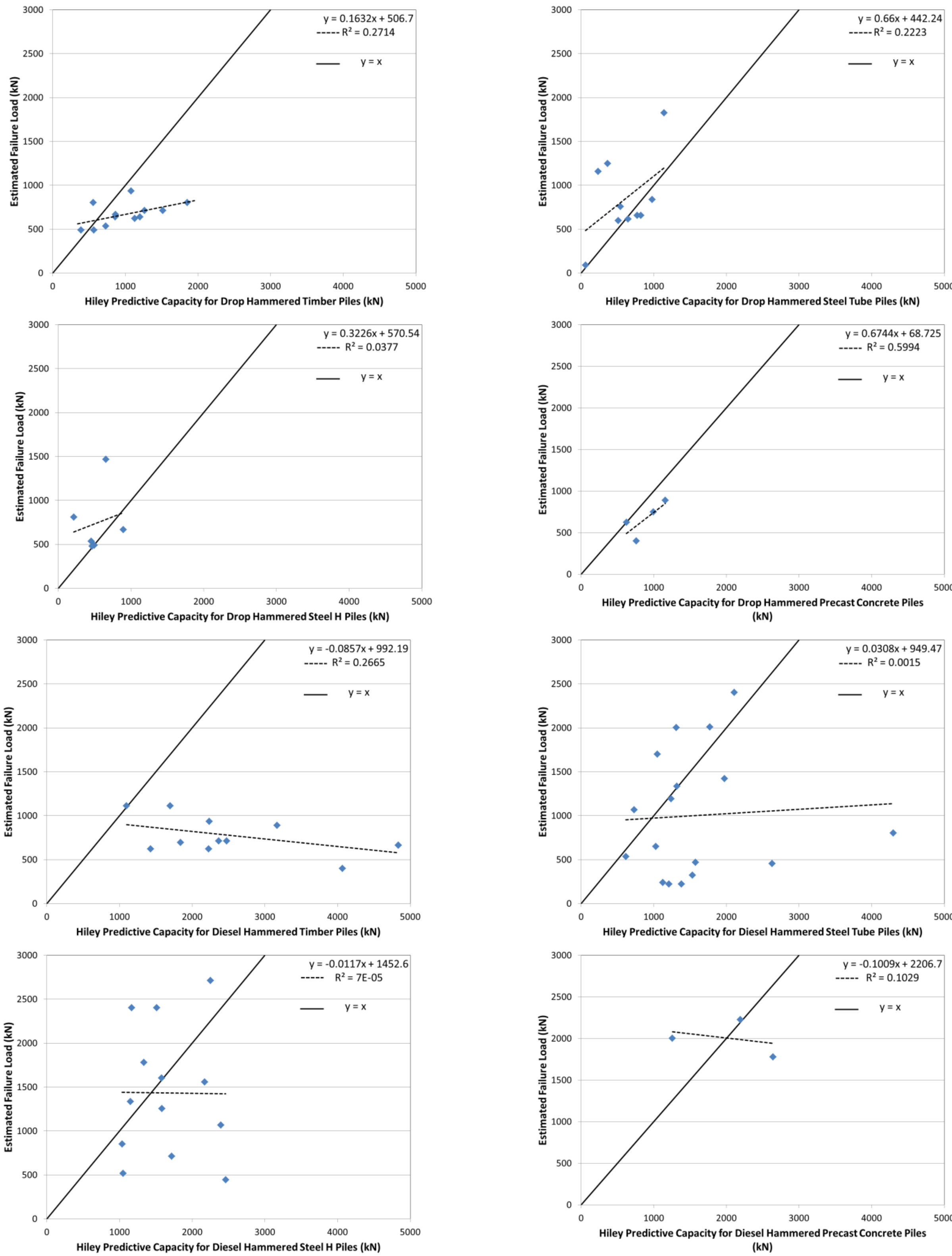


Figure 42: Hiley formula predicted capacities vs. pile test failure loads divided by hammer and pile type

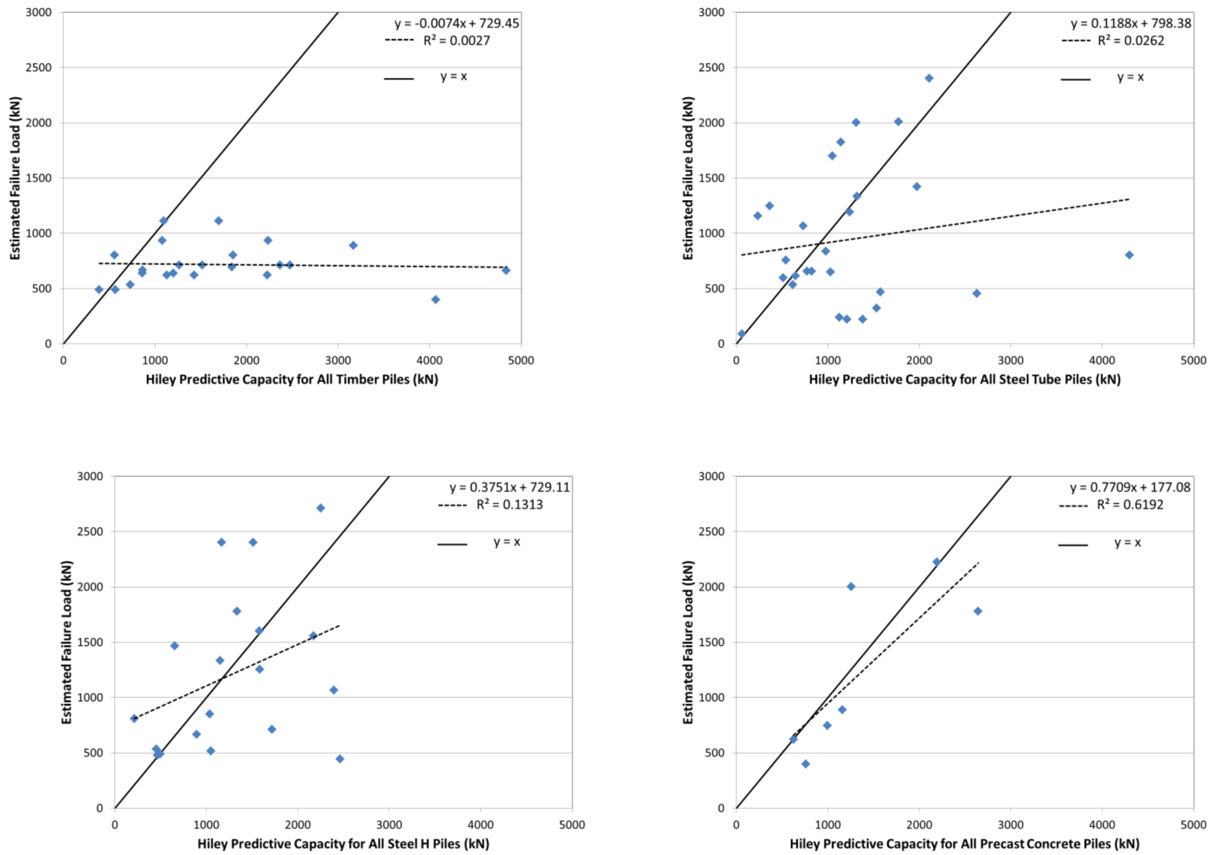


Figure 43: Hiley formula predicted capacities vs. pile test failure loads divided by pile type

From the plots above, the Hiley formula seems to result in generating the most highly variable data points with respect to the values of bearing capacity obtained from field tests. Unlike the other fomrulae mentioned thus far, the Hiley formula seems to be better suited for predicting bearing capacity piles driven by drop hammers rather than those driven by diesel hammers. While examining individual piles types, it is determined that combining both diesel and drop hammered piles does not increase the predictive pile capacity accuracy with the exception of piles composed of precast concrete. The observed increased accuraacy may be due to the limited number of samples and thus increasing them increases the predicted reliability; however, if more samples are added the increased variation may be counter productive and cause the predicted capacities to be as inaccurate as for the other pile types.

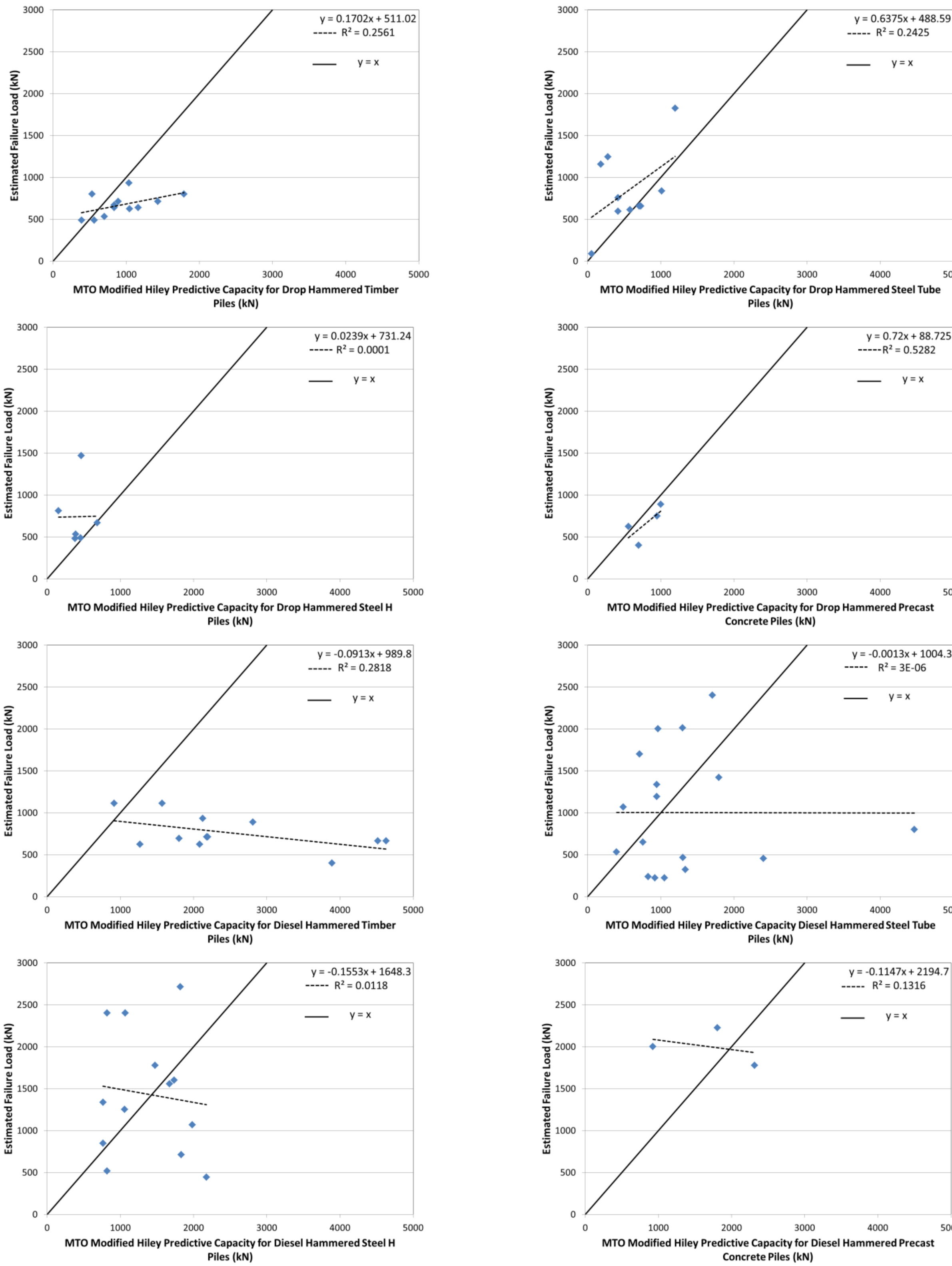


Figure 44: MTO modified Hiley formula predicted capacities vs. pile test failure loads divided by hammer and pile type

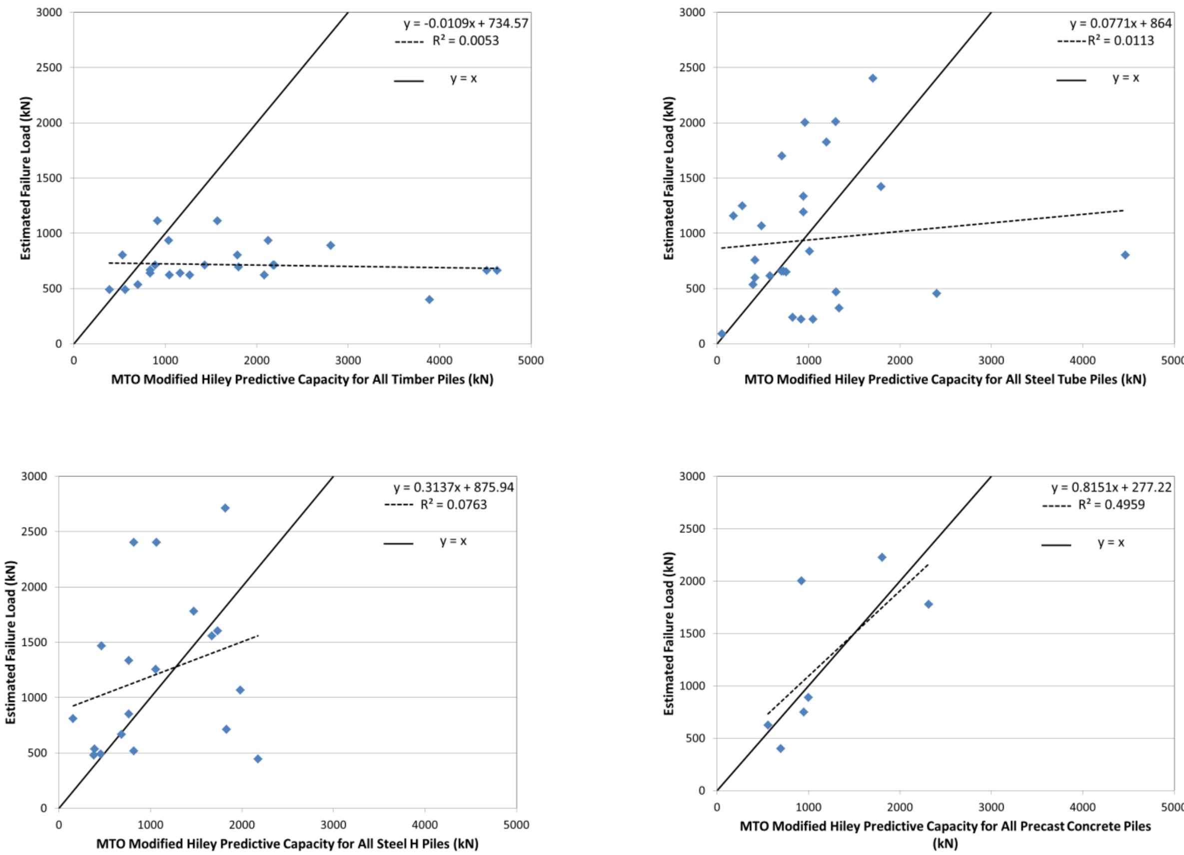


Figure 45: MTO modified Hiley formula predicted capacities vs. pile test failure loads divided by pile type

The predicted capacities derived from the MTO modified Hiley formula exhibits similar trends to those derived from the original Hiley formula, however the variability within the dataset is greater than for the original formula with consistently lower coefficient of determination values. The values derived from the two Hiley formulae similarly over predict the pile capacity by factors as high as nine times the actual value.

To quantify which formulae are best suited to predict pile capacities, at least for the MTO piles examined from southern and central Ontario, a statistical analysis on the results is carried out to determine the values of predicted to measured capacity factors, standard deviations, percent differences, correlations, and is discussed further in section 5.3 below.

5.3 Statistical Analysis

The results of the statistical analysis are summarized in Tables 25 to 27 below and are based upon the 77 piles as a whole as well as separately as the 32 piles installed by drop hammered drivers and the 45 piles installed by diesel hammered drivers.

Table 25: Ratios and Standard Deviations of Dynamic Predicted to Field Estimated Pile Capacities

Hammer	ENR Formula	Gates Formula	FHWA Modified Gates Formula	Hiley Formula	MTO Modified Hiley Formula
Maximum					
Drop	0.381	0.990	2.153	2.179	2.105
Diesel	3.581	1.563	6.820	14.508	13.870
Minimum					
Drop	0.072	-0.191	-6.000	0.201	0.156
Diesel	0.357	0.173	0.812	0.552	0.388
Average					
Drop	0.174	0.331	0.852	1.115	1.013
Diesel	1.279	0.504	2.240	3.057	2.684
All	0.820	0.432	1.663	2.250	1.990

St. Dev.	ENR Formula	Gates Formula	FHWA Modified Gates Formula	Hiley Formula	MTO Modified Hiley Formula
Drop	0.073	0.179	1.315	0.509	0.495
Diesel	0.767	0.324	1.395	2.896	2.730
All	0.802	0.285	1.518	2.427	2.259

Comparing the predicted bearing capacities to the estimated values based on field tests results in the ratios within Table 25. Ideally, a ratio of one indicates a perfect prediction of pile bearing capacity while those greater than one indicates that the bearing capacity is over predicted and those less than one indicates that the bearing capacity is under predicted. Negative values are due to dynamic formulae limitations and indicate a negative bearing capacity value which is not physically possible; for the purposes of analysis they should be ignored except as a tool to aid in defining boundaries within which the formulae are valid.

The ratios spread from a minimum of -6 to a maximum of 14.5 with an average range of 0.4 to 2.3 when considering all piles. While examining piles installed by drop hammers, the MTO modified Hiley formula is the most accurate with an average difference of 0.013 from the ideal case of a predicted to estimated ratio of one. The ENR formula resulted in the most accurate prediction when piles are installed by diesel hammers with an average difference of 0.279. When combining installation methods, the ENR formula provides the closest match of predicted to estimated pile bearing capacities with an average ratio of 0.820.

Considering the standard deviations associated with the ratios of predicted to estimated bearing capacities of each pile installation method; the ENR formula results in the smallest amount of variation for drop hammered piles while the Gates formula results in the smallest amount of variation for piles installed by diesel hammer as well as when examining all driven piles simultaneously.

In addition to examining the ratios of predicted to field estimated pile capacities, the percent difference of predicted to estimated pile capacities are also investigated. In order to ensure that positive percent differences correspond to predicted capacities being greater than field estimated capacities and conversely negative values corresponding to estimated capacities being greater than predicted capacities the following formula is used:

$$Percent\ Difference = \left(\frac{Predicted\ Capacity - Field\ Estimated\ Capacity}{Field\ Estimated\ Capacity}\right) \times 100 \qquad (48)$$

Table 26: Percent Difference and Standard Deviations of Predicated to Field Estimated Capacities

Hammer	ENR Formula	Gates Formula	FHWA Modified Gates Formula	Hiley Formula	MTO Modified Hiley Formula
			Maximum		
Drop	-61.889	-0.977	115.250	117.853	110.487
Diesel	258.108	56.306	581.982	1350.750	1287.000
			Minimum		
Drop	-92.826	-119.101	-700.000	-79.862	-84.356
Diesel	-64.301	-82.713	-18.762	-44.838	-61.241
			Average		
Drop	-82.578	-66.894	-14.841	11.527	1.329
Diesel	27.890	-49.585	123.982	205.715	168.375
All	-18.019	-56.778	66.289	125.013	98.953

St. Dev.	ENR Formula	Gates Formula	FHWA Modified Gates Formula	Hiley Formula	MTO Modified Hiley Formula
Drop	0.073	0.179	1.315	0.509	0.495
Diesel	0.767	0.324	1.395	2.896	2.730
All	0.802	0.285	1.518	2.427	2.259

From Table 26 it is seen that the ENR and Gates formula consistently under predicts drop hammered pile capacity as well as when considering all piles overall. Additionally, the ENR and Gates formulae are the only formulae which on average under predict pile capacity; the FHWA modified Gates, Hiley, and MTO modified Hiley formulae generally over predict pile capacity. The ENR formula, despite having a negative overall percent difference, matches the field estimated value the closest with an average of -18 percent. The Gates and FHWA modified Gates formula under and over predict the estimated pile capacities by approximately the same amount, respectively while the Hiley and MTO modified Hiley formulae over predict the developed pile capacities.

The standard deviations of the percent difference are exactly the same as for the ratios of predicted to estimated pile capacities based upon field testing. A final analysis of determining the correlation between the data sets of predicted and estimated pile capacities is conducted in order to aid in determining which formula best predicts the developed pile bearing capacity. The results are shown in Table 27 below.

Table 27: Correlation of Predicted to Field Tested Results of Pile Capacities

Hammer	ENR Formula	Gates Formula	FHWA modified Gates Formula	Hiley Formula	MTO modified Hiley Formula
Drop	0.333	0.323	0.528	0.209	0.204
Diesel	0.651	0.683	0.685	-0.150	-0.196
All Piles	0.643	0.632	0.662	0.103	0.042

Since an ideal correlation results in a value of one, the FHWA modified Gates formula gives the closest values of dynamic formula predicted to field estimated capacities. For diesel hammered piles and both diesel and drop hammered piles all three; ENR, Gates, and FHWA modified Gates formulae result in the highest and very similar values of correlation; however, the correlation values from the FHWA modified Gates formula are approximately 1.6 times greater than the previously mentioned formulae and 2.5 to 15 times greater than the Hiley and MTO modified Hiley formulae.

From the ratios of predicted to estimated pile capacities, percent differences and correlation values, it is evident that on average the ENR, Gates, and FHWA modified Gates formulae predict pile bearing capacities more accurately than the Hiley and MTO modified Hiley formulae whose data points contain much more scatter as shown by the higher standard deviation and lower coefficient of determination values.

Despite the relatively high correlation, coefficient of determination, low standard deviation, and relatively low percent difference values, the Gates formula is not recommended for predicting pile bearing capacity. This is due to the vertical trending behaviour of the calculated data points. While the field estimated capacities range from 89 kN to 2 713 kN with an average value of approximately 966 kN and a median value of 712 kN, the Gates predictive formula produces predicted capacities with a minimum and maximum value of -17 kN and 707 kN, respectively, an average of approximately 338 kN and a median value of 315 kN. This low upper predicted limit results in a very small spread of data however it also severely under predicts the pile capacity, especially when the actual value is relatively high and thus the amount of error developed increases proportionally.

The Hiley and MTO modified Hiley formulae are not recommended to use since the correlation values, coefficients of determinations, and best fit line slopes are relatively low, the standard deviations and percent differences are relatively high and the ratios of predicted to estimated field capacities are on average 2.2 and 1.9 times greater than unity, respectively.

Since the FHWA modified Gates formula results in best fit lines with slopes closer to unity, higher coefficient of determination values, and produces data points which better predicts pile capacity for drop hammered piles it is recommended for use rather than the ENR formula. However, the ENR formula results in a smaller percent difference, ratios of predicted to estimated field capacities closer to one, smaller standard deviations, and produces data points which better center around the 1:1 line for diesel hammered piles, thus it is recommended to use when diesel hammer drivers advance piles.

No formula is accurate for predicting the developed bearing capacities of timber piles; the only potential exception to this is when using the FHWA modified Gates formula to predict capacities for drop hammered timber piles. However, the predicted results only result in a coefficient of determination of 26 percent and piles with predicted capacities of greater than 1 500 kN are severely over predicted.

To determine if the results from the statistical analysis are typical or not, they are compared to other studies. If the results are largely dissimilar it may provide insight on the cause of the errors which occurred during one of the studies or illustrate the sensitivity to geographic locations or geologic conditions and if similar it adds confidence to the findings from this thesis.

5.3.1 Comparison To WSDOT Studies

To check the validity of the calculated predictive capacities, with regards to other independent findings, the derived values are compared to the results of similar studies performed by the Washington State Department of Transportation (WSDOT) in 1985 and 2005. The studies are commissioned to investigate and determine the accuracy of predicted piles. A 1985 report entitled <u>Development Of Guidelines For Construction Control Of Pile Driving And Estimation Of Pile Capacity</u> by Fragaszy, in which Higgins and Lawton surveyed all U.S. DOTs as well as conducted a literature search on the current state of predicting static bearing capacities for piling sites.

The 1985 report is comprised of a survey of the United States DOTs in regards to their methods of determining installation and quality control procedures when piling. Of all the states questioned, 34 responded and 11 replied that they use the ENR formula to predict pile capacity, while 10 use a modified version of the ENR formulae, five stated that they stopped utilizing the ENR formula in favour of wave equation analysis such as WEAP or TTI programs, while three others use wave equation analysis in conjunction with ENR formulae, two states indicated that they use pile analyzers with ENR formulae and wave analysis to aid in pile predictions and are satisfied with the results. Unfortunately, the majority of states do not explicitly transcribe the form of the ENR formulae which they use or present any data which are used for comparison purposes. However on the basis of prior experience most DOTs state, qualitatively, that the wave analysis if properly calibrated and pile analyzers are used that they provide much more accurate estimates of pile capacity then do dynamic pile driving formulae.

A literature search of comparisons between measured pile load test capacities to those derived from predictive dynamic formulae and wave equation analyses is also completed. The search consists of reviewing data from 10 publications which examines results from sites which installed 5 to 171 piles from across the United States as well as one site in Ontario. Piles are terminated into sand, gravel, clayey silt, and silty sand strata and consist of thin mandrel, fluted steel, timber, precast concrete, H, and closed and open ended tube piles with and without concrete. The pile driving rigs used to install the piles ranged from double acting to differential acting to drop hammers. The pile load tests capacities are compared to the ENR, modified ENR, Eytelwein, modified Eytelwein, Navy-McKay, Canadian National Building Code (CNBC), Pacific Coast Uniform Building Code (PCUBC), Hiley, Gates, Gow, Rabe, Janbu, Danish, modified Danish, and Weisbach dynamic formulae. Although the ENR, Gates, Hiley, and Janbu formulae are used most often in the comparative studies, no single equation is used in all of them.

The task of comparing the predicted values to load tests is made more difficult since certain studies use the allowable capacities from the dynamic formulae while others correct for the intended safety factor (such as 3 and 6 for the Gates and ENR formula, respectively) and use the ultimate bearing capacity as the basis of comparison. The failure loads themselves vary depending upon the method used to calculate them; while all studies base the estimated failure on the load settlement graph of a pile load test, the criteria for failure depends upon the author's method. Some studies set failure load to the load which causes a specific amount of settlement, such as 6.35 to 25.4 millimetres (0.25 to 1 inches) or 10% of the pile width or diameter, while others set the failure load as those which cause a plunging rate to exceed a certain limit, others still use the load which corresponds to the location that the tangents of the steep and horizontal portions of the settlement curve intersect, and finally others calculate the estimated failure load as the average of all the aforementioned methods. Due to the various methods used to calculate the predicted and estimated failure loads the data from the studies cannot be directly compared to each other; however the findings from each study are compared to the others to see if the results are similar.

The literature search suggested that the Hiley, PCUBC, Janbu, Gates, and Danish predictions are the most consistent, that wave equation and pile driver analysis is as or more accurate than dynamic formulae, and that the ENR and Modified ENR formulae are among the least accurate. The only exception to this is the study by Ramey and Hudgins which found that the ENR formula is the most accurate and the Hiley formula the least.

Although the raw data used in the studies is not provided, the predicted to measured ultimate load ratios, safety factors, best fit trend line parameters, and correlation coefficients are provided. These values, given in Tables 26 to 28 below are presented in a format which allows them to be compared to the results from this thesis.

Table 28: Summary of Comparative Studies (after Fragaszy et al, 1985)

Dynamic Formula	Predicted to Actual Bearing Capacity (Safety Factor)	Coefficient of Determination (R^2)
ENR	0.40 – 9.4 Average of 0.964 – 2.89 26 to guarantee SF > 1, 98% of the time	0.012 – 0.689
Gates	1.4 – 2.16	0.085 – 0.740
`Hiley	0.55 – 3.83 Average of 0.92 – 1.4	0.002 – 0.712
Wave Equation	0.80 – 4.04 Average of 2.6	0.526 – 0.863

From Table 25, the ratio of predicted to actual bearing capacity for the ENR, Gates, and Hiley formulae ranged from 0.07 – 3.58, 0.17 – 1.56, and 0.20 – 14.51 with averages of 0.82, 0.43, and 2.25, respectively. The coefficients of determination from Table 22 for the ENR, Gates, and Hiley formula range from 0.111 – 0.424, 0.104 – 0.466, and 0.011 – 0.044, respectively.

Although the lower end values of the predicted to measured bearing capacity ratios from this thesis are less than those of the WSDOT 1985 study, the values are comparable with the higher end and average values within in range given in the older study. The only exceptions to this are the high end and average value from the Hiley and Gates formula respectively, which are lower and higher than the given range by approximately a factor of 3.5.

The coefficients of determination for the ENR and Gates formula fall within the range of the WSDOT report as well; however, the values from the Hiley formula fall in the lower end of the given range, which indicates that the amount of scatter from the results of this thesis are considerably higher than the average from the results of the comparative papers.

Allen (2005) builds upon by load test data by Paikowsky in 1994 and 2004 and compares the pile load test results to predicted values derived from CAPWAP/TEPWAP analysis and the WSDOT dynamic equation. The WSDOT dynamic equation is based upon the original Gates formula but modified to better reflect soil conditions around Washington State.

The study consisted of analyzing 141 individual piles; 118 from 24 of the 50 United States, 10 from Southern Ontario, 6 from China, 3 from Holland, 2 from Israel and 2 from Australia. Of the 141 piles installed, 40 are installed by air/steam hammers, 62 are installed by open ended diesel hammers, 29 are installed by closed ended diesel hammers, 5 are installed by hydraulic hammers, and 3 are installed by drop hammers into a variety of soil types including till, silty sand, silty clay, clay, clayey sand, sand, gravel, and bedrock including limestone and shale. This diversity in hammer, piles, and geology ensure that the results are not biased due to installation method or geographic location.

The report compared the WSDOT, ENR, and FHWA Gates predictive bearing capacities as well as those from CAPWAP analysis to pile load test values and presented the results as Figures 46 to 48.

Unfortunately, only the plots are published and not the actual values of the predicted capacities; however, since the raw data used is presented, the predicted bearing capacity values are calculated using the ENR, Gates, and FHWA modified Gates formulae. The results using the WSDOT pile load test data are then compared to those based on the MTO database to determine if any similarities exist between the data sets and corresponding conclusions.

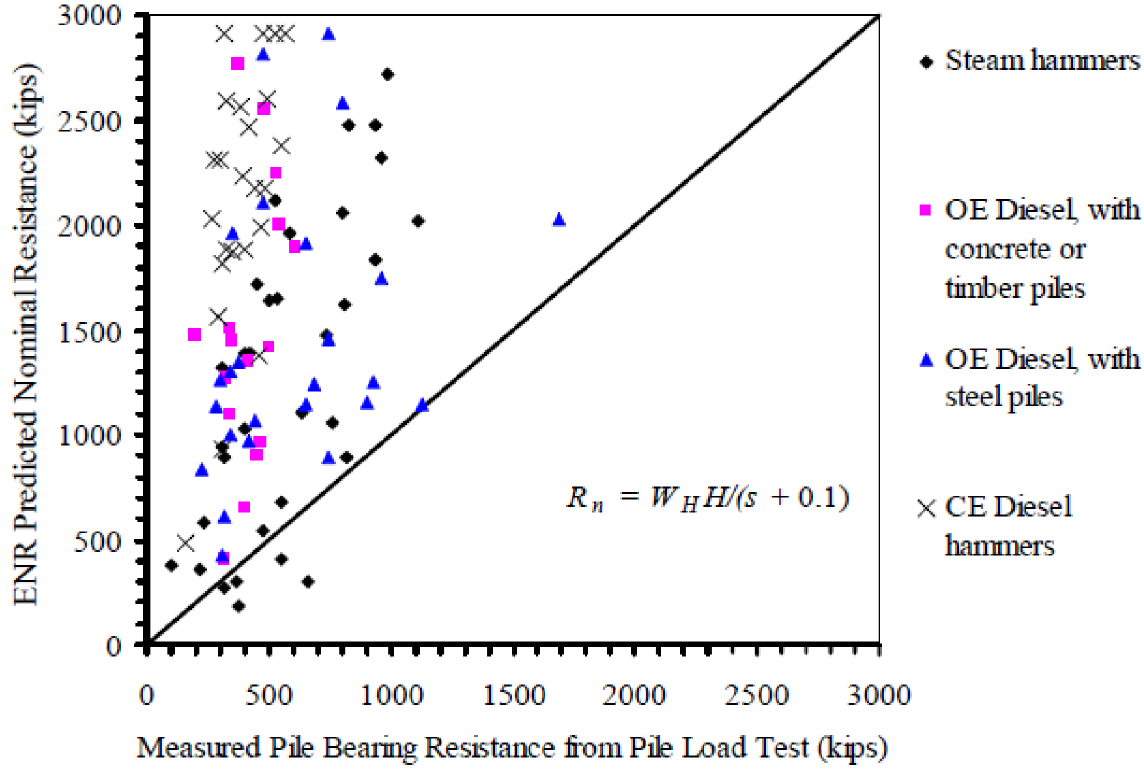


Figure 46: Measured Pile Bearing Capacity Versus ENR Nominal Resistance (Allen, 2005)

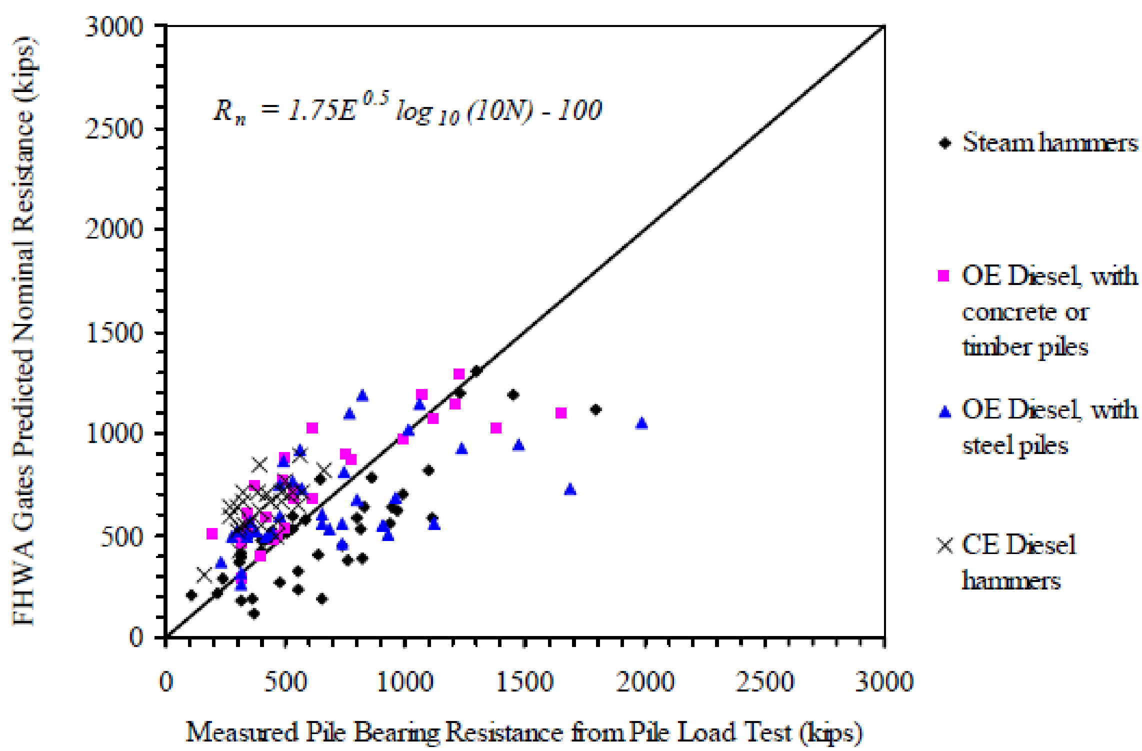


Figure 47: Measured Pile Bearing Capacity Versus FHWA Gates Nominal Resistance (Allen, 2005)

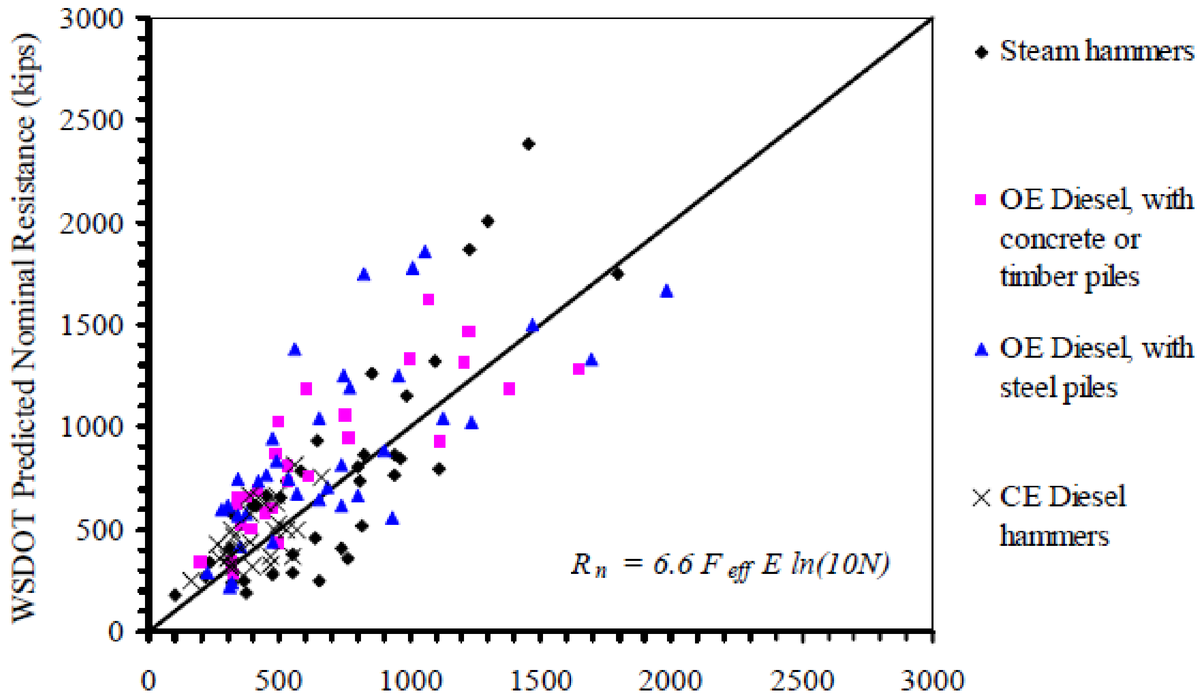


Figure 48: Measured Pile Bearing Capacity Versus WSDOT Nominal Resistance (Allen, 2005)

Table 29: Summary of Predictive Findings using Averaged Values from the WSDOT Database

Dynamic Equation	ENR Formula	Gates Formula	FHWA Modified Gates Formula
Equation of Best Fit Line	y = 0.7409x + 1365.4	y = 6.0757x – 393.31	y = 1.0771x + 86.074
Coefficient of Determination	0.4420	0.4422	0.4443
Predicted to Measured Capacity Ratio	0.731	0.225	1.056
Standard Deviation	0.392	0.095	0.438
Correlation	0.665	0.665	0.667
Percent Difference	-26.872	-77.469	5.596

From Table 29 it is seen that in general the ENR and Gates formulae under predict pile capacity while on average, the FHWA modified Gates formula predicts the measured pile capacity very accurately when using the WSDOT database. Comparing Tables 24 to 27 with Table 29 it is seen that the trends are similar for the predictive capacities based on the MTO database; where the ENR formula over predicts capacities for piles which support more than 1 200 kN loads and under predicts capacities for loads less than 1 200 kN, the Gates formula similarly extremely under predicts pile capacity, and on average the FHWA modified Gates formula over predicts pile capacity.

It is also observed that the MTO database results in similar values of coefficients of determination; the values from the MTO database range from 0.400 to 0.438 while using data from the WSDOT paper range from 0.442 to 0.444. Likewise, the correlation values are also similar ranging from 0.632 to 0.662 and 0.665 to 0.667 for the MTO and WSDOT datasets, respectively. The average ratio of predicted to pile load tested bearing capacities are also similar between the two datasets; for the ENR, Gates, FHWA modified Gates formulae the values are 0.820, 0.432, 1.663, and 0.731, 0.225, 1.056, for the MTO and WSDOT calculated capacities, respectively. The percent difference between the predicted and field estimated pile bearing capacities for the ENR, Gates, and FHWA modified Gates formula are -18.019, -56.778, 66.289, and -26.872, -77.469, 5.596 for MTO and WSDOT data, respectively. The percent difference between the MTO and WSDOT values differ by a factor of approximately 1.4 for the ENR and Gates formulae while the FHWA modified Gates formula values differ by a factor of 11, however the trends remain similar for both datasets; under and over predicted bearing values from the different formulae remain under and over predicted respectively despite the database being used.

Both, the 1985 and 2005 WSDOT reports as well as the 1993 MTO report conducted comparisons of predicted pile capacity using wave equation analysis programs such as GRLWEAP and signal matching programs such as CAPWAP with pile driving analyzers.

The literature search from the 1985 WSDOT report contained three studies, ranging from analysis based on 5 to 78 pile load tests, two of which used wave equation analysis and the other which used the signal matching methodology. The 2005 WSDOT report compared CAPWAP/TEPWAP analysis performed on 136 piles to pile load test results while the MTO report compared wave equation and CAPWAP analysis to 22 and 12 pile load test results, respectively. The summary of the findings are presented in Table 30 below.

Table 30: Summary of Wave Equation and Signal Matching Analysis

Study	Agerschou	Ramey and Hudgins	Kazmierowski and Devata	2005 WSDOT Report	1993 MTO Report	
Analysis	Wave Equation	Wave Equation	Pile Analyzer	CAPWAP Analysis	Wave Equation	CAPWAP
Equation of Best Fit Line			y = 0.7134x + 257.05	y = 0.9667x + 1014.9	y = 0.611x + 470.1	y = 0.8308x + 358.36
Coefficient of Determination			0.6931	0.5296	0.7846	0.8955
Predicted to Measured Capacity Ratio	0.385		1.062	0.712	0.995	0.877
Standard Deviation	0.23	0.065 – 0.341	0.339	0.318	0.327	0.196
Correlation		0.725 – 0.929	0.833	0.728	0.886	0.946
Percent Difference			6.186	-28.751	-0.451	-12.339

The Agerschou paper from the 1985 WSDOT paper implies that the ratio of predicted to measured pile capacity is comparatively low compared to the wave equation analysis of the MTO report and analyses using pile analyzers. This may be due to the fact that the examinations performed are during the infancy of wave equation analysis in 1962. Later analysis, performed from the mid-1970s to early 1980s, shows that predictions using both wave equation software with signal matching programs are much more accurate with ratios which range from 0.712 to 1.062. The cause for which may be that as experience is gained and computer code is refined both computationally as well as the values for soil and hammer properties used, the predictive accuracy increases.

The standard deviation in the data remained relatively constant from 19.6 to 34.1 percent; the only exception being the value from Ramey and Hudgins for concrete piles, which is 6.5 percent. This is most likely caused by the low number of piles examined, six, as well as the high correlation value of 92.9 percent reported. The correlation and corresponding coefficients of determination values from the other studies are consistent from 72.8 to 94.6 and 53.0 to 89.6 percent, respectively. The correlation of predicted to measured bearing capacity from the other analyses ranges from 72.8 to 94.6 percent with coefficients of determination on the order of 0.530 to 0.895.

For wave equation analysis, the percent difference could only be calculated for the MTO study and is determined as -0.451 percent. The percent difference from pile analyzer data ranges from -28.751 to 6.186 percent. From the ratio and percent difference data it is seen that in general that the wave analysis and signal matching techniques result in bearing capacities being over predicted except at relatively low capacities, on the order of 1 000 kN or less.

Comparing the pile capacity predictions from dynamic formulae and computer programs using the MTO database to those of the WSDOT reports show that the results are similar and thus the trends observed are typical despite geographic location, geology embedded into, driving equipment, and material types used. This gives confidence to the findings; however, with the correlation values and coefficients of determination being relatively low it is decided to attempt to improve on the predictive capacities which the dynamic formulae gives in order to increase accuracy and diminish the amount of scatter within the dataset. This is accomplished by adjusting the coefficients in the formulae and analyzing the resulting predictive values.

For the raw calculations of predicted bearing capacity from dynamic formulae using the WDSOT piles, please see Appendix D.

5.3.2 Piles Bearing Predictions From Revised Coefficients

To aid in potentially improving and confirming which dynamic formula is the most accurate for predicting pile capacity, an analysis of the constants used in each pile driving formula, drop and diesel hammer, is conducted in which the parameters are varied to determine if revised values can be ascertained.

The parameters adjusted are the c coefficient of the ENR formula, the $^1/_7$ coefficient at the beginning of the Gates formula and the 1.75 and 100 coefficients at the beginning and end of the FHWA modified Gates formula. The Hiley and MTO modified Hiley formulae describe the amount of compression that the pile soil system undergoes during driving by the C coefficient. For the Hiley formula, the coefficient is the sum of the compression of the pile cap and head (C_1), pile (C_2), and soil (C_3) as given by Tables 16 to 18; for the MTO modified Hiley formula the C coefficient is measured in the field. Since this data is not available, the C coefficients are assumed equal between the two formulae. As a result, for the analysis, only the MTO modified Hiley formula is analysed with respect to varying the soil – pile system compression values.

The analysis consisted of determining which coefficient values are required, on a case by case basis, to result in the dynamic formulae predictions being identical to the field test estimated values, whenever possible. The findings are summarized in Table 31 below.

Table 31: Summary of Revised Dynamic Formulae Coefficients

Formula	ENR		Gates		FHWA Modified Gates				MTO Modified Hiley	
	Drop	Diesel	Drop	Diesel	Drop		Diesel		Drop	Diesel
Original Value	**1**	**0.1**	**$^1/_7$**		**1.75**	**100**	**1.75**	**100**	**See Tables 16 to 18**	
Revised Values										
Minimum Value	0.01	0.00	0.90	1.21	1.10	1	0.60	46	2.0	0.8
Maximum Value	0.13	0.75	4.31	10.90	3.55	253	2.09	496	34.5	129
Average Value	**0.05**	**0.21**	**2.29**	**3.53**	**1.80**	**119**	**1.17**	**276**	**16.2**	**39.7**
Standard Deviation	0.05	0.19	0.69	2.27	0.52	0.37	54	111	10.5	33.5

The above values are based on matching the predicted to field estimated pile capacities for the 77 piles used in the study. However, due to the nature of the formulae, when using the ENR formula to predict bearing capacities only 39 pile records could be matched to pile load test results. When considering the Gates predictive formula, as well the first coefficient of the FHWA modified Gates formula only 76 pile records could be matched to field tested values. Varying the second coefficient of the FHWA modified Gates formula could result in only matching 72 pile records to pile load test results. Revised coefficients of the MTO modified Hiley formula could match 66 pile records to the field estimated capacities. The reasons for which the piles could not be matched to field values via revised predictive formulae is that the coefficients required would not make physical sense, such as the coefficient having a negative value or that as the coefficient becomes increasingly larger the predicted capacity approaches a limiting value and never reaches the field estimated value.

For the purposes of the analysis the average values in Table 31 are used, omitting the piles which could not be matched to the estimated bearing capacity from pile load tests. The predictive capacities are then recalculated using the average value as the revised coefficients, in bold. The results are graphed and shown in Figures 49 to 53 and summarized in Table 32 below.

Table 32: Summary of Best Fit Lines and Coefficient of Determinations of Revised Pile Capacities

Hammer	ENR Formula	Gates Formula	1.75 FHWA Modified Gates Formula	100 FHWA Modified Gates Formula	MTO Modified Formula
All Piles	y = 1.075x + 322.65	y = 1.7129x – 333.02	y = 0.9118x + 86.365	y = 0.7145x + 316.55	y = 0.5044x + 608.45
	R^2 = 0.4930	R^2 = 0.4292	R^2 = 0.4556	R^2 = 0.4867	R^2 = 0.0898
Drop	y = 0.5966x + 526.69	y = 0.8014x + 195.64	y = 0.4613x + 390.25	y = 0.4587x + 442.55	y = 0.3032x + 562.73
	R^2 = 0.2176	R^2 = 0.2758	R^2 = 0.2846	R^2 = 0.2706	R^2 = 0.0719
Diesel	y = 1.3081x + 117.15	y = 2.1148x - 599.59	y = 1.1074x - 107.26	y = 0.7431x + 312.27	y = 0.388x + 816.89
	R^2 = 0.5062	R^2 = 0.4660	R^2 = 0.4661	R^2 = 0.4662	R^2 = 0.0432

Comparing Figures 31 to 35 to Figures 49 to 53, the revised predictions using the new c values to that of the original ones, it is concluded that for the ENR formula, the revised values of 0.05 and 0.21 rather than 1 and 0.1 for drop and diesel hammers, respectively, results in less scatter and trend lines closer to the 45^{o} line, especially for piles installed by drop hammer as well as higher coefficient of determination values. However, in contrast to the original ENR predictions, the calculations become more conservative and bearing capacities become under predicted using the revised average c coefficient from Table 29.

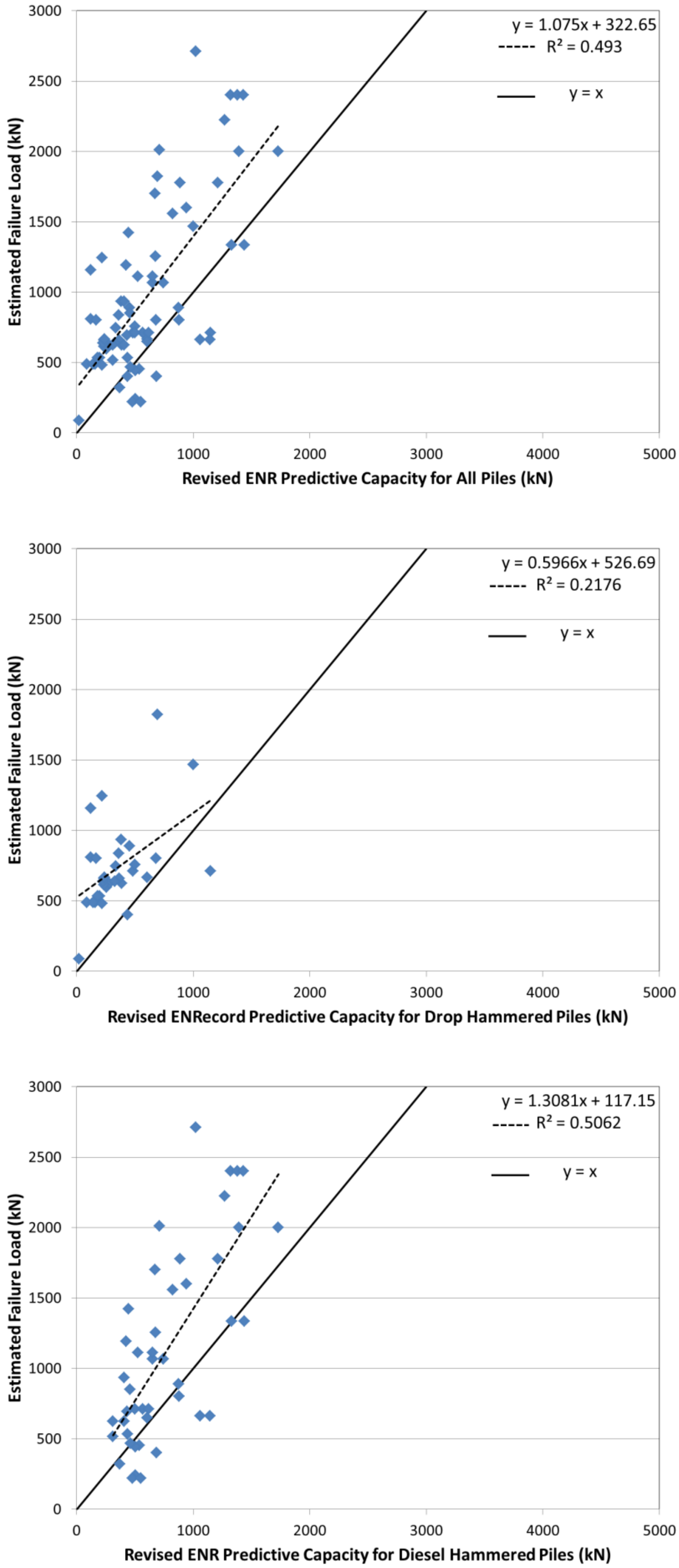
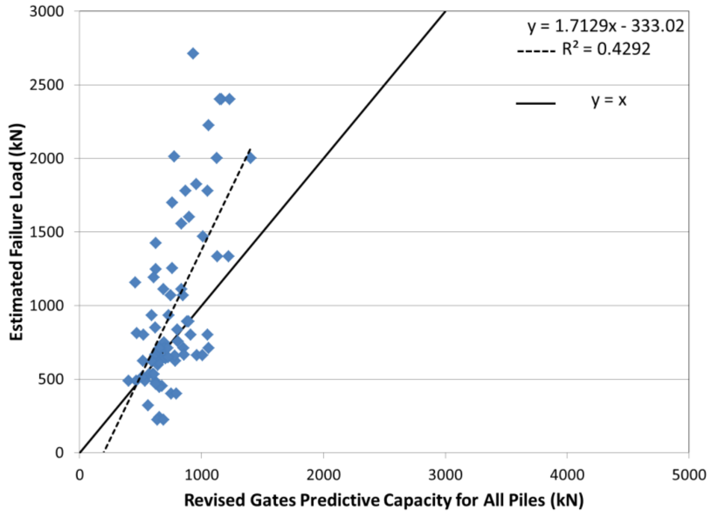
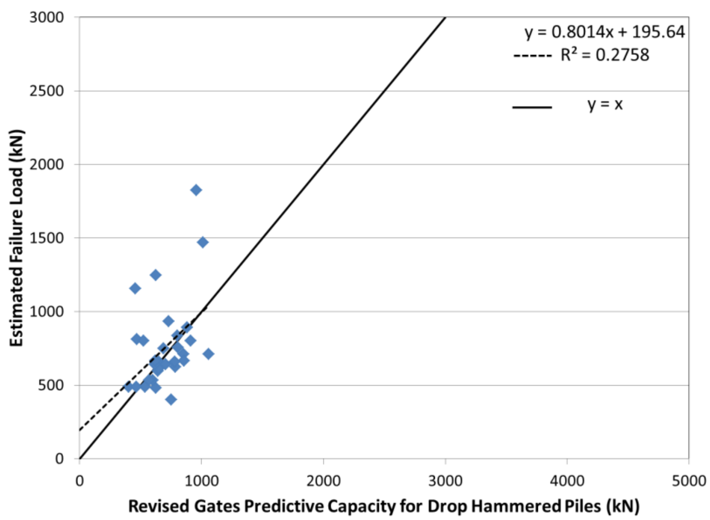


Figure 49: Revised c coefficient ENR formula predicted capacities vs. pile test failure loads





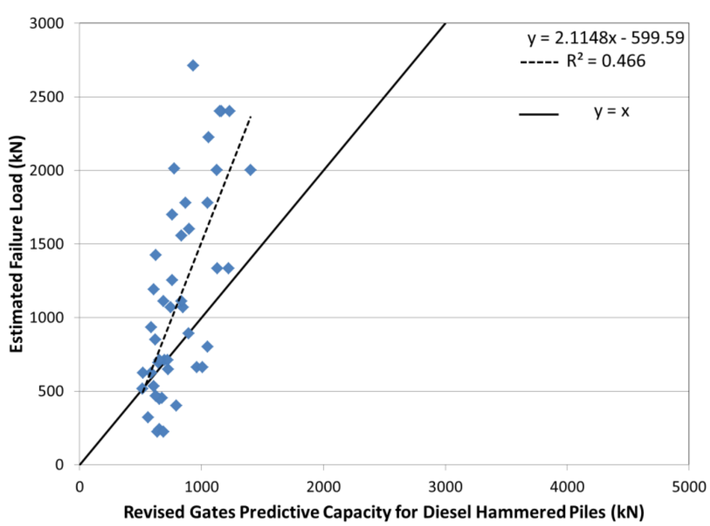


Figure 50: Revised $^{1}/_{7}$ coefficient Gates formula predicted capacities vs. pile test failure loads

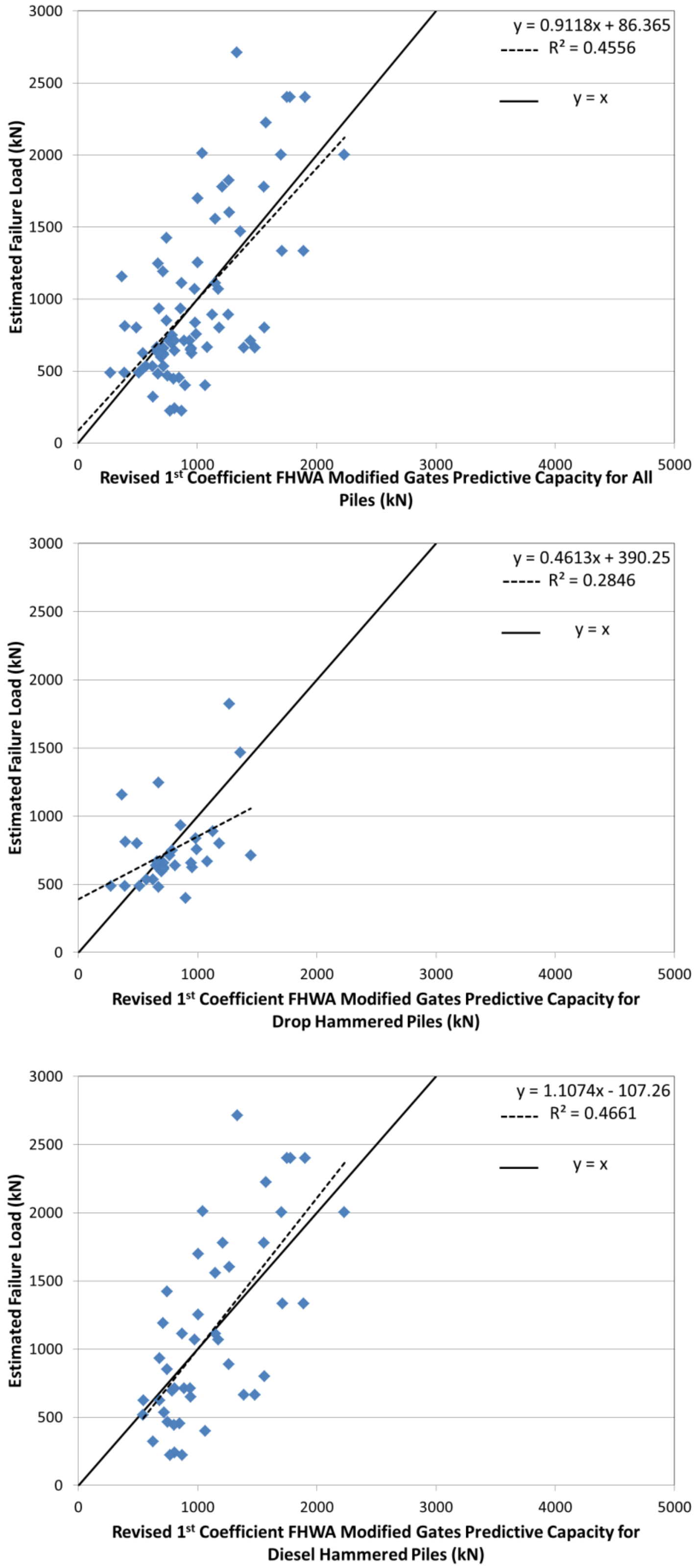


Figure 51: Revised 1.75 coefficient FHWA modified Gates formula predicted capacities vs. pile test failure loads

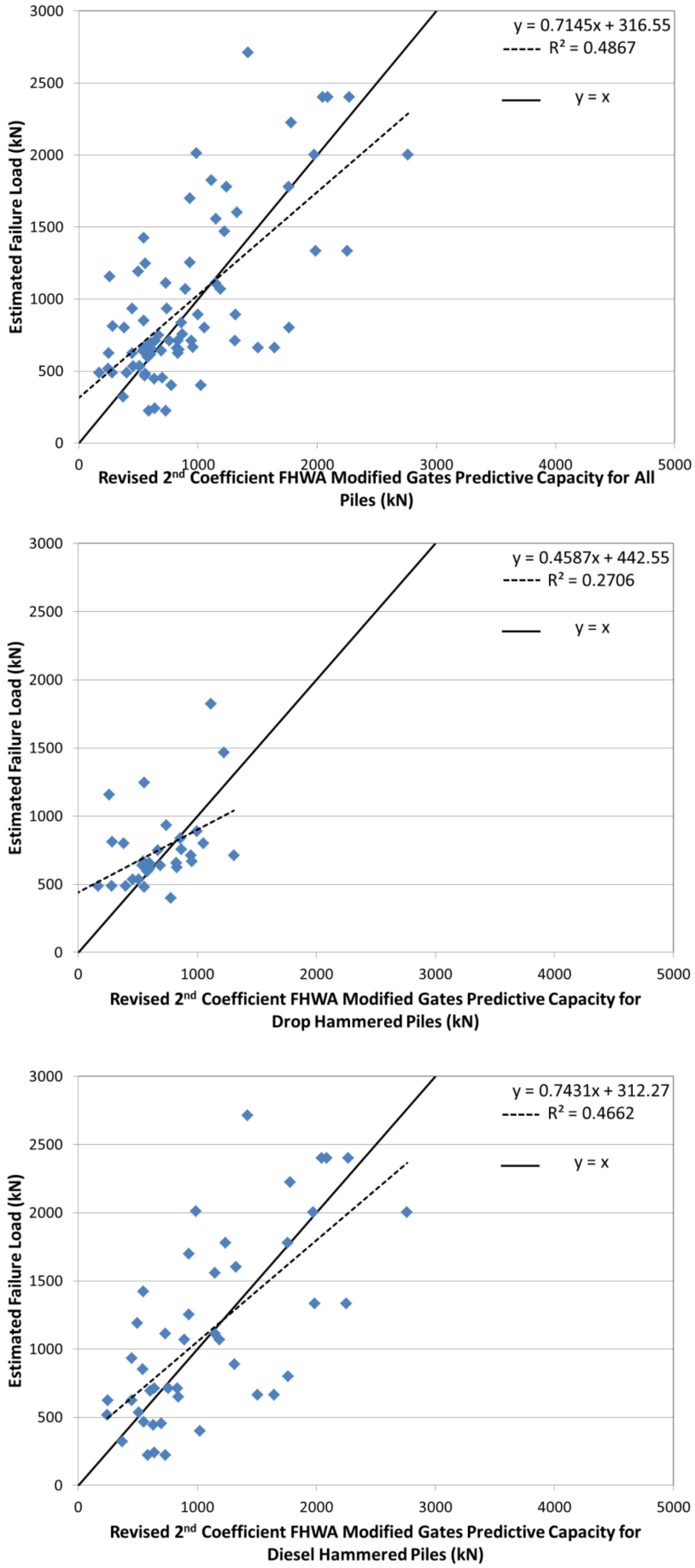


Figure 52: Revised 100 coefficient FHWA modified Gates formula predicted capacities vs. pile test failure loads

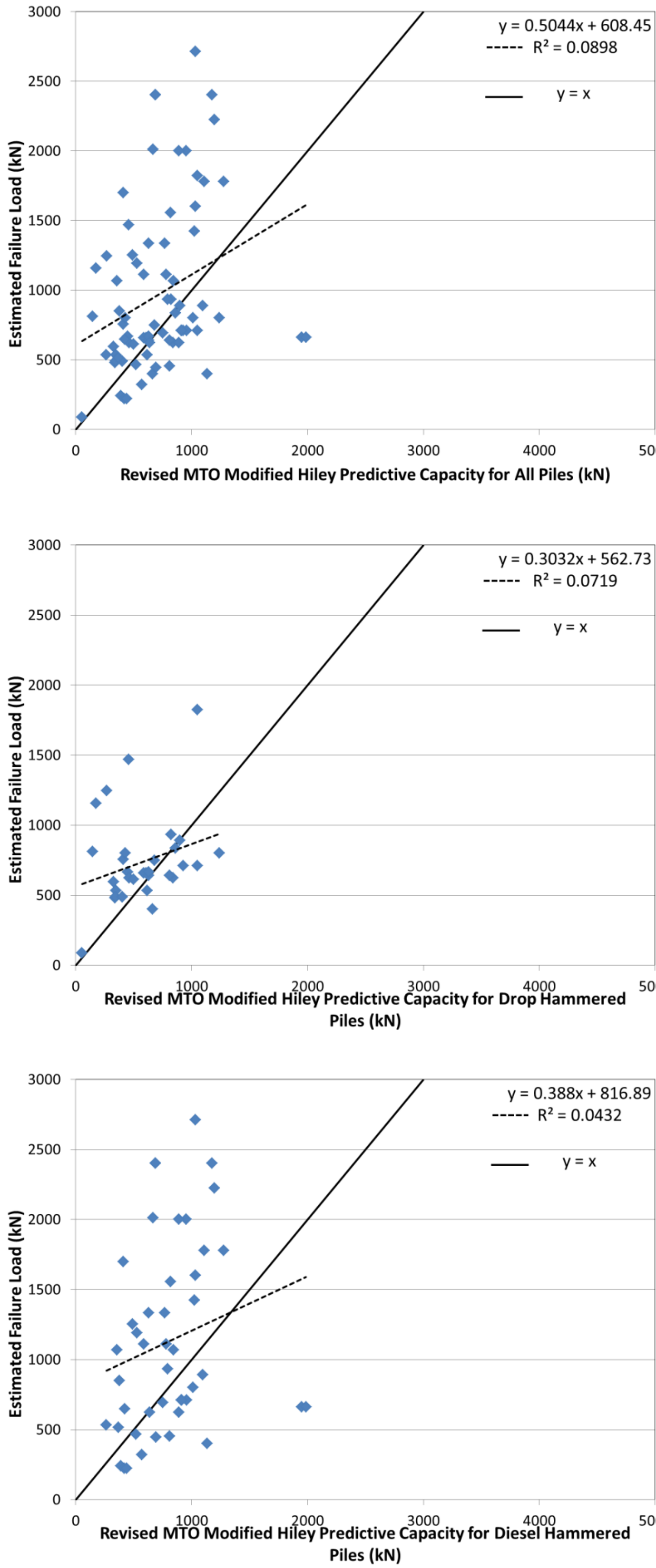


Figure 53: Revised c coefficient MTO modified Hiley formula predicted capacities vs. pile test failure loads

Applying the revised average coefficients of 2.29 and 3.53, for drop and diesel hammers respectively, rather than the original $^{1}/_{7}$ at the beginning of the Gates formula yields similar results, with a decrease in scatter and thus an increase in the coefficient of determination and best fit lines which match up closer to the 45^{o} line. However the data points still appears to trend vertically, thus as high capacity piles are used or as actual developed capacities increase so do the differences between it and the predicted values.

Whereas the original FHWA predicted values tend to overestimate the pile capacities, the revised predictions using values of 1.80 for drop hammer installed piles and 1.17 for diesel hammer installed piles rather than 1.75 seem to centre about the 1:1 line more accurately. Even though the revised data's best fit lines match the 45^{o} line better than the original predicted values, the amount of scatter is approximately the same. Varying the second coefficient of 100 to 119 and 276 for drop and diesel hammer drivers, respectively, results in similar values to those due from changing the 1.75 coefficient. However, the best fit lines from the revised 100 coefficient predictions exhibit a greater difference from the 1:1 line than does the best fit line derived from data points calculated using the revised 1.75 coefficient values. Similarly, the data points from the revised 100 coefficient formula result in a larger spread of predicted capacity data than that from the revised 1.75 coefficient value.

Since all the parameters of the MTO modified Hiley formula are predetermined, either by the MTO guidelines, pile driver manufacturers published specifications or the pile itself the only coefficient which can be altered is the C coefficient, the rebound of the pile due to a single hammer blow. Ideally, this amount of rebound is measured in the field and depends on the type of pile being used, the equipment and method of installation, and the soil conditions. Since these factors vary from site to site, the coefficient is different for each pile installation, therefore using an average value for all piles sites does not seem justified, yet it is useful to determine the amount of influence the C factor produces on the final pile prediction. If a single value causes varying degrees of difference in predicted capacities; for example if the revised value causes a large change in the predicted capacity from the original for some piles and little to no change in others then the capacity is assumed to be sensitive to the coefficient and as such it should be measured for each site for each pile type.

However, if the changes in revised capacities are all large or all small, by the same amount for all piles then the predicted bearing capacity can be deemed as is insensitive in regards to the C coefficient and possibly cause one value to accurately estimate actual developed pile capacity. Since, in this study, the C coefficient of the MTO modified Hiley formula is taken as the sum of all the c coefficients of the

Hiley formula, only the MTO formula is considered and it assumes that the results apply to the original Hiley formula in the same manner.

Using the average C coefficient values of 16.2 and 39.7 for drop and diesel hammer installed piles, respectively; it is seen from Figure 53 that the data points plot much more closely to the 1:1 line than in Figure 35. This is especially true for diesel hammered piles rather than drop hammered piles and is most likely due to the C coefficient being more than twice as large for diesel hammer drivers than drop hammer drivers. However, since the predicted capacities for drop hammered piles seem to centre about the 1:1 line, the chosen coefficient value appears to be appropriate. Despite the improvements in the coefficient of determination values; an increase by a factor of 1.1, 1.7, and almost 50 when considering diesel hammered piles, drop hammered piles, and all installed piles, respectively, and they are still relatively low in the range of 4% to 9% indicating that the amount of variation within the dataset is still high.

Similarly, as with the original predicted capacities the revised coefficient predictions are also subdivided by pile type and installation method to determine if the adjusted coefficients better predicted one pile composition, driver used or combination thereof. The graphs are presented below as Figures 54 to 63.

Comparing the below 12 figure to those of the original ENR formula predicted capacities it is seen that points which represent drop hammered piles no longer trend vertically but follow more closely to the 1:1 line, however they still under predict the estimated failure load capacity. The revised diesel hammered piles which previously centers about the 1:1 line are now also under predicted. In general, the coefficients of determination are increased by a factor of approximately one to more than five, the only exceptions being for drop hammered concrete piles which decreased from 49 percent to less than 1 percent and diesel hammered timber piles which increase by a factor of 880 but still has an absolute value of less than 1 percent.

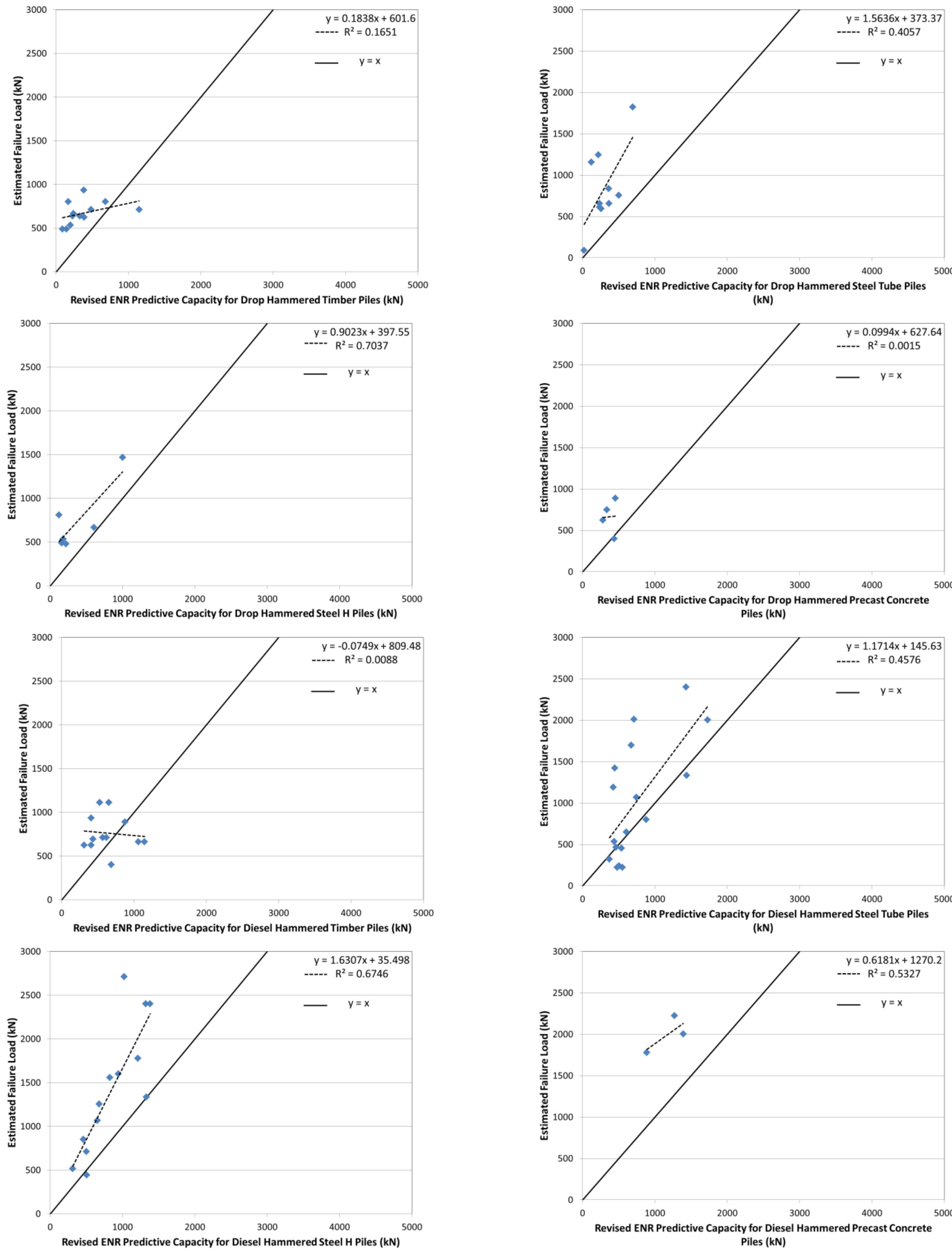


Figure 54: Revised ENR formula predicted capacities vs. pile test failure loads divided by hammer and pile type

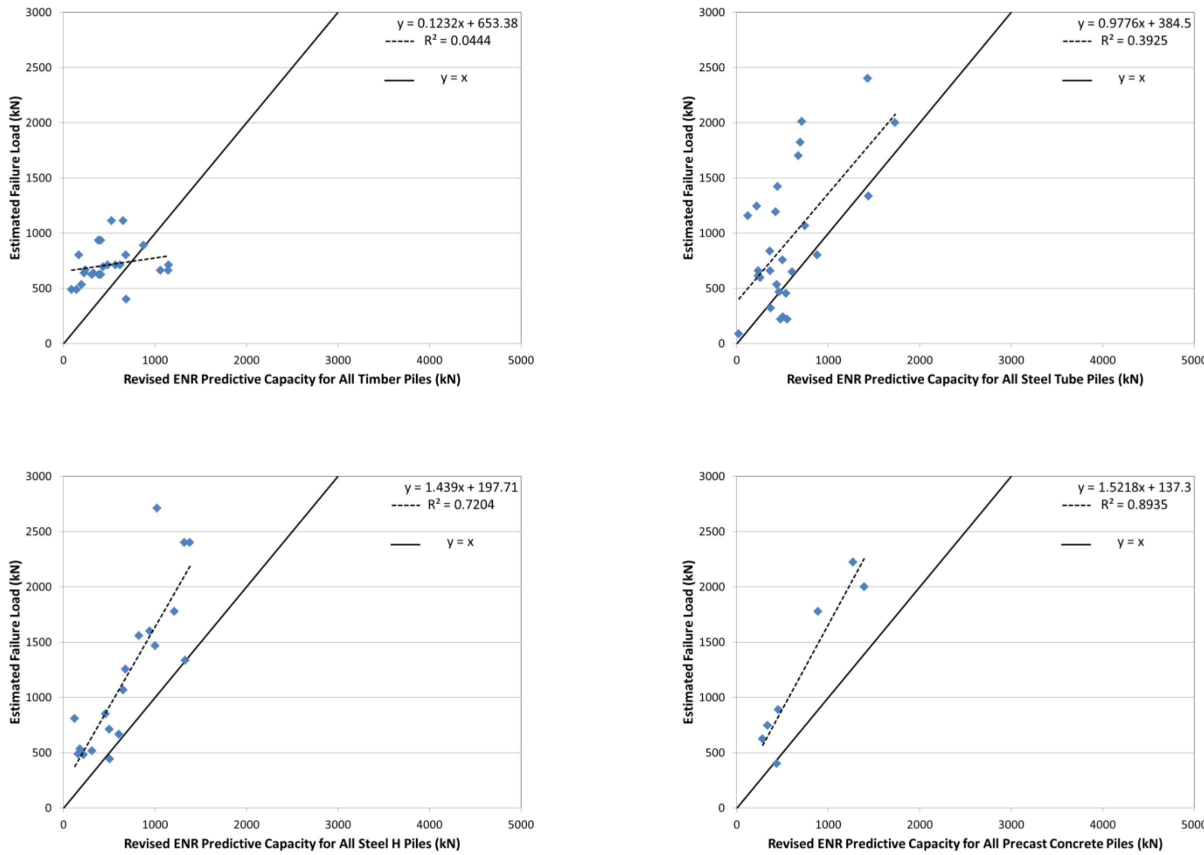


Figure 55: Revised Engineering News Record formula predicted capacities vs. pile test failure loads divided by pile type

When comparing the revised predicted values to the original ones by pile type, the coefficients of determination are similar, differences range from factors of 0.87 to 1.5, being lower for timber and concrete piles and higher for H and steel tube piles. Similarly the slopes of the best fit lines follow the same pattern.

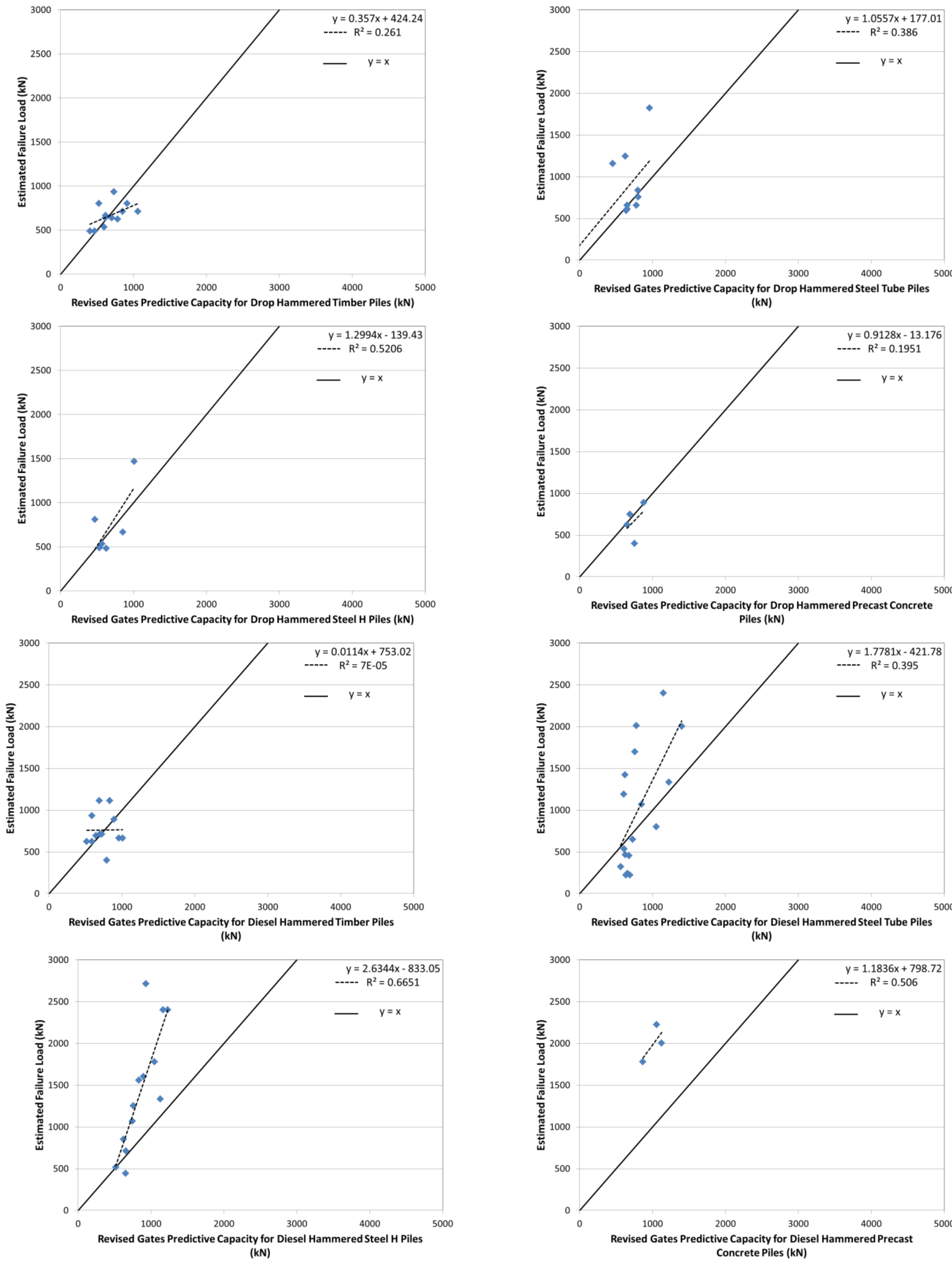


Figure 56: Revised Gates formula predicted capacities vs. pile test failure loads divided by hammer and pile type

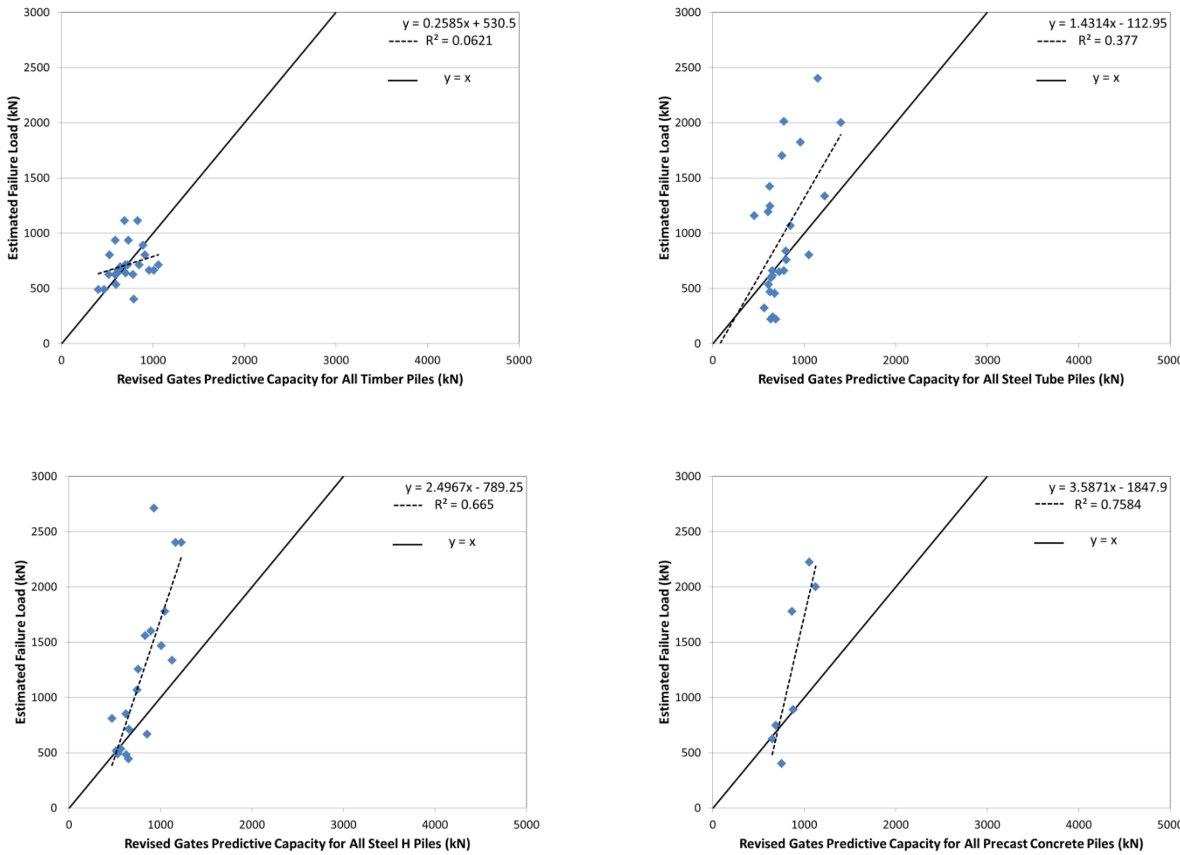


Figure 57: Revised Gates formula predicted capacities vs. pile test failure loads divided by pile type

When comparing the revised predicted pile capacities to original ones, by installation method and pile type, it is seen that the for drop hammered piles, the data points centre around the 1:1 line rather than trending vertically as previously calculated. When examining diesel hammered piles, the data points still appear to trend vertically however they are shifted to the right towards the 1:1 line. Using the revised coefficients causes the slopes of the best fit lines to approach a value closer to unity for all pile types, except timber piles. Despite the coefficients used, original or revised, the coefficients of determination remains similar for all piles whether installed by either drop or diesel hammer drivers.

When considering predictive accuracy solely on the basis of pile type, the revised coefficients results in best fit lines with slopes closer to unity than for the original Gates formula, except for timber piles which decreases by approximately a factor of two even though the data points center around the 1:1 line. The coefficients of determination decreases for timber and concrete piles, increases for concrete filled steel tube piles, and remain approximately constant for H piles.

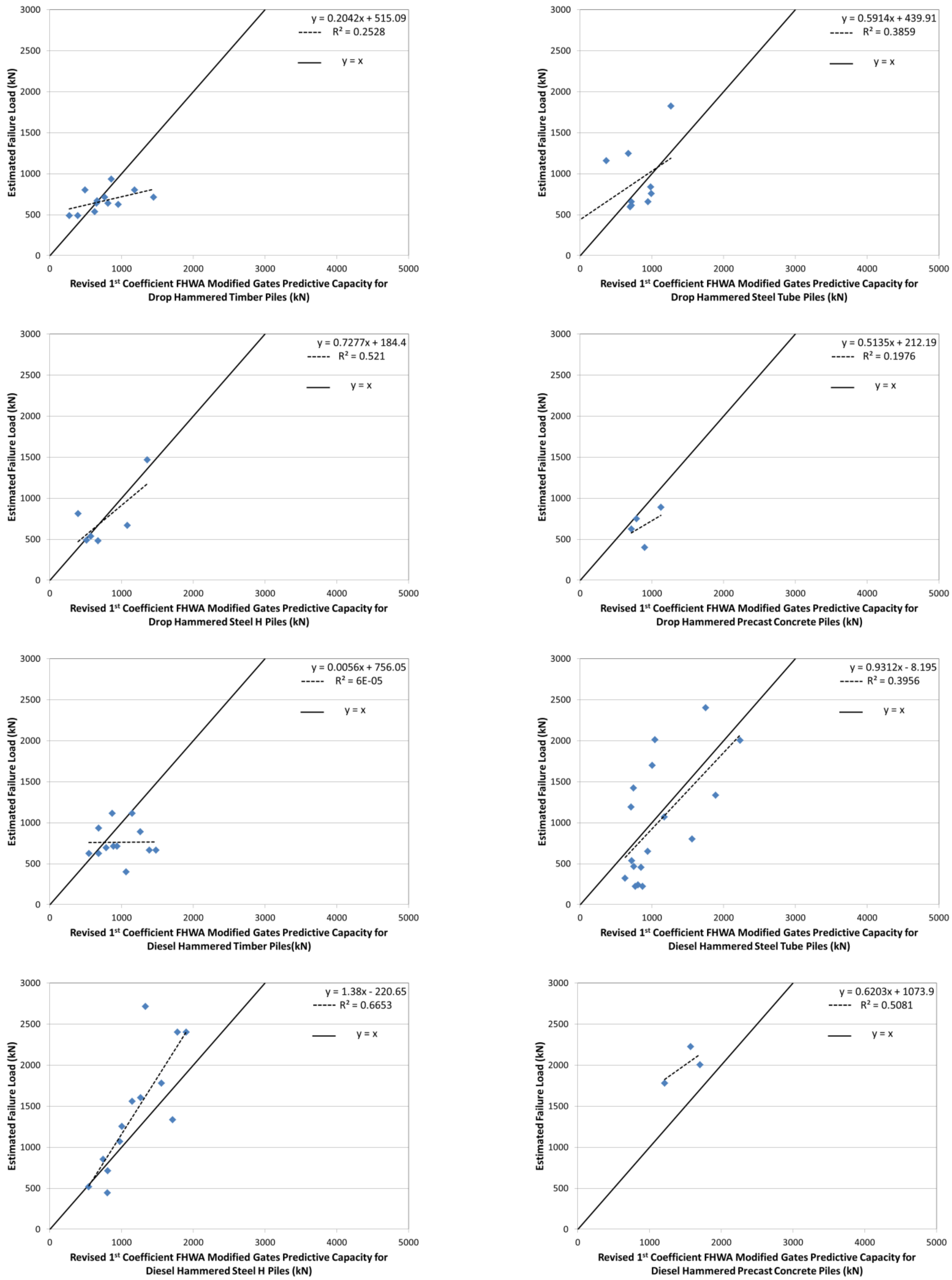


Figure 58: Revised 1.75 coefficient from FHWA modified Gates formula predicted capacities vs. pile test failure loads divided by hammer and pile type

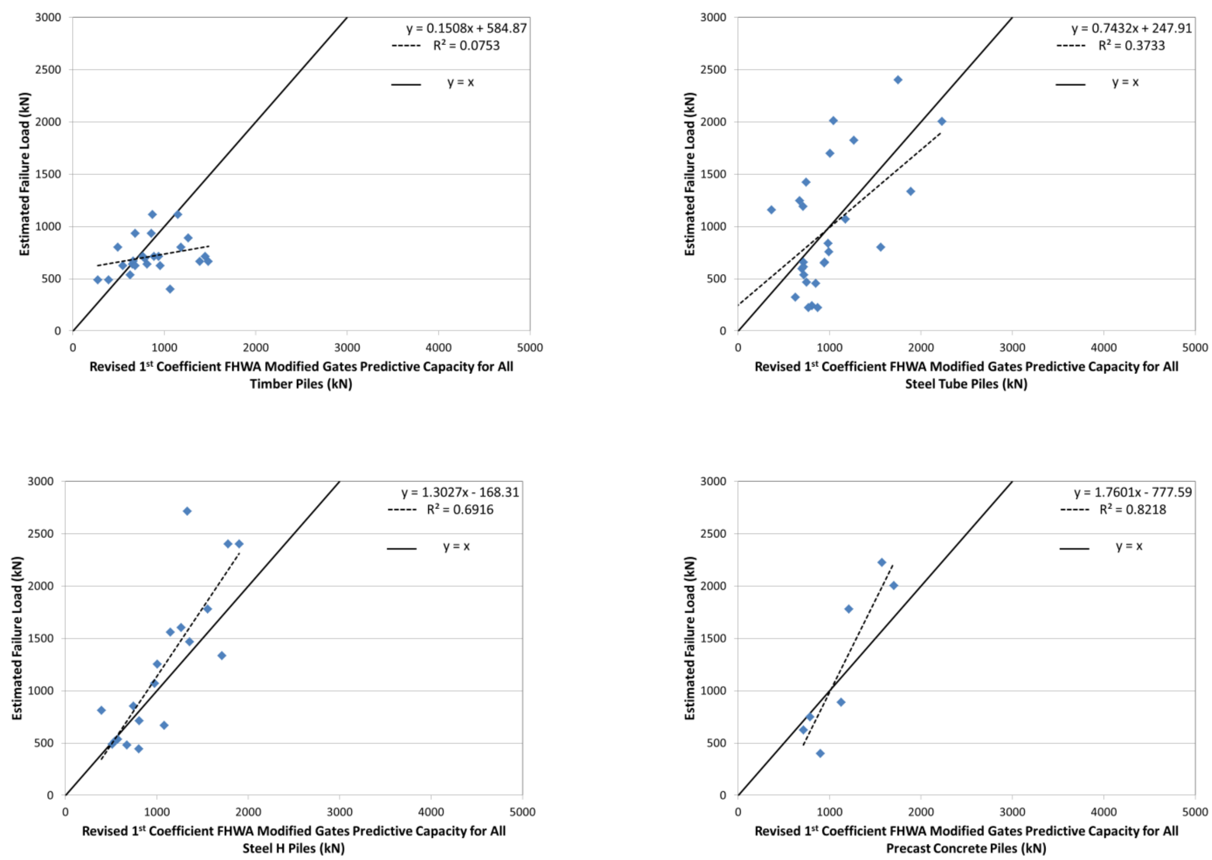


Figure 59: Revised 1.75 coefficient from the FHWA modified Gates formula predicted capacities vs. pile test failure loads divided by pile type

For drop hammered piles, the revised coefficient does not cause a significant impact on predictive pile values since the equations of the best fit line and coefficients of determination remain approximately constant to that of the FHWA modified Gates formula. Using the revised coefficient for diesel hammered piles causes the data points to shift to the left and center about the 45^{o} line. This slightly improves the predictive accuracy for all pile types by causing the slope of the best fit line to approach unity except for H piles which become slightly under predicted. Similarly to drop hammered piles, the coefficients of determination for diesel hammered piles are approximately equal to those derived from the original FHWA modified Gates formula.

On the basis of pile types, the revised coefficients cause the data points to shift to the left, centre around the 1:1 line, and plot nearer to each other. Using the revised coefficients improves the slope of the best fit lines for timber and concrete filled steel tube piles, worsens for precast concrete piles, and remained approximately constant for H piles. The coefficients of determination increased for concrete filled steel tube piles, remained constant for timber and H piles, and decreased for precast concrete piles.

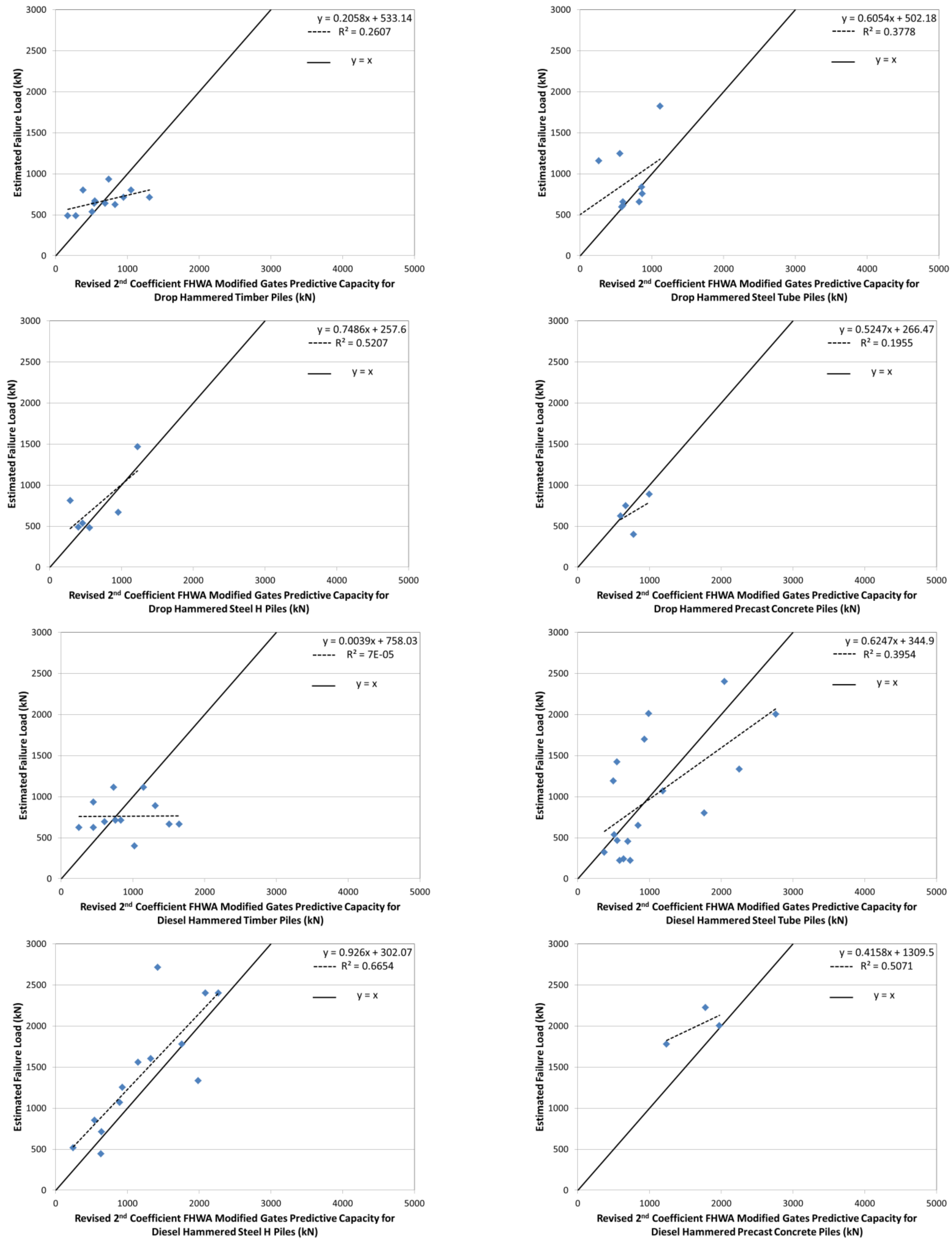


Figure 60: Revised 100 coefficient from the FHWA modified Gates formula predicted capacities vs. pile test failure loads divided by hammer and pile type

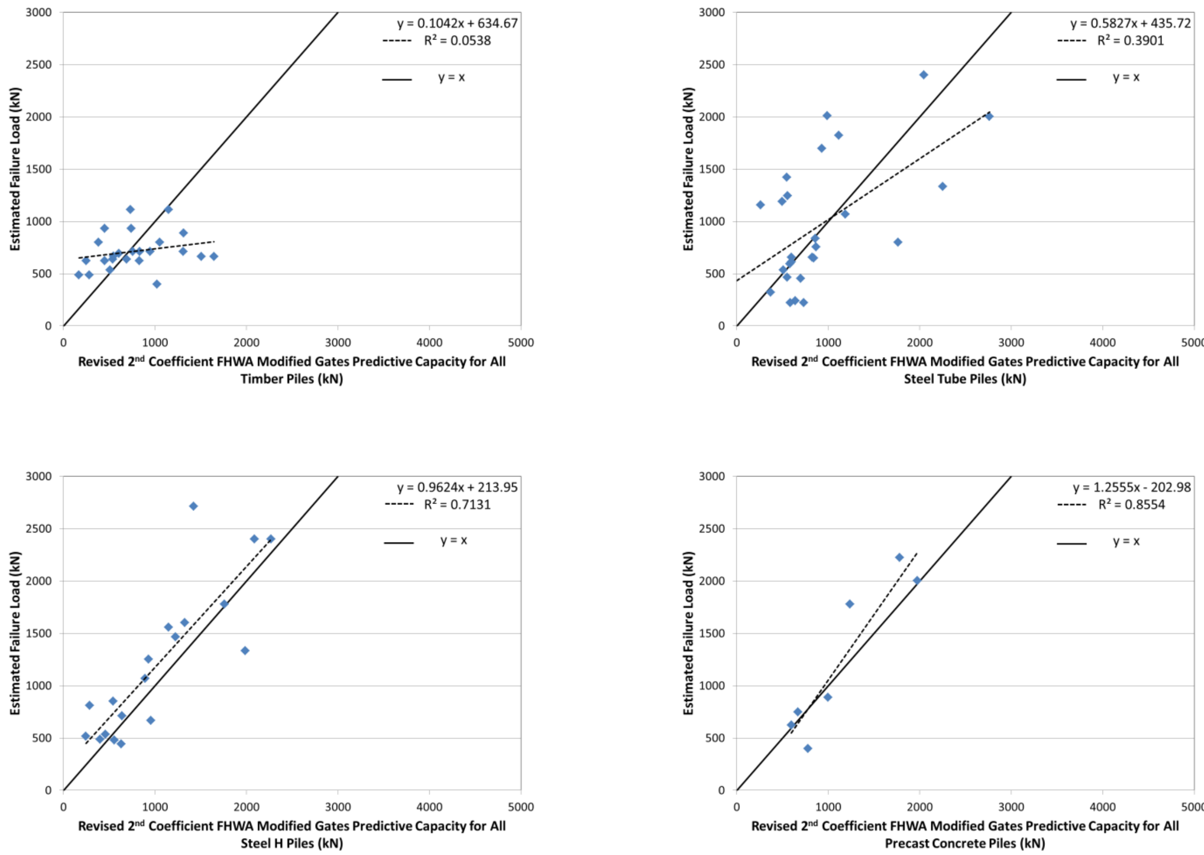


Figure 61: Revised 100 coefficient from the FHWA modified Gates formula predicted capacities vs. pile test failure loads divided by pile type

The predictive capacities calculated from the revised coefficients results in values similar to the capacities using the original coefficient value of 100, even though the new data points are shifted towards the 1:1 line for diesel hammer piles, those installed by drop hammer remain constant. This is demonstrated by the best fit line equations and coefficients of determinations which are approximately equal between the two formulae when considering piles installed by hammer type.

Analysis of predictive capacities with respect to pile type exhibits that the revised coefficient causes the data point to shift to the left, about the 1:1 line and increase predictive accuracy for concrete filled steel tube and H piles. This is seen by the increase in the coefficients of determination and the slopes of the best fit lines being closer to unity. The only exceptions are for timber and precast concrete piles whose coefficients of determination decrease. For precast concrete piles, the calculated bearing capacities become slightly under predicted as well.

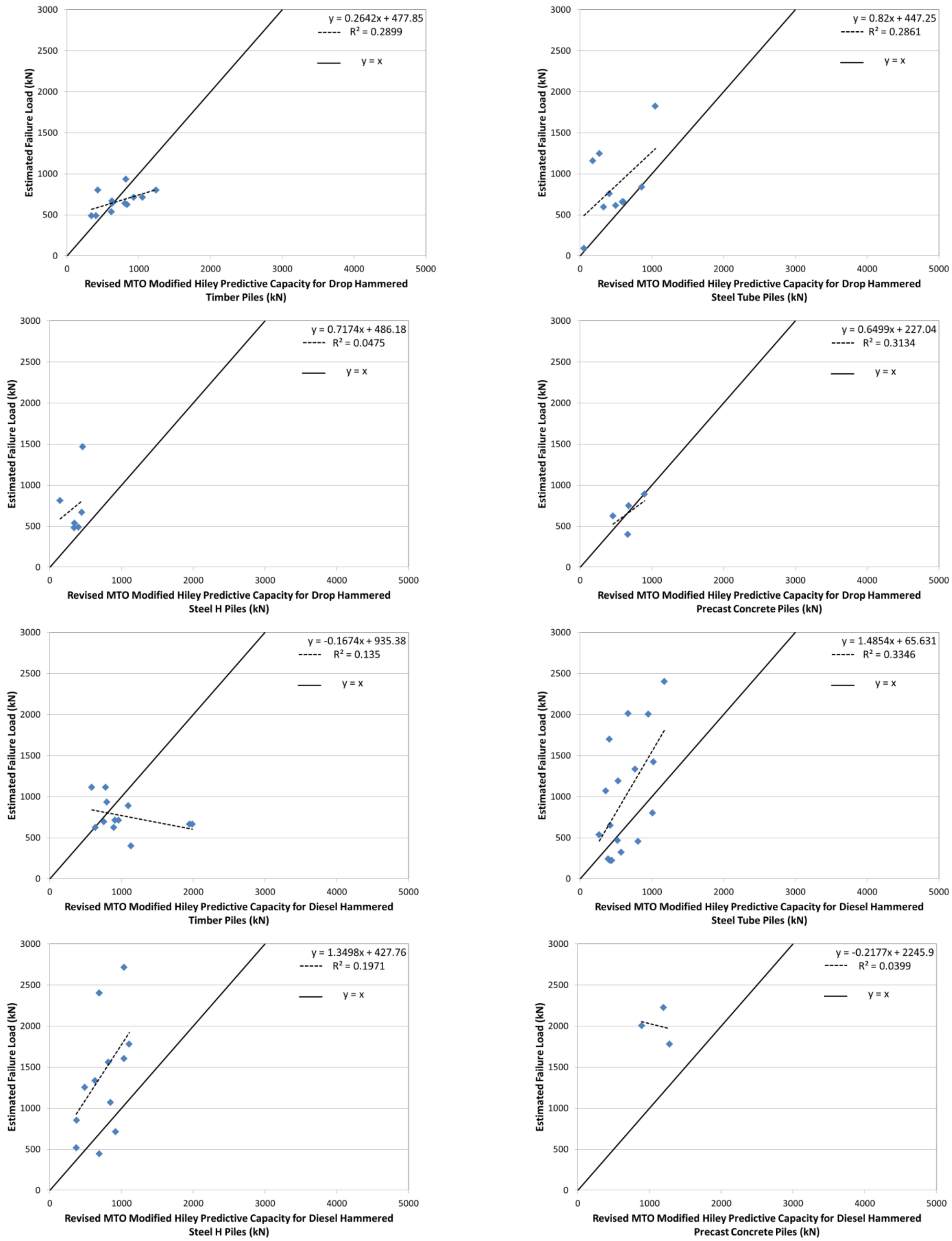


Figure 62: Revised MTO modified Hiley formula predicted capacities vs. pile test failure loads divided by hammer and pile type

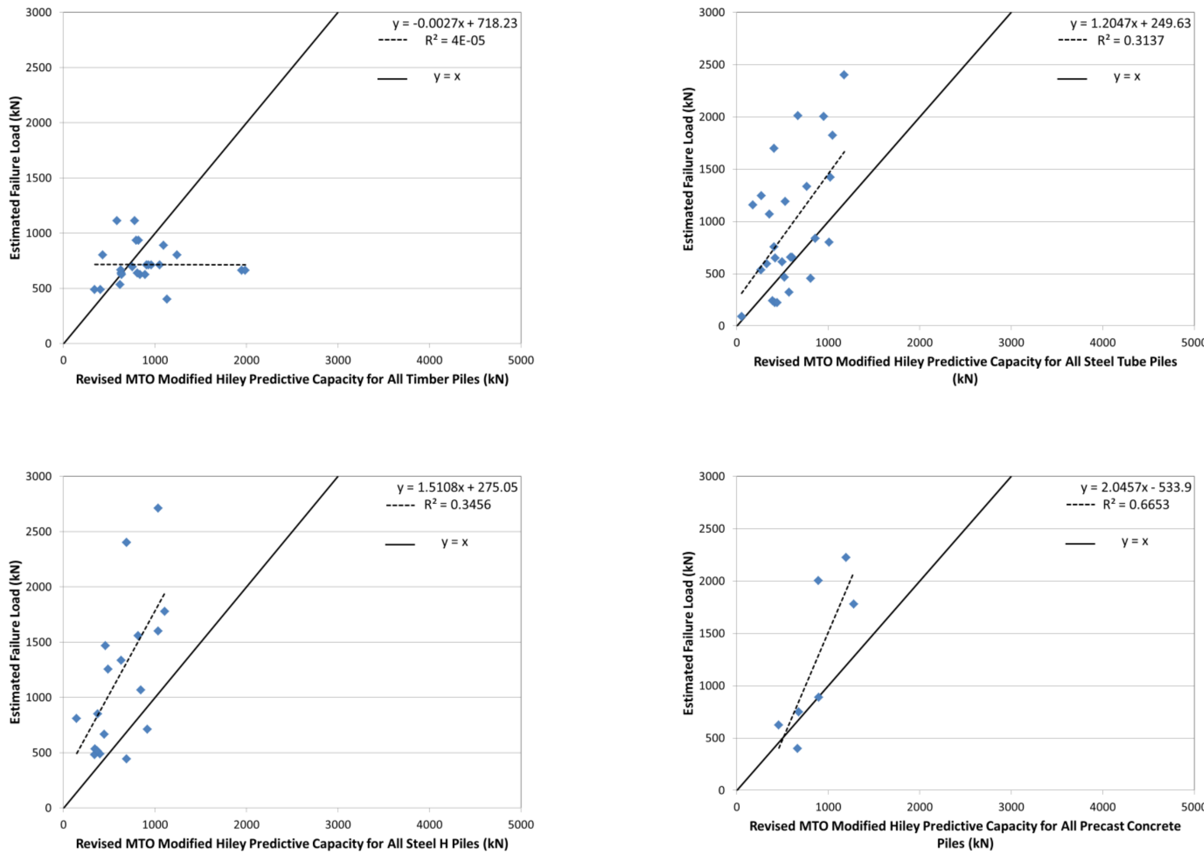


Figure 63: Revised MTO modified Hiley formula predicted capacities vs. pile test failure loads divided by pile type

The revised coefficient causes the predicted values of the MTO modified Hiley formula to plot closer together. For all drop hammered pile types this results in improved equations of best fit lines and coefficients of determination with the exception of precast concrete piles which results in a smaller coefficient of determination value and a slope of best fit line further away from unity. For diesel hammered piles, the effects of using the revised coefficient are more pronounced, however in general the capacities become under predicted. The coefficient of determination increases for all pile types except for timber. The slopes of the best fit approach a value nearer unity for concrete filled steel tube and H piles but not for Timber and precast concrete piles.

When considering all pile types regardless of installation method, the revised coefficients improve the coefficients of determination for all pile types except timber piles. The slopes of the best fit line approach closer to unity for all material types except precast concrete piles.

Similarly as with the original dynamic analysis of MTO driven piles, a statistical analysis of the results using the revised coefficients is also performed. Tables 33 to 35 present the ratios of predicted to field measured pile capacities, percent difference, and standard deviation of predicted to field measured pile capacities and the correlation values between predicted and field measured pile capacities, respectively.

Table 33: Ratios and Standard Deviations of Revised Dynamic Predicted to Field Tested Pile Capacities

Hammer	ENR Formula	Gates Formula	1.75 FHWA Modified Gates Formula	100 FHWA Modified Gates Formula	MTO Modified Hiley Formula
Maximum					
Drop	1.611	1.833	2.248	1.943	1.658
Diesel	2.486	3.099	3.914	3.288	2.997
Minimum					
Drop	0.105	-0.584	-6.034	-6.955	0.151
Diesel	0.314	0.343	0.491	0.383	0.241
Average					
Drop	0.486	0.951	0.894	0.708	0.840
Diesel	0.861	1.000	1.304	1.174	0.989
All	0.705	0.979	1.133	0.981	0.927

St. Dev.	ENR Formula	Gates Formula	1.75 FHWA Modified Gates Formula	100 FHWA Modified Gates Formula	MTO Modified Hiley Formula
Drop	0.303	0.408	1.330	1.452	0.382
Diesel	0.507	0.644	0.796	0.704	0.710
All	0.470	0.555	1.063	1.095	0.598

In general, using the revised coefficients results in ratios of predicted to measured capacities close to unity and shows an improvement over values from the original dynamic analysis from piles in the MTO database (Table 25). This is true for the ENR formula when considering drop and diesel hammered piles separately; however, when considering them together the ratio actually moves away from unity by a factor of approximately 1.16. For the Gates formula, the ratio is improved for all pile installation methods and approaches a value very close to unity. The ratios from the FHWA modified Gates formulae for all installation methods and revised values improved except when adjusting the 100 coefficient for drop hammered piles which became under predicted. Similarly, the results from the MTO modified formula using the revised C3 coefficient causes the ratios to decrease towards unity, the exception is again for piles installed by drop hammers which become under predicted.

The variation in the data, on average, as given by the standard deviations also improves when using the revised coefficients for the analysis. The only notable exception to this for values derived from the Gates formula, which increases despite the pile driver type. For the ENR formula the standard deviation decreases for all installation methods except drop hammered piles, this is also true for the revised FHWA modified gates formulae. The standard deviations derived from using the MTO modified formula decrease for all pile driver types utilized.

Table 34: Percent Difference and Standard Deviations of Revised Predicated to Estimated Capacities

Hammer	ENR Formula	Gates Formula	1.75 FHWA Modified Gates Formula	100 FHWA Modified Gates Formula	MTO Modified Hiley Formula
Maximum					
Drop	61.096	88.250	124.750	94.250	65.750
Diesel	148.649	209.910	291.441	228.829	199.698
Minimum					
Drop	-89.542	-158.427	-703.371	-795.506	-84.875
Diesel	-68.587	-65.721	-50.903	-61.701	-75.882
Average					
Drop	-51.440	-4.929	-10.639	-29.162	-16.020
Diesel	-13.881	-0.023	30.388	17.425	-1.114
All	-29.490	-2.062	13.338	-1.936	-7.308

St. Dev.	ENR Formula	Gates Formula	1.75 FHWA Modified Gates Formula	100 FHWA Modified Gates Formula	MTO Modified Hiley Formula
Drop	0.303	0.408	1.330	1.452	0.382
Diesel	0.507	0.644	0.796	0.704	0.710
All	0.470	0.555	1.063	1.095	0.598

Comparing Table 34 to 26, it is evident that the percent difference between predicted and measured capacities using the revised coefficients significantly decrease when compared to the original dynamic analysis for all pile driver types, except when considering drop and diesel hammered piles together for the ENR formula which increases by approximately 11 percent and drop hammered piles from the revised 100 coefficient of the FHWA modified Gates formula and MTO modified Hiley formula which decrease by approximately 15 and 17 percent, respectively.

Table 35: Correlation of Revised Predicted to Field Tested Results of Pile Capacities

Hammer	ENR Formula	Gates Formula	1.75 FHWA Modified Gates Formula	100 FHWA Modified Gates Formula	MTO Modified Hiley Formula
Drop	0.466	0.525	0.533	0.520	0.268
Diesel	0.711	0.683	0.683	0.683	0.208
All Piles	0.702	0.655	0.675	0.698	0.300

Comparing Tables 35 and 27 indicates that in general the correlation values stayed constant or slightly increased when predicting piles bearing capacities from dynamic formulae using the revised coefficients rather than the original values. On average, all correlation values range from 48.6 to 71.1 percent except for those the MTO modified Hiley formula which range from 20.8 to 30 percent, which is significantly lower than the values from the other formulae.

Since the decision to use the average value for each coefficient is somewhat arbitrary, a multi regression analysis is also completed to determine if other coefficient values produce more accurate predictions of pile bearing capacities.

For complete calculation using the revised coefficients, see Appendix E.

5.3.3 Pile Bearing Capacities from Multi Regression Analysis

Multi regression analysis (MRA) is performed using the built in solver function of Microsoft Excel. Select coefficients from each dynamic formula are varied so that when the predicted bearing capacity is compared to the measured pile capacity the lowest possible amount of normalized error is produced. The normalized error is given by the formula below:

$$Normalized\ Error = \frac{\sqrt{\overline{(Predicted\ Capacity - Measured\ Capacity)^2}}}{\overline{Predicted\ Capacity}} \tag{49}$$

For the ENR, Gates, FHWA modified Gates, Hiley, and MTO modified Hiley dynamic formulae the normalized error is calculated for piles installed by drop hammers and diesel hammers separately as well as together. Additionally, for the Hiley and MTO modified Hiley formulae the normalized error is also calculated for piles by material type; timber, concrete, H, and open ended concrete filled steel tube as well as installation method; drop hammer, diesel hammer, and combined drop and diesel hammered piles together.

For each coefficient value calculated by solver, the trend line equation and coefficients of determination are recorded for comparison purposes and are given as Tables 36 to 47 below. The trend line is calculated in two ways; using the least squares method as well as forcing the y intercept to equal zero. Since Excel contains a known issue of calculating the coefficient of determination graphically within the chart function when the intercept is forced to zero it is instead calculating using the LINEST equation within Excel which is designed to overcome this error.

The tables are divided into three sections, the top third is the analysis of piles installed by drop hammer, the middle third is the results for piles installed by diesel hammer and the bottom third is for both drop and diesel hammered piles. The formula for each dynamic formula is given in the second column and the variable being adjusted is described in the third column. The values of the variables are given in the next two columns. The sixth column gives the value of the normalized error. The seventh and eighth columns contain the equation of the best fit regression line and corresponding coefficient of determination, respectively. The last column gives the number of piles used in the analysis.

The analysis consisted of varying the c coefficient so that a minimal amount of error is produced while increasing the coefficient of determination as much as possible and having the resulting slope and intercept of the trend line equation be as close as possible to one and zero, respectively. For each revised coefficient value the trend line equation is recorded as is and with the intercept equal to zero.

Table 36: Multi Regression Analysis for ENR Dynamic Formula Predictions

Pile Type	Formula	Coefficient Condition	c Coefficient	Modified Energy	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	R = 2 * Energy / (s+c)	Original Value	1	1	6.0016	y = 2.5172x + 447.24 y = 5.8106x	0.1505 0.8203	32
		Solving for c	0.000	1	1.1452	y = 0.3089x + 600.97 y = 1.0296x	0.1650 0.6051	
		Solving for Modified Energy	1	7.0831	0.4603	y = 0.3554x + 447.24 y = 0.8203x	0.1505 0.8203	
		Solver Values	0.4175	4.3217	0.4124	y = 0.4781x + 353.97 y = 0.8549x	0.2525 0.8549	
		1:1 Slope	0.4	2	1.1920	y = 1.0125x +354	0.2556	
			1	5.8100	0.5083	y = 1.0001x	0.8203	
	R = 2 * Energy / (s+c) + 100 000	Best Fit Equation	0.131	1	0.3730	y = 0.9995x + 4.8866	0.2552	
Diesel	R = 2 * Energy / (s+c)	Original Value	0.1	1	0.4877	y = 0.6246x + 386.83 y = 0.8655x	0.4239 0.8258	45
		Solving for c	0.108	1	0.4848	y = 0.6712x + 362.68 y = 0.909x	0.4318 0.8313	
		Solving for Modified Energy	0.1	1.048	0.4851	y = 0.5959x + 386.83 y = 0.8258x	0.4239 0.8258	
		Solver Values	0.8443	4.776	0.3517	y = 1.1468x - 379.85 y = 0.8899x	0.6169 0.8899	
		1:1 Slope	0.3	1.85	0.4026	y = 1.0232x – 28.588	0.5453	
			0.1	0.8655	0.5337	y = 1x	0.8258	
	R = 2 * Energy / (s+c) + 50 000	Best Fit Equation	0.1625	1	0.4320	y = 1x - 2.2513	0.4761	
Combined	R = 2 * Energy / (s+c)	Original Value	1/0.1	1	0.8462	y = 0.5007x + 594.50 y = 0.8941x	0.4139 0.7012	77
		Solving for c	0.0945	1	0.6686	y = 0.5384x + 511.29 y = 0.8847x	0.4471 0.7633	
		Solving for Modified Energy	1/0.1	1.2751	0.7797	y = 0.3927x + 594.5 y = 0.7012x	0.4139 0.7012	
		Solver Values	1.0446	6.0056	0.4005	y = 0.7967x + 100.45 y = 0.8710x	0.5284 0.8710	
		1:1 Slope	1.7	7.35	0.4762	y = 0.9987x + 78.352	0.4972	
			1	5.0000	0.4332	y = 1.0093x	0.8713	
	R = 2 * Energy / (s+c) + 90 000	Best Fit Equation	0.198	1	0.4294	y = 1.0013x – 11.024	0.4920	

For data derived from the ENR formula, forcing the intercept equal to zero always results in the slope increasing in value and becoming closer to unity and in all but one case. The coefficient of determination values associated with the best fit line equations seems artificially high, especially when comparing it to the coefficients of determination of best fit lines using the regression method of least squares. For this reason only the results from the least squares regression without forcing the intercept to zero is discussed below.

When examining the results it becomes clear that no change in the c coefficient can cause the predicted values or trend lines to match the measured pile bearing capacities or 1:1 line, respectively. To try and overcome this limitation a modifier is added to the energy term, initially set to a value of one and varied to determine if a trend line which lies closer to the 1:1 line is produced.

Solving for one variable at a time does not produce the desired effect however solving for both simultaneously results in the lowest normalized error. For drop and both diesel and drop hammered piles examined together, this does not result in a trend line with slope of one, however for diesel hammered piles solely the solver determined values of the c and energy modifier coefficient does produce a trend line with a slope near unity.

To force the trend line slope towards unity c values of 0.4 and 0.3 and energy modifier values of 2 and 1.85 are required for drop and diesel hammered piles respectively. An alternative technique to obtain a best fit equation with a minimal intercept value is to add an offset to the dynamic equation which shifts the least squares line to almost match the 1:1 line. However this introduces the same problem as with the FHWA modified Gates formula; where a set of zero results in the pile being able to support some load, either positive or negative, which does not make physical sense.

The original coefficient of $^{1}/_{7}$ at the beginning of the Gates formula is denoted by the variable D. Similarly as with the analysis performed on the ENR formula a term is multiplied to the energy value in an effort to improve the predictive capability of the dynamic formula.

Similarly, as with the regression analysis of predicted bearing capacities derived from the ENR formula, forcing the best fit line to intercept the y axis at zero causes the coefficient of determination values to increase significantly. Unlike the data from the ENR formula, forcing the intercept to zero causes the slope of the best fit lines to decrease for all installation methods except when considering drop hammered piles solely. Solving for the D and energy modifying terms simultaneously does not result in a best fit line with a slope of one and produces coefficients of determination which are similar to those from when solving for the D and energy modifying coefficients individually, which gives the lowest normalized error. Setting the D coefficient value an adding an extra term at the end of the formula which results in a best fit line with a slope of one also increases the amount of error between predicted and field estimated bearing capacities.

Table 37 Multi Regression Analysis for Gates Dynamic Formula Predictions

Pile Type	Formula	Coefficient Condition	Coefficient D	Modified Energy	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	R = D * Energy$^{1/2}$ * (1-log(s))	Original Value	1/7	1	2.6711	y = 2.4497x + 195.6 y = 3.2585x	0.2757 0.8821	32
		Solving for D	0.5277	1	0.3592	y = 0.6631x + 195.6 y = 0.8821x	0.2757 0.8821	
		Solving for Modified Energy	1/7	13.6486	0.3592	y = 0.6631x + 195.6 y = 0.8821x	0.2757 0.8821	
		Solver Values	0.5397	0.9561	0.3592	y = 0.6631x + 195.6 y = 0.8821x	0.2757 0.8821	
		1:1 Slope	0.35	1	0.6148	y = 0.9999x +195.6	0.2757	
			0.4655	1	0.3825	y = 1x	0.8821	
	R = D * Energy$^{1/2}$ * (1-log(s)) + 21	Best Fit Equation	0.350	1	0.3701	y = 0.9999x + 8.6738	0.2757	
Diesel	R = D * Energy$^{1/2}$ * (1-log(s))	Original Value	1/7	1	2.2767	y = 4.1958x – 600.47 y = 2.8360x	0.4663 0.8456	45
		Solving for D	0.4791	1	0.4067	y = 1.251x - 600.47 y = 0.8456x	0.4663 0.8456	
		Solving for Modified Energy	1/7	11.2481	0.4067	y = 1.251x - 600.47 y = 0.8456x	0.4663 0.8456	
		Solver Values	0.5381	0.7927	0.4067	y = 1.251x - 600.47 y = 0.8456x	0.4663 0.8456	
		1:1 Slope	0.6	1	0.4496	y = 0.999x – 600.47	0.4663	
			0.4051	1	0.4423	y = 1.0001x	0.8456	
	R = D * Energy$^{1/2}$ * (1-log(s)) - 68	Best Fit Equation	0.600	1	0.4381	y = 0.999x + 4.2588	0.4663	
Combined	R = D * Energy$^{1/2}$ * (1-log(s))	Original Value	1/7	1	2.4405	y = 2.9418x - 12.711 y = 2.9089x	0.4472 0.8507	77
		Solving for D	0.4896	1	0.4068	y = 0.8584x - 12.711 y = 0.8488x	0.4472 0.8507	
		Solving for Modified Energy	1/7	11.7451	0.4068	y = 0.8584x - 12.711 y = 0.8488x	0.4472 0.8507	
		Solver Values	0.5538	0.7817	0.4068	y = 0.8584x - 12.711 y = 0.8488x	0.4472 0.8507	
		1:1 Slope	0.419	1	0.4397	y = 1.003x - 12.711 y = 0.9918x	0.4472 0.8507	
	R = D * Energy$^{1/2}$ * (1-log(s))	Best Fit Equation	0.4156	1	0.4434	y = 0.9999x	0.8507	

For the analysis of data calculated from the FHWA modified Gates formula, only the coefficients at the beginning and end of the formula are varied rather than adding a term to modify the energy imparted into the soil – pile system. Unlike the previous formulae, forcing the best fit line to intercept the y axis at zero does not automatically increase or decrease its slope.

Solving for both coefficients simultaneously results in the lowest normalized error and best fit lines with a slope of one however the offset is significant. Changing the second coefficient reduces the offset considerably while keeping the coefficient of determination value the same and without significantly increasing the amount of error.

Table 38: Regression Analysis for FHWA modified Gates Dynamic Formula Predictions

Pile Type	Formula	Coefficient Condition	1st Coefficient	2nd Coefficient	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	R = C1*0.75*Energy$^{1/2}$ *log(10N) - C2	Original Value	1.75	100	0.4565	y = 0.4618x + 401.16 y = 0.9049x	0.2757 0.8380	32
		Solving for 1st Coefficient	1.8982	100	0.4377	y = 0.4258x + 385.11 y = 0.8067x	0.2757 0.8451	
		Solving for 2nd Coefficient	1.75	64.2405	0.4105	y = 0.4618x + 327.65 y = 0.7791x	0.2757 0.8636	
		Solver Values	0.8082	-66.0723	0.3433	y = 1x – 98.493 y = 0.8866x	0.2757 0.8866	
		1:1 Slope	0.8082	-66.0723	0.3433	y = 1x – 98.493	0.2757	
			0.8082	-43.9500	0.3655	y = 1x	0.8872	
		Best Fit Equation	0.808	-44	0.3654	y = 1x – 0.2491	0.2757	
Diesel	R = C1*0.85*Energy$^{1/2}$ *log(10N) - C2	Original Value	1.75	100	0.4858	y = 0.743x – 269.75 y = 0.6138x	0.4663 0.8553	45
		Solving for 1st Coefficient	1.3421	100	0.4005	y = 0.9688x - 169.25 y = 0.8567x	0.4663 0.8576	
		Solving for 2nd Coefficient	1.75	215.834	0.4173	y = 0.743x + 113.33 y = 0.8117x	0.4663 0.8581	
		Solver Values	1.3003	86.477	0.4003	y = 1x - 215.56 y = 0.8566x	0.4663 0.8566	
		1:1 Slope	1.3003	86.477	0.4003	y = 1x – 215.56	0.4663	
			1.3003	135	0.4370	y = 1.0003x	0.8594	
		Best Fit Equation	1.3003	135	0.4372	y = 1x + 0.4186	0.4663	
Combined	R = C1*Energy$^{1/2}$ *log(10N) - C2	Original Value	1.75	100	0.5214	y = 0.501x + 263.62 y = 0.6448x	0.4382 0.8355	77
		Solving for 1st Coefficient	1.4745	100	0.4808	y = 0.5946x + 305.28 y = 0.7985x	0.4382 0.8295	
		Solving for 2nd Coefficient	1.75	117.9591	0.5191	y = 0.5010x + 303.6612 y = 0.6717x	0.4382 0.8297	
		Solver Values	0.8767	-54.276	0.4149	y = 1x - 200.94 y = 0.8451x	0.4382 0.8451	
		1:1 Slope	0.8767	-54.276	0.4149	y = 1x - 200.94	0.4382	
			0.9094	0	0.4592	y = 1x	0.848	
		Best Fit Equation	0.8767	-9	0.4563	y = 1x + 0.5819	0.4382	

The Hiley formula is the most complex equation discussed so far, with variables for nine unique parameters which are specific to the pile driver being used, soil being driven into, and the pile itself. Since most variables are either given; such as rated energy from the pile driver, measured; such as the set and weight of the ram or calculated; such as the pile weight, only two parameters are varied during the analysis; the compression of the soil during driving, C_3, and the coefficient of restitution, e.

The compression and coefficient of restitution variables are chosen since the values are somewhat arbitrarily assigned. According to Chellis, the amount of soil compression is taken as a constant of 0.254 cm (0.1 inches) despite soil type, pile type or difficulty of driving. The coefficient of restitution is chosen as 0.5 since the pile cap material and physical condition is not available in many if not in all of the pile records.

Table 39: Multi Regression Analysis for Hiley Dynamic Formula Predictions

Pile Type	Formula	Coefficient Condition	C3 Coefficient	Coefficient of Restitution (e)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)$	Original Value	0.1	0.5	0.5639	y = 0.182x + 598.24 y = 0.8213x	0.0437 0.7480	32
		Solving for C3 Coefficient	0.8159	0.5	0.5636	y = 0.1788x + 598.64 y = 1.3006x	0.0441 0.7605	
		Solving for e Coefficient	0.1	0.7986	0.4936	y = 0.274x + 475.04 y = 0.7014x	0.1277 0.8090	
		Solver Values	0.5555	1.0000	0.4333	y = 0.4062x + 400.87 y = 0.8250x	0.1769 0.8379	
		1:1 Slope	21.5	1	9.2303	y = 1.003x + 657.86	0.0250	
			0.997	1	0.4918	y = 0.9999x	0.8275	
	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p) + 150\ 000$	Best Fit Equation	21.5	1	0.4189	y = 1.003x – 11.761	0.0250	
Diesel	$R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)$	Original Value	0.1	0.5	0.7432	y = -0.0685x + 1258 y = 0.4750x	0.0084 0.5666	45
		Solving for C3 Coefficient	0.4500	0.5	0.6695	y = -0.0098x + 1116.1 y = 0.7109x	7E-5 0.6293	
		Solving for e Coefficient	0.1	0.7290	0.7016	y = 0.0487x + 1011.2 y = 0.4132x	0.0044 0.6640	
		Solver Values	1.553	1.0000	0.4169	y = 1.1221x - 382.78 y = 0.8599x	0.4175 0.8401	
		1:1 Slope	1.320	1	0.4268	y = 1.0004x – 358.03	0.3973	
			1.953	1	0.4453	y = 1x	0.8455	
	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p) – 80\ 000$	Best Fit Equation	1.320	1	0.4637	y = 1.0004x – 1.8014	0.3973	
Combined	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)$ $R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)$	Original Value	0.1	0.5	0.7794	y = 0.0999x + 824.96 y = 0.5112x	0.0242 0.5796	77
		Solving for C3 Coefficient	0.4144	0.5	0.7089	y = 0.2135x + 738.17 y = 0.7317x	0.0501 0.6370	
		Solving for e Coefficient	0.1	0.7026	0.7441	y = 0.1604x + 687.31 y = 0.4517x	0.0802 0.6546	
		Solver Values	1.3910	1.0000	0.4794	y = 0.6472x + 265.57 y = 0.8419x	0.3692 0.8182	
		1:1 Slope	2.53	1	0.6888	y = 1x + 226.65	0.3975	
			1.834	1	0.5173	y = 1x	0.8243	
	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p) + 50\ 000$ $R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p) + 50\ 000$	Best Fit Equation	2.53	1	0.4742	y = 1x + 4.1048	0.3975	

Solving for both coefficients simultaneously improves the predictive capabilities of the dynamic formula as well as the coefficient of determination values; however the slope of the best fit line does not reach unity except when examining diesel hammered piles solely.

When attempting to obtain a best fit line with a slope of one, the coefficient of restitution is set to a value of unity and the C_3 coefficient depended upon the hammer type, with diesel hammers dominating the required value. The coefficients of determination values are comparable to those corresponding to solver determined values except for drop hammered piles which decrease by a factor of seven. The y intercept of the best fit line equations are considerably offset from the zero coordinate. To overcome this a term of 15 000, -8 000, and 50 000 is added to the end of the dynamic formula for hammers installed by drop, diesel, and drop and diesel hammered piles together, respectively. This is similar to the format of the FHWA modified Gates formula and thus also contains the same concerns, where a set of zero results in a pile being able to support some load either positive or negative.

Due to the large differences in C_3 values between drop and diesel hammered installed piles it is decided to redo the analysis according to pile type to determine the significance that pile material plays in bearing capacity predictions.

Table 40: Multi Regression Analysis for Hiley Dynamic Formula Predictions for Timber Piles

Pile Type	Formula	Coefficient Condition	C3 Coefficient	Coefficient of Restitution (e)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (e_f{*}E_n) / (s+{}^{1}/_{2}(C1+C2+C3)) * ((W_r+e^2{*}W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.4578	y = 0.1702x + 507.23 y = 0.6292x	0.2516 0.8998	12
		Solving for C3 Coefficient	0.6129	0.5	0.2910	y =0.2692x + 492.1 y = 0.9442x	0.1991 0.9236	
		Solving for e Coefficient	0.1	0	0.4270	y = 0.1814x + 506.12 y = 0.6681x	0.2469 0.9020	
		Solver Values	0.5276	0.0000	0.2893	y = 0.2749x + 489.19 y = 0.9495x	0.2042 0.9247	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			0.607	0	0.2944	y = 0.9998x	0.9267	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	
Diesel	$R = (12{*}e_f{*}E_n) / (s+{}^{1}/_{2}(C1+C2+C3)) * ((W_r+e^2{*}W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.9512	y = -0.1037x + 1013.7 y = 0.2427x	0.3174 0.6750	12
		Solving for C3 Coefficient	1.6884	0.5	0.5640	y = -0.2849x + 1002 y = 0.7385x	0.2267 0.7260	
		Solving for e Coefficient	0.1	0.3690	0.8507	y = -0.1055x + 1007.7 y = 0.2494x	0.3217 0.6641	
		Solver Values	2.3908	1.0000	0.4946	y = -0.2917x + 1005.5 y = 0.7888x	0.1642 0.7802	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			3.231	1	0.5542	y = 1x	0.7834	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	
Combined	$R = (e_f{*}E_n) / (s+{}^{1}/_{2}(C1+C2+C3)) * ((W_r+e^2{*}W_p)/(W_r+W_p))$ $R = (12{*}e_f{*}E_n) / (s+{}^{1}/_{2}(C1+C2+C3)) * ((W_r+e^2{*}W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.8828	y = -0.0138x + 739.17 y = 0.2927x	0.0075 0.6447	24
		Solving for C3 Coefficient	1.3182	0.5	0.6081	y = -0.0293x + 737.27 y = 0.7491x	0.0044 0.7160	
		Solving for e Coefficient	0.1	0	0.8730	y = -0.0196x + 746.36 y = 0.3126x	0.0134 0.6313	
		Solver Values	1.8353	1.000	0.5718	y = 0.0086x + 709.53 y = 0.7687x	0.0044 0.7442	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			2.618	1	0.6437	y = 0.9999x	0.7523	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	

When examining timber piles it is determined that no combination of C_3 and coefficient of restitution values could result in a best fit equation with a slope near unity even with an additional term added to the end of the formula.

For precast concrete piles, best fit equations with slopes of one are achieved for all three categories of pile installation methods. For drop hammered piles the coefficient of restitution value which yields the best result is that of the original value of 0.5, while for diesel hammered piles, a value of one is used, and that for diesel and drop hammered piles together, a value of approximately 0.73 is used. The C_3

coefficients range from 1 to 6.3 to 8.5 for drop and diesel hammered, diesel hammered, and drop hammered piles, respectively.

Table 41: Multi Regression Analysis for Hiley Dynamic Formula Predictions for Precast Piles

Pile Type	Formula	Coefficient Condition	C3 Coefficient	Coefficient of Restitution (e)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.3118	y = 0.6768x + 46.312 y = 0.7253x	0.5269 0.9676	4
		Solving for C3 Coefficient	0.5002	0.5	0.2065	y =0.6312x + 218.94 y = 0.9133x	0.6007 0.9638	
		Solving for e Coefficient	0.1	0	0.2403	y = 0.6768x + 144.94 y = 0.8576x	0.3589 0.9545	
		Solver Values	0.3688	0.2950	0.1807	y = 0.7866x + 130.26 y = 0.9663x	0.5831 0.9694	
		1:1 Slope	8.507	0.5	3.1799	y = 1x + 502.64	0.5447	
			0.7277	0.5	0.2376	y = 1x	0.9525	
	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 115\,000$	Best Fit Equation	8.507	0.5	0.1805	y = 1x – 9.21	0.5447	
Diesel	$R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.3033	y = -0.0084x + 2198.2 y = 0.8060x	0.0652 0.9292	3
		Solving for C3 Coefficient	0.2855	0.5	0.2639	y = -0.1112x + 2223.8 y = 0.9485x	0.0751 0.9339	
		Solving for e Coefficient	0.1	0.4640	0.3007	y = -0.0878x + 2198.7 y = 0.8355x	0.0723 0.9231	
		Solver Values	1.421	1.0000	0.1019	y = 0.124x + 1752.1 y = 0.9921x	0.0056 0.9896	
		1:1 Slope	6.286	1	1.9966	y = 1x + 1329.6	0.0047	
			1.441	1	0.1023	y = 1.0002x	0.9897	
	$R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 300\,000$	Best Fit Equation	6.286	1	0.0903	y = 1x – 5.6992	0.0047	
Combined	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$ $R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.3357	y = 0.7236x + 134.44 y = 0.7925x	0.7232 0.9331	7
		Solving for C3 Coefficient	0.3098	0.5	0.2840	y = 0.8823x + 98.461 y = 0.9425x	0.7329 0.9363	
		Solving for e Coefficient	0.1	0.4390	0.3287	y = 0.7529x + 155.27 y = 0.8376x	0.6834 0.9231	
		Solver Values	0.9716	0.845	0.2152	y = 0.9042x + 65.592 y = 0.9433x	0.8512 0.9646	
		1:1 Slope	1	0.7323	0.2484	y = 1.05357x - 29.752	0.8147	
			1.589	1	0.2428	y = 0.9999x	0.9569	
	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 20\,000$ $R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 20\,000$	Best Fit Equation	1	0.7323	0.2289	y = 0.999x + 0.1676	0.8291	

When determining values which results in slopes of best fit lines near unity, the low corresponding coefficient of determination values for diesel hammered piles seem to be affected by the small number of piles within the sample set. When more piles are analyzed such as with the drop hammered piles, and drop and diesel hammered piles together, a much higher value is obtained.

Considering only H piles, a slope of one for the best fit line for drop hammered piles is not possible to obtain. Using solver to simultaneously calculate C_3 and coefficient of restitution values for drop and diesel hammered piles also does not result in trend lines with slopes close to one; however, for combined drop and diesel hammered pile as well as solely diesel hammered pile analyses, slopes of one are obtained with correlation values only slightly lower than those from the solver determined values when setting the C3 value to 1.9 and 0.88, respectively and the coefficient of restitution to unity.

Table 42: Multi Regression Analysis for Hiley Dynamic Formula Predictions for H Piles

Pile Type	Formula	Coefficient Condition	C3 Coefficient	Coefficient of Restitution (e)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (e_f*E_n) / (s+{}^{1/}{}_{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.7991	y = 0.3212x + 571.23 y = 1.2562x	0.0374 0.7638	6
		Solving for C3 Coefficient	-0.1427	0.5	0.6441	y =0.2466x +571.26 y = 0.8767x	0.0765 0.7190	
		Solving for e Coefficient	0.1	0.8249	0.4924	y = 0.3551x + 442.38 y = 0.7899x	0.157 0.7995	
		Solver Values	0.4781	0.5000	0.4271	y = 0.524x + 315.45 y = 0.8735x	0.1685 0.8373	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			0.697	1	0.4558	y = 1.0001x	0.8382	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	
Diesel	$R = (12*e_f*E_n) / (s+{}^{1/}{}_{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.7991	y = 0.0027x + 1532.6 y = 0.8394x	3E-6 0.7672	14
		Solving for C3 Coefficient	0.1136	0.5	0.5077	y = 0.0124x + 1516.5 y = 0.8519x	7E-5 0.7692	
		Solving for e Coefficient	0.1	0.6122	0.4768	y = 0.0957x + 1342.7 y = 0.7201x	0.0044 0.7917	
		Solver Values	1.2977	1.0000	0.3609	y = 1.3082x - 769.49 y = 0.8841x	0.3084 0.8736	
		1:1 Slope	0.881	1	0.4006	y = 1.0002x - 614.09	0.2670	
			1.599	1	0.3819	y = 1.0002x	0.8759	
	$R = (12*e_f*E_n) / (s+{}^{1/}{}_{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) – 135\ 000$	Best Fit Equation	0.881	1	0.3903	y = 1.0002x – 13.084	0.2670	
Combined	$R = (e_f*E_n) / (s+{}^{1/}{}_{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$ $R = (12*e_f*E_n) / (s+{}^{1/}{}_{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.5625	y = 0.439x + 705.96 y = 0.8572x	0.1698 0.7595	20
		Solving for C3 Coefficient	0.865	0.5	0.5623	y = 0.4277x + 713.64 y = 0.8445x	0.1666 0.7575	
		Solving for e Coefficient	0.1	0.6255	0.5205	y = 0.425x + 600.32 y = 0.7214x	0.2194 0.7856	
		Solver Values	1.2263	1.0000	0.4067	y = 0.7456x + 219.93 y = 0.8735x	0.4236 0.8610	
		1:1 Slope	1.89	1	0.5061	y = 0.998x + 177.33	0.4354	
			1.553	1	0.4331	y = 1.001x	0.8634	
	$R = (e_f*E_n) / (s+{}^{1/}{}_{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 40\ 000$ $R = (12*e_f*E_n) / (s+{}^{1/}{}_{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 40\ 000$	Best Fit Equation	1.89	1	0.4156	y = 0.998x – 0.3515	0.4156	

Solving for C_3 and the coefficient of restitution simultaneously to achieve the lowest amount of error does not produce trend lines with a slope of approximately one. Although selecting certain values does allow for slopes near unity, however the normalized error is slightly larger and coefficients of determination to range from significantly smaller to slightly larger.

A final regression analysis is performed on the results from using the MTO modified Hiley formula and is given in Tables 44 to 47 below.

Since the only difference between the MTO modified Hiley and original Hiley formulae are the values of the coefficient of restitution, it is decided that the coefficient are varied as well. Since the formulae are so similar, instead of varying the C coefficient which may result in similar findings the energy efficiency factor is also changed to determine the effect it plays on pile bearing predictions.

Table 43: Multi Regression Analysis for Hiley Dynamic Formula Predictions for Steel Tube Piles

Pile Type	Formula	Coefficient Condition	C3 Coefficient	Coefficient of Restitution (e)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.7739	y = 0.6598x + 442.31 y = 1.2311x	0.2222 0.7843	10
		Solving for C3 Coefficient	-0.3500	0.5	0.5303	y =0.4739x + 429.7 y = 0.8296x	0.3316 0.7981	
		Solving for e Coefficient	0.1	0.9500	0.4530	y = 0.5442x + 306.87 y = 0.7992x	0.3205 0.8332	
		Solver Values	0.2579	1.0000	0.4444	y = 0.5881x + 236.09 y = 0.8397x	0.3134 0.8354	
		1:1 Slope	1.48	1.0000	0.825	y = 0.9988x + 272	0.2451	
			0.584	1.0000	0.4868	y = 1.0001x	0.8324	
	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 60\,000$	Best Fit Equation	1.48	1.0000	0.4609	y = 0.9988x + 5.2533	0.2451	
Diesel	$R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.8051	y = -0.0054x + 893.11 y = 0.4383x	6E-5 0.5188	16
		Solving for C3 Coefficient	0.4980	0.5	0.6644	y = 0.2267x + 649.87 y = 0.7692x	0.0239 0.6429	
		Solving for e Coefficient	0.1	0.7053	0.7736	y = 0.034x + 815.87 y = 0.3671x	0.0027 0.5861	
		Solver Values	1.7465	1.0000	0.4297	y = 1.7983x - 1101.3 y = 0.8555x	0.6328 0.8265	
		1:1 Slope	0.858	0.8	0.5024	y = 1.0002x - 367.61	0.2752	
			2.1944	1	0.4584	y = 1x	0.8349	
	$R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) – 80\,000$	Best Fit Equation	0.858	0.8	0.5700	y = 1.0002x – 11.467	0.2752	
Combined	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$ $R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p))$	Original Value	0.1	0.5	0.8590	y = 0.0465x + 814.13 y = 0.5073x	0.005 0.5084	26
		Solving for C3 Coefficient	0.3824	0.5	0.7598	y = 0.1725x + 713.65 y = 0.7742x	0.0257 0.6076	
		Solving for e Coefficient	0.1	0.7152	0.8180	y = 0.0663x + 765.6 y = 0.4162x	0.0145 0.5608	
		Solver Values	1.3789	1.0000	0.5266	y = 0.5907x + 269.56 y = 0.8157x	0.2183 0.7682	
		1:1 Slope	2.48	1	0.6985	y = 0.9995x + 166.14	0.283	
			1.9001	1	0.5707	y = 1x	0.7829	
	$R = (e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 40\,000$ $R = (12*e_f*E_n) / (s+\frac{1}{2}(C1+C2+C3)) * ((W_r+e^2*W_p)/(W_r+W_p)) + 40\,000$	Best Fit Equation	2.48	1	0.5248	y = 0.9995x – 11.814	0.283	

Table 44 examines all the piles studied; coefficient of restitution values of 0.25, 0.32, and 0.55 are applied to timber piles, steel piles, and steel piles driven without a cushion, respectively. The hammer efficiencies of 0.75 and 1 are used for piles installed by drop and diesel hammers, respectively.

Solving for both parameters simultaneously resulted in the closest match of predicted to measured pile capacities, rather than for one coefficient at a time. Similarly to the results of the Hiley formula, the optimal value of the coefficients of restitution is found at a value of unity. To obtain the lowest amount of normalized error, the efficiency value is calculated as approximately 0.55, 0.44, and 0.40 for drop, diesel, and combined drop and diesel hammered piles together, respectively. In order to obtain trend lines with a slope of unity efficiency values of 0.22, 0.49, and 0.29 are required for drop, diesel, and combined drop and diesel hammered piles, respectively. Changing these values does increase the normalized error by an approximate factor of three; however the coefficient of determination values stay constant at 8, 13, and 20 percent for diesel, combined drop and diesel, and drop hammered pile analysis, respectively.

Table 44: Multi Regression Analysis for MTO Modified Hiley Formula

Pile Type	Formula	Coefficient Condition	Coefficient of restitution (e)	Hammer Efficiency (e_f)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (n*e_f*WgH) / (s+{}^{c}/_{2})$	Original Value	0.25/0.32/0.55	0.75	0.6223	y = 0.1787x + 613.19 y = 0.8748x	0.0415 0.7259	32
		Solving for e Coefficient	0.801	0.75	0.4906	y =0.2837x + 471.33 y = 0.7076x	0.1321 0.8103	
		Solving for e_f Coefficient	0.25/0.32/0.55	0.9038	0.5903	y = 0.1483x + 613.19 y = 0.7259x	0.0415 0.7259	
		Solver Values	1.0000	0.5482	0.4465	y = 0.3997x + 414.7 y = 0.8323x	0.1956 0.8323	
		1:1 Slope	1	0.2191	1.5592	y = 1x + 414.7	0.1956	
			1	0.4562	0.5966	y = 1x	0.7725	
	$R = (n*e_f*WgH) / (s+{}^{c}/_{2}) + 415$	Best Fit Equation	1	0.2191	0.3851	y = 1x – 0.2911	0.1956	
Diesel	$R = (n*e_f*E) / (s+{}^{c}/_{2})$	Original Value	0.25/0.32/0.55	1	0.8202	y = -0.1099x + 1308.2 y = 0.4931x	0.0234 0.4955	45
		Solving for e Coefficient	0.7248	1	0.7006	y = 0.0476x + 1014.5 y = 0.4153x	0.0041 0.6633	
		Solving for e_f Coefficient	0.25/0.32/0.55	0.9950	0.8202	y = -0.1105x + 1308.2 y = 0.4956x	0.0234 0.4955	
		Solver Values	1.0000	0.4356	0.5333	y = 0.4077x + 536.09 y = 0.7419x	0.0789 0.7420	
		1:1 Slope	1	0.1776	1.4178	y = 1x + 536.09	0.0789	
			0.4	0.4934	1.0830	y = 1.001x	0.5232	
	$R = (n*e_f*E) / (s+{}^{c}/_{2}) + 537$	Best Fit Equation	1	0.1776	0.5735	y = 1x – 0.9042	0.0789	
Combined	$R = (n*e_f*WgH) / (s+{}^{c}/_{2})$ $R = (n*e_f*E) / (s+{}^{c}/_{2})$	Original Value	0.25/0.32/0.55	0.75/1	0.9546	y = 0.0175x + 940.93 y = 0.4082x	0.0012 0.4554	77
		Solving for e Coefficient	0.7728	0.75/1	0.8451	y = 0.1059x + 737.25 y = 0.3452x	0.0644 0.6208	
		Solving for e_f Coefficient	0.25/0.32/0.55	0.8481	0.8998	y = 0.0035x + 961.72 y = 0.4754x	3E-5 0.4754	
		Solver Values	1.0000	0.4030	0.6147	y = 0.3362x + 569.42 y = 0.7085x	0.1348 0.7084	
		1:1 Slope	1	0.1355	1.9889	y = 0.999x + 569.42	0.1348	
			1	0.2855	0.7303	y = 1x	0.7084	
	$R = (n*e_f*WgH) / (s+{}^{c}/_{2}) + 570$ $R = (n*e_f*E) / (s+{}^{c}/_{2}) + 570$	Best Fit Equation	1	0.1355	0.5655	y = 0.9999x – 0.5318	0.1348	

In order to determine if pile types combined with driving equipment influences the accuracy of the bearing capacity predictions, the analysis is redone using coefficients of restitution as the basis of study.

First timber piles are examined with an original coefficient of restitution value of 0.25 and hammer efficiencies of 75 and 100 percent for drop and diesel hammered piles, respectively.

It is found that there exists no combination of coefficient of restitution and hammer efficiency values which results in timber pile predictions with a trend line slope near unity, except for those installed by drop hammers.

Table 45: Multi Regression Analysis for MTO Hiley Formula for Timber Piles

Pile Type	Formula	Coefficient Condition	Coefficient of restitution (e)	Hammer Efficiency (e_f)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	R = $(n*e_f*WgH)$ / $(s+{}^{c}/_{2})$	Original Value	0.25	0.75	0.4344	y = 0.1704x + 517.38 y = 0.6685x	0.2275 0.8941	12
		Solving for e Coefficient	0	0.75	0.4274	y =0.1727x + 517.54 y = 0.6784x	0.2257 0.8944	
		Solving for e_f Coefficient	0.25	0.5607	0.3503	y = 0.228x + 517.38 y = 0.8941x	0.2275 0.8941	
		Solver Values	0	0.5689	0.3497	y = 0.2277x + 517.54 y = 0.8944x	0.2257 0.8944	
		1:1 Slope	1	0.1028	3.5772	y = 0.9999x + 521.21	0.2403	
			0	0.5088	0.3698	y = 1.0001x	0.8944	
	R = $(n*e_f*WgH)$ / $(s+{}^{c}/_{2})$ + 522	Best Fit Equation	1	0.1028	0.1658	y = 0.9999x + 521.21	0.2403	
Diesel	R = $(n*e_f*E)$ / $(s+{}^{c}/_{2})$	Original Value	0.25	1	0.8459	y = -0.1097x + 1009.2 y = 0.2566x	0.3306 0.6602	12
		Solving for e Coefficient	0.1664	1	0.8459	y = -0.1103x + 1007.3 y = 0.2588x	0.3319 0.6565	
		Solving for e_f Coefficient	0.25	0.3886	0.6438	y = -0.2822x + 1009.2 y = 0.6602x	0.3306 0.6602	
		Solver Values	1.000	0.2808	0.5643	y = -0.3135x + 1031.8 y = 0.724x	0.2748 0.7240	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			1.000	0.2033	0.6632	y = 1x	0.7240	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	
Combined	R = $(n*e_f*WgH)$ / $(s+{}^{c}/_{2})$ R = $(n*e_f*E)$ / $(s+{}^{c}/_{2})$	Original Value	0.25	0.75/1	0.8740	y = -0.0179x + 744.14 y = 0.3102x	0.0114 0.6339	24
		Solving for e Coefficient	0	0.75/1	0.8713	y = -0.0195x + 746.01 y = 0.3156x	0.0131 0.6305	
		Solving for e_f Coefficient	0.25	0.4475	0.6417	y = -0.0531x + 756.84 y = 0.6882x	0.0176 0.6883	
		Solver Values	1	0.3373	0.6099	y = -0.0145x + 726.86 y = 0.7154x	0.0012 0.7153	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			0.8	0.264	0.7329	y = 1.0002x	0.7088	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	

Like the majority of solver determined values for the coefficient of restitution from the Hiley analysis, the MTO modified Hiley optimal coefficient of restitution value is also one. The hammer efficiencies for timber piles are also similar for all the piles overall, except those installed by diesel hammers. For drop and drop and diesel hammered timber piles together, the efficiencies which results in the lowest normalized error are 57 and 34 percent respectively, while for diesel hammers it is 28 percent, significantly lower than for the piles installed by diesel hammers.

For steel piles installed with a driving cushion the solver determined values for coefficients of restitution which gives the lowest normalized error is one and the hammer efficiencies are 57, 48, and 49 percent for drop, diesel, and combined drop and diesel hammer driven piles, respectively.

Table 46: Multi Regression Analysis for MTO Hiley Formula for Steel Piles Driven With a Cushion

Pile Type	Formula	Coefficient Condition	Coefficient of restitution (e)	Hammer Efficiency (e_f)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (n*e_f*WgH) / (s+{}^{c}/_{2})$	Original Value	0.32	0.75	0.8871	y = -0.0005x + 715.98 y = 1.1168x	2E-7 0.6700	18
		Solving for e Coefficient	0.8341	0.75	0.5214	y =0.2151x + 529.48 y = 0.7266x	0.0695 0.7757	
		Solving for e_f Coefficient	0.32	1.000	0.6827	y = -0.0004x + 715.98 y = 0.8376x	2E-7 0.6700	
		Solver Values	1.000	0.5736	0.5016	y = 0.282x + 489.8 y = 0.7907x	0.1054 0.7907	
		1:1 Slope	1	0.1617	2.5332	y = 1.0003x + 489.8	0.1054	
			1	0.4535	0.5641	y = 1x	0.7907	
	$R = (n*e_f*WgH) / (s+{}^{c}/_{2}) + 490$	Best Fit Equation	1	0.1617	0.4130	y = 1.0003x – 0.321	0.1054	
Diesel	$R = (n*e_f*E) / (s+{}^{c}/_{2})$	Original Value	0.32	1	0.6824	y = 0.2582x + 942.81 y = 0.8784x	0.0373 0.6749	29
		Solving for e Coefficient	0.6035	1	0.5354	y = 0.4249x + 476.7 y = 0.6551x	0.1198 0.7554	
		Solving for e_f Coefficient	0.32	1.000	0.6824	y = 0.2582x + 942.81 y = 0.8784x	0.0373 0.6749	
		Solver Values	1.000	0.4755	0.4518	y = 1.0003x - 312.65 y = 0.815x	0.2929 0.8090	
		1:1 Slope	1	0.4755	0.4518	y = 1.0001x – 312.65	0.2929	
			1	0.3875	0.5023	y = 1.0001x	0.8090	
	$R = (n*e_f*E) / (s+{}^{c}/_{2}) - 312$	Best Fit Equation	1	0.4755	0.5062	y = 1.0001 – 0.6145	0.2929	
Combined	$R = (n*e_f*WgH) / (s+{}^{c}/_{2})$ $R = (n*e_f*E) / (s+{}^{c}/_{2})$	Original Value	0.32	0.75/1	0.7515	y = 0.4717x + 650.51 y = 0.9013x	0.1332 0.6701	47
		Solving for e Coefficient	0.6159	0.75/1	0.5850	y = 0.4277x + 444.98 y = 0.6656x	0.2455 0.7472	
		Solving for e_f Coefficient	0.32	1	0.7071	y = 0.392x + 649.29 y = 0.8719x	0.1066 0.6740	
		Solver Values	1.000	0.4910	0.4896	y = 0.6782x + 195.57 y = 0.8040x	0.3531 0.8041	
		1:1 Slope	1	0.333	0.6791	y = 1x + 195.57	0.3531	
			1	0.3948	0.5460	y = 1x	0.8041	
	$R = (n*e_f*WgH) / (s+{}^{c}/_{2}) + 196$ $R = (n*e_f*E) / (s+{}^{c}/_{2}) + 196$	Best Fit Equation	1	0.333	0.5214	y = 1x – 0.4259	0.3531	

To obtain trend lines with a slope of one; the hammer efficiencies for drop, diesel, and combined drop and diesel installed piles changes to 16, 48, and 33 percent, respectively. Although the magnitude of normalized errors increases, the coefficients of determination values remain constant.

The final analysis is performed on piles that are driven without caps. The values of the coefficient of restitution and hammer efficiency which gives the lowest normalized error values are similar to those of the other analyses performed with data derived from the MTO modified Hiley formula, being one for the coefficient of restitution and hammer efficiencies of 0.35 and 0.42 for diesel and combined drop and diesel hammered piles, respectively. The only exception to this is the optimal hammer efficiency of drop hammered piles which is set to 0.64. It is determined that no combination of coefficient of restitution and hammer efficiency values could achieve a trend line with a slope of one. Although the coefficient of determination values are relatively high, this should not be taken as an indication of the accuracy of the dynamic formula, but rather it is more likely due to the low number of samples.

Table 47: Multi Regression Analysis for MTO Hiley Formula for Steel Piles Driven Without a Cushion

Pile Type	Formula	Coefficient Condition	Coefficient of restitution (e)	Hammer Efficiency (e_f)	Norm. Err.	Equation of Straight Line	R^2 Value	Sample Set
Drop	$R = (n*e_f*WgH) / (s+{}^{c}/_{2})$	Original Value	0.55	0.75	0.4177	y = 5.4265x - 4666.1 y = 1.2321x	1 0.9271	2
		Solving for e Coefficient	0.8610	0.75	0.2479	y = 3.0963x – 3258.3 y = 0.9227x	1 0.9396	
		Solving for e_f Coefficient	0.55	0.9968	0.2710	y = 4.0829x – 4666.1 y = 0.9271x	1 0.9271	
		Solver Values	1	0.6383	0.2382	y = 2.9031x – 2880.3 y = 0.9440x	1 0.9440	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			0.7788	0.7500	0.2616	y = 1x	0.9336	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	
Diesel	$R = (n*e_f*E) / (s+{}^{c}/_{2})$	Original Value	0.55	1	0.8093	y = -0.229x + 1767.4 y = 0.3610x	0.3464 0.5396	4
		Solving for e Coefficient	0.7309	1	0.7923	y = -0.2043x + 1836.9 y = 0.3146x	0.302 0.6120	
		Solving for e_f Coefficient	0.55	0.6689	0.7618	y = -0.3423x + 1767.4 y = 0.5396x	0.3464 0.5396	
		Solver Values	1	0.3507	0.5811	y = -0.4616x + 1913.6 y = 0.6948x	0.2279 0.6948	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			1	0.2436	0.6973	y = 1.0002x	0.6948	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	
Combined	$R = (n*e_f*WgH) / (s+{}^{c}/_{2})$ $R = (n*e_f*E) / (s+{}^{c}/_{2})$	Original Value	0.55	0.75/1	0.8157	y = -0.1785x + 1607.5 y = 0.43302x	0.1843 0.5252	6
		Solving for e Coefficient	0.7016	0.75/1	0.8016	y = -0.1452x + 1600 y = 0.3793x	0.1494 0.5675	
		Solving for e_f Coefficient	0.55	0.7614	0.7371	y = -0.2544x + 1660.8 y = 0.5725x	0.1884 0.5725	
		Solver Values	1	0.4207	0.6143	y = -0.2299x + 1607 y = 0.6773x	0.0884 0.6774	
		1:1 Slope	N/A	N/A	N/A	N/A	N/A	
			1	0.2849	0.7465	y = 1.0002x	0.6774	
	N/A	Best Fit Equation	N/A	N/A	N/A	N/A	N/A	

From the analysis it becomes clear that no revised c coefficient, pile parameters, soil compression, or coefficient of restitution values, either by themselves or in conjunction with each other can result in a dynamic formula which accurately predicts pile bearing capacity and results in a minimal scatter of the data. For the ENR and Gates formulae, the modification found which produces the most accurate pile predictions is to add a positive or negative term to the end of the formula similar to the modified Gates formula. This forces the trend line to fall on the 1:1 ideal line for predictions using piles from the MTO database, however adding a term which shifts the trend line left or right does not make physical sense since it implies that a pile which is not driven, a set equal to zero, can already support some load, either positive or negative.

5.3.4 Predictive Capacities Removing Safety Factors

As discussed in Section 4.2, the dynamic formulae in this study contain various factors of safety. The factor of safety in the ENR, Gates and FHWA modified Gates formula is six, three and three, respectively. The Hiley and MTO modified Hiley formula do not contain any factor of safety thus the predicted capacity is meant to represent the ultimate bearing capacity of the pile while the other formulae give the safe working capacity of the pile. Additionally, the bearing capacities derived from the MTO pile load tests are taken as the estimated failure loads thus the ultimate capacity that the pile can withstand. To allow for a more direct comparison between the dynamic formulae themselves and to the pile load test results the multi regression analysis was repeated with the factors of safety omitted from each formula.

Figures 64 to 66 shows the predicted pile versus the estimated failure load tested capacities without any applied factors of safety and Tables 48 to 53 gives the statistics of the predicted dynamic capacities to the measured capacities derived from pile load tests. The analysis is not performed for Hiley and MTO modified Hiley derived capacities since these formulae do not contain any factors of safety and thus the results would be redundant.

Table 48: Best fit lines and Coefficients of Determination of Pile Capacities without Safety Factors

Hammer	ENR Formula	Gates Formula	FHWA Modified Gates Formula
All	y = 0.0834x + 594.5	y = 0.9806x – 12.711	y = 0.167x + 114.97
	R^2 = 0.4139	R^2 = 0.4472	R^2 = 0.4382
Drop	y = 0.4195x + 447.24	y = 0.8166x + 195.6	y = 0.1539x + 264.12
	R^2 = 0.1505	R^2 = 0.2757	R^2 = 0.2757
Diesel	y = 0.1041x + 386.83	y = 1.3986x – 600.47	y = 0.2477x – 490.23
	R^2 = 0.4239	R^2 = 0.4663	R^2 = 0.4663

Table 49: Ratio Dynamic Predicted to Field Estimated Pile Capacities without Safety Factors

Hammer	ENR Formula	Gates Formula	FHWA Modified Gates Formula
Maximum			
Drop	2.100	1.847	8.686
Diesel	21.490	4.688	24.466
Minimum			
Drop	0.431	-0.568	-8.014
Diesel	2.142	0.519	2.766
Average			
Drop	1.012	0.933	4.159
Diesel	7.673	1.513	7.935
All	4.905	1.272	6.366

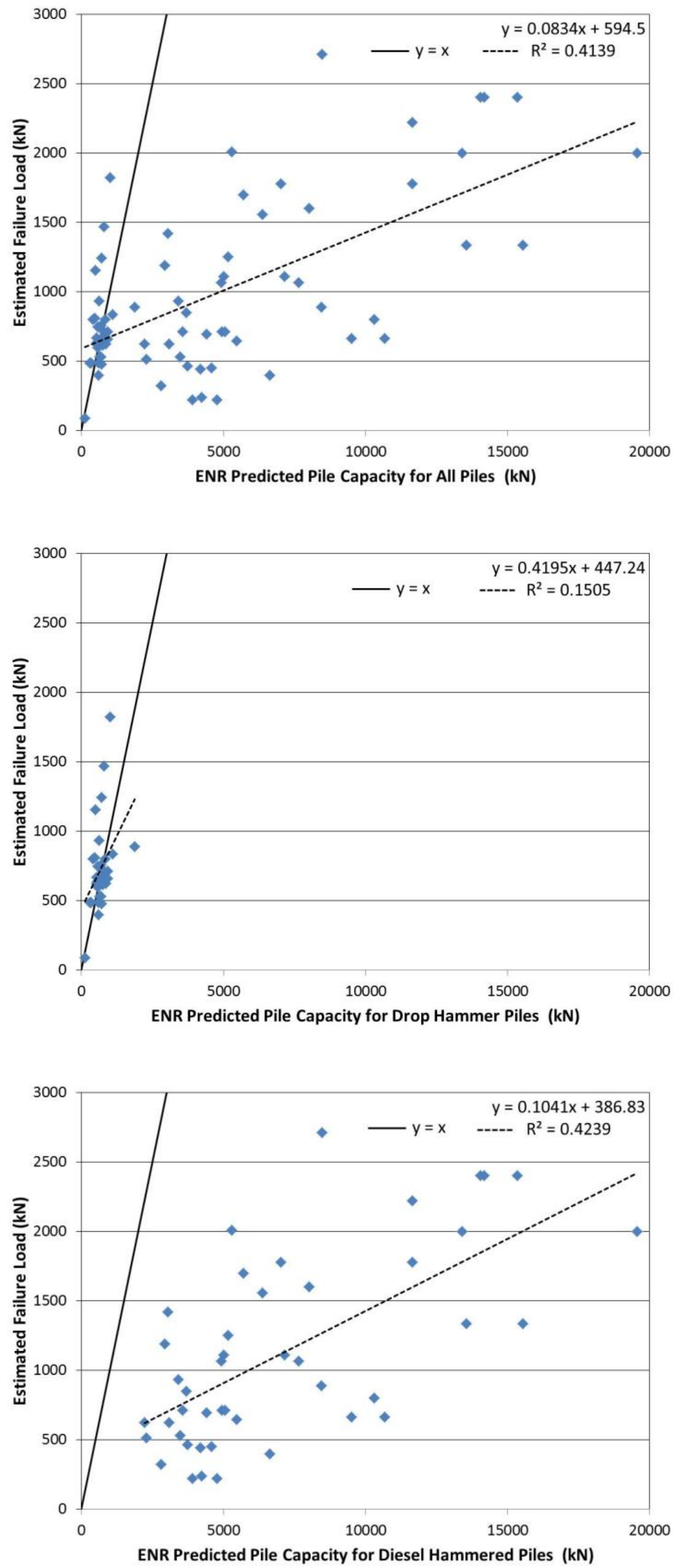
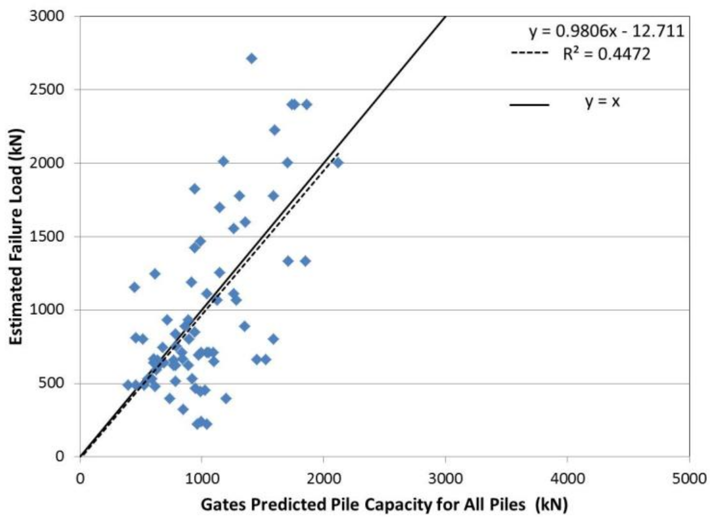
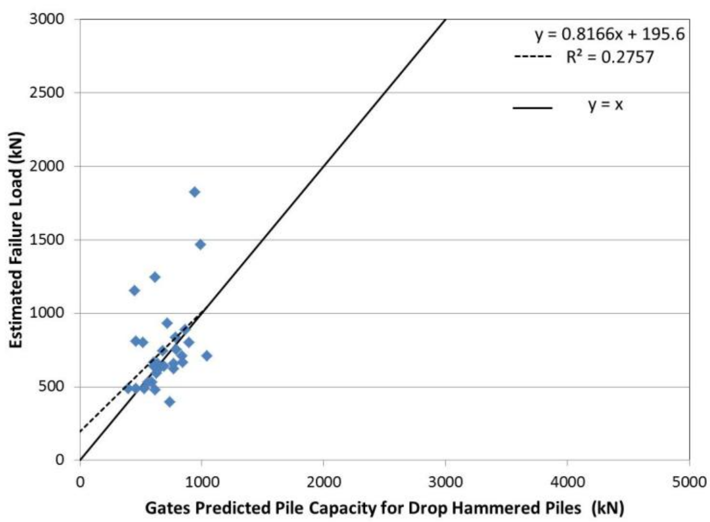


Figure 64: ENR formula predicted capacities without safety factors vs. pile test failure loads





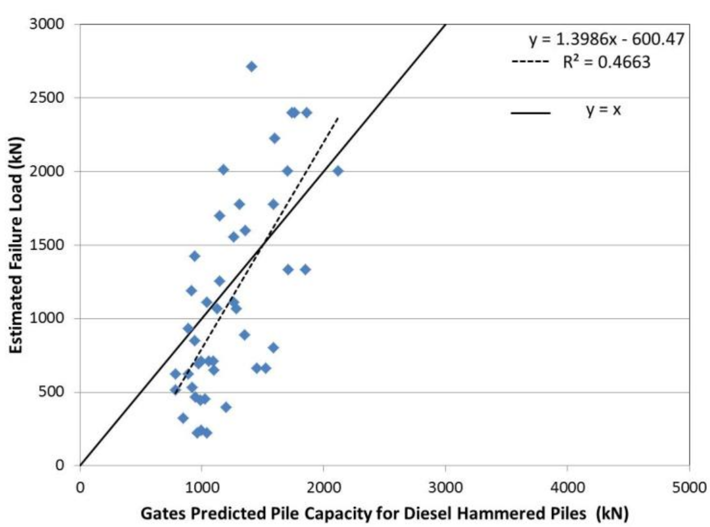
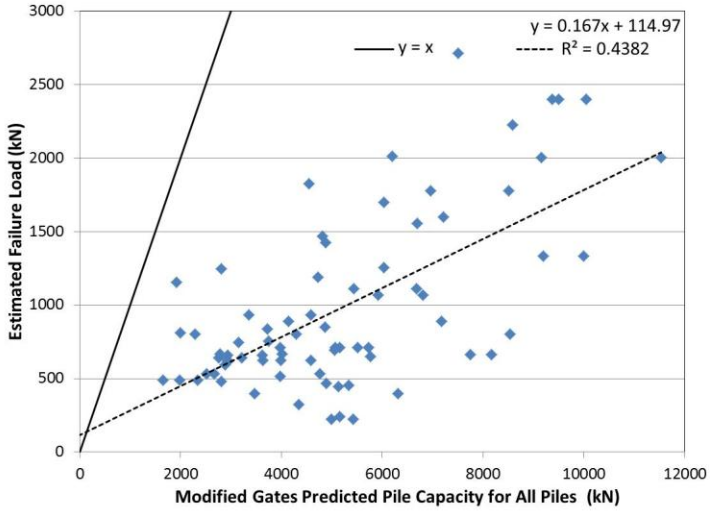
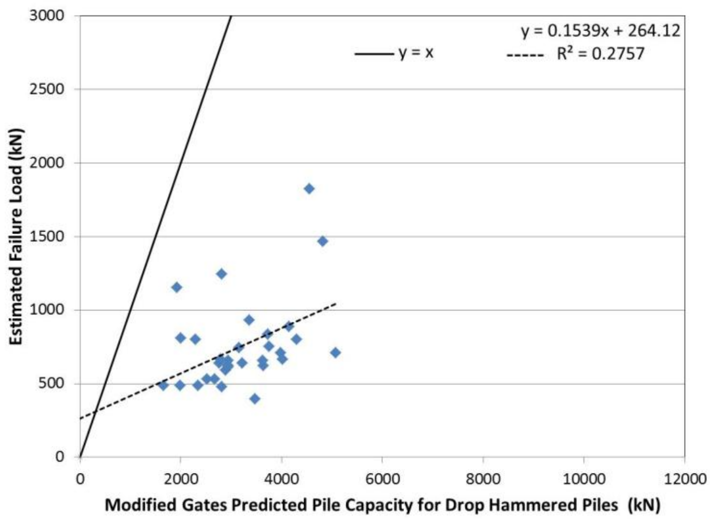


Figure 65: Gates formula predicted capacities without safety factors vs. pile test failure loads





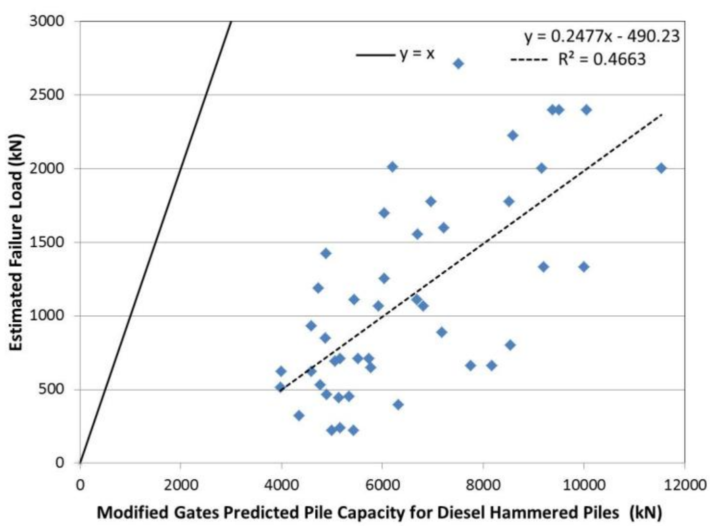


Figure 66: FHWA Modified Gates formula predicted capacities without safety factors vs. pile test failure loads

Table 50: Percent Difference of Predicted to Field Estimated Pile Capacities without Safety Factors

Hammer	ENR Formula	Gates Formula	FHWA Modified Gates Formula
Maximum			
Drop	110.047	84.721	768.560
Diesel	2049.029	368.759	2346.571
Minimum			
Drop	-56.913	-156.797	-901.384
Diesel	114.250	-48.119	176.564
Average			
Drop	1.166	-6.679	315.917
Diesel	667.294	51.250	693.497
All	390.462	27.176	536.581

Table 51: Standard Deviations of Predicted to Field Estimated Pile Capacities without Safety Factors

St. Dev.	ENR Formula	Gates Formula	FHWA Modified Gates Formula
Drop	0.376	0.400	2.635
Diesel	4.599	0.973	5.055
All	4.819	0.834	4.597

Table 52: Correlation of Predicted to Field Estimated Pile Capacities without Safety Factors

Hammer	ENR Formula	Gates Formula	FHWA Modified Gates Formula
Drop	0.388	0.525	0.525
Diesel	0.651	0.683	0.683
All Piles	0.643	0.669	0.662

Table 53: Coefficients of Variation of Predicted to Field Estimated Capacities without Safety Factors

C of V	ENR Formula	Gates Formula	FHWA Modified Gates Formula
Drop	0.372	0.428	0.634
Diesel	0.599	0.644	0.637
All	0.983	0.656	0.722

From the above Tables and Figures it is seen that removing the safety factors from the dynamic formulae increases the average predicted capacities by an approximate factor of three to five when compared to the original dynamic formulae derived capacities. For the ENR and FHWA modified Gates formulae this causes the predicted capacities to be greatly over estimated.

However the Gates formula, which significantly underestimated pile capacities specifically as the estimated failure loads becomes relatively large, now is comparatively accurate. This is especially true when all piles are examined simultaneously. For only drop hammered piles, the best fit trend line slightly underestimates the estimated field capacity while for diesel hammered piles, on average, the predicted capacities are overestimated for piles which can support loads less than 1500 kN and overestimated for piles which can support loads greater than 1500 kN.

On average the percent difference between the predicted and field estimated bearing capacities range from 27 to 546 percent and the standard deviation values ranges from 0.8 to slightly greater than 4.5. The former is from capacities derived from the Gates formula while the other latter is from the ENR and FHWA modified Gates formulae.

Despite the best fit trend line closely matching the 1:1 line from the Gates formula by removing the factor of safety and the greatly increased predicted capacities from the ENR and FHWA modified Gates formulae, the average correlation coefficient from all three formulae are approximately constant at values of 0.64 to 0.66. However, the coefficient of variation increases by as much as 1.5 times.

For consistency and to improve the predictive capabilities of the dynamic formulae, the analysis of predicted bearing capacities without the factors of safety was then by using Solver within Excel to determine the optimal value for various coefficients. For the ENR formula, the value of the c coefficient which results in the most accurate prediction of pile capacity was determined by Solver. For the Gates formula the $^{1}/_{7}{}^{th}$ term at the front of the formula is varied. For the FHWA modified Gates formula the 1.75 and -100 coefficients are varied independently and then simultaneously. For the Hiley formula the e and C3 coefficients are varied independently and then simultaneously. For the MTO modified Hiley Formula the e and e_f coefficients are varied independently and then simultaneously. To be more comparable to the results from the Hiley formula, the e and C coefficients of the MTO modified Hiley formula are varied as well, first independently and then simultaneously.

The results from the analysis are given in Figures 67 to 79 and Tables 56 to 61 below.

5.3.5 Multi Regression Analysis Without Safety Factors

As with the original dynamic formulae; the analysis of the formulae without any safety factors is performed a second time by varying specific parameters to determine whether or not the accuracy of the predictive capacities of the formulae can be significantly improved.

The coefficients that are altered are the same as in section 5.3.3 and are presented in Tables 54 and 55 below. The revised values are determined by the Solver function of Excel in the same method as previously performed.

Table 54 presents the coefficients which are altered independently; where two coefficients are altered first one is varied while the other remains constant, then the other is varied while the first is set to its original value.

Table 55 presents the determined coefficient values where both coefficients are varied simultaneously which results in improving the accuracy of the predicted pile bearing capacities.

Table 54: Summary of Independently Varied Dynamic Formulae Coefficients

Formula	ENR		Gates		FHWA Modified Gates		Hiley		MTO Modified Hiley	
	Drop	Diesel	Drop	Diesel	Drop	Diesel	Drop	Diesel	Drop	Diesel
Coefficient	c	c	-		-	-	e/C3	e/C3	e_f/e/C	e_f/e/C
Original Value	1	0.1	$^1/_7$		1.75/100	1.75/100	0.5/See Tables 16 to 18	0.5/See Tables 16 to 18	0.75/0.25, 0.32, 0.55 /See Tables 16 to 18	1/0.25, 0.32, 0.55 /See Tables 16 to 18
Varied Value	0.70	1.10	0.18	0.16	0.63/348	0.44/1234	0.80/0.12	0.78/0.48	0.89/0.81 /8.34	0.99/0.77 /19.37

Table 55: Summary of Simultaneously Varied Dynamic Formulae Coefficients

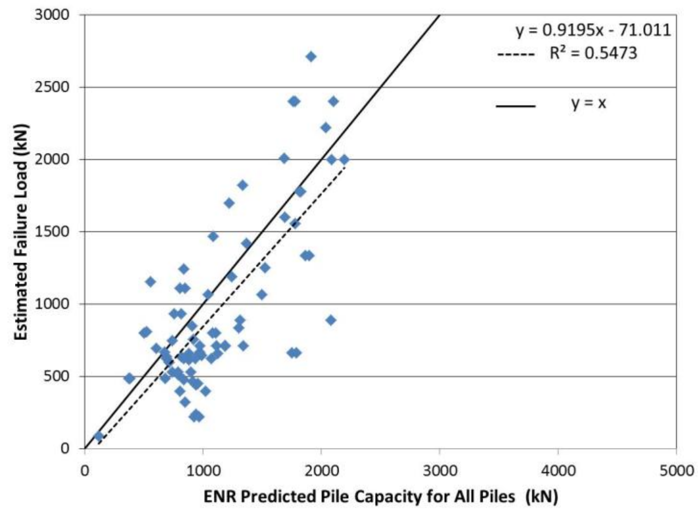
Formula	FHWA Modified Gates		Hiley		MTO Modified Hiley			
	Drop	Diesel	Drop	Diesel	Drop	Diesel	Drop	Diesel
Coefficient	-	-	e/C3	e/C3	e/e_f	e/e_f	e/C	e/C
Original Value	1.75/100	1.75/100	0.5/See Tables 16 to 18	0.5/See Tables 16 to 18	0.25, 0.32, 0.55/0.75	0.25, 0.32, 0.55/1	0.25, 0.32, 0.55/See Tables 16 to 18	0.25, 0.32, 0.55/See Tables 16 to 18
Varied Value	0.27/-66	0.69/344	1/0.58	1/1.55	1/0.54	1/0.44	1/21.66	1/55.66

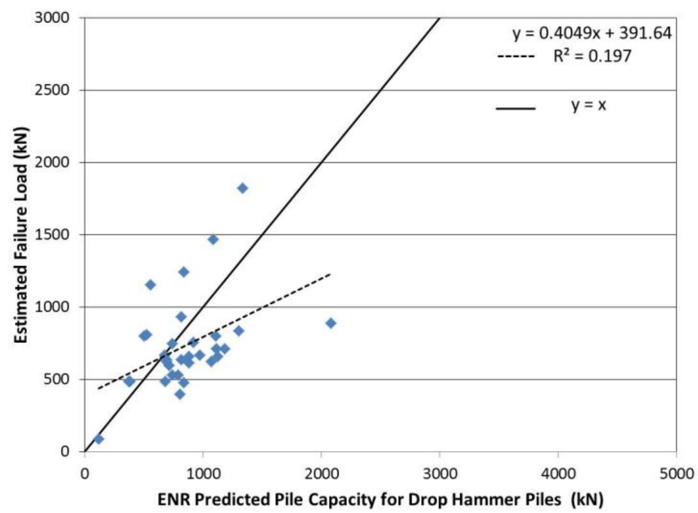
The results of the analysis using the values from the above Tables are shown in Figures 67 to 79. Statistical analysis consisting of determining the coefficients of determination, trend lines, percent differences, standard deviations, correlation and coefficients of variation values from predicted to field measured values are also calculated and presented in Tables 56 to 61.

Table 56: Best fit lines and Coefficients of Determination from MRA without Safety Factors

Hammer	c ENR Formula	$^1/_7$ Gates Formula	1.75 FHWA Modified Gates Formula	100 FHWA Gates Formula
All	y = 0.9195x – 71.011	y = 0.9304x – 101.13	y = 0.8016x + 76.291	y = 0.2069x + 616.97
	R^2 = 0.5473	R^2 = 0.4593	R^2 = 0.4628	R^2 = 0.3178
Drop	y = 0.4049x + 391.64	y = 0.6632x + 195.6	y = 0.4258x + 385.12	y = 0.1539x + 434.07
	R^2 = 0.197	R^2 = 0.2757	R^2 = 0.2757	R^2 = 0.2757
Diesel	y = 1.1732x – 423.84	y = 1.2511x – 600.47	y = 0.9844x – 162.32	y = 0.2477x + 760.32
	R^2 = 0.6187	R^2 = 0.4663	R^2 = 0.4663	R^2 = 0.4663
1.75/100 FHWA Gates	e Hiley Formula	C3 Hiley Formula	e/C3 Hiley Formula	e_f MTO Mod Hiley Formula
y = 0.6753x + 245.32	y = 0.1405x + 695.21	y = 0.1624x + 787.15	y = 0.8244x + 33.533	y = 0.0134x + 948.71
R^2 = 0.4844	R^2 = 0.0745	R^2 = 0.026	R^2 = 0.3824	R^2 = 0.0004
y = 0.9993x - 98.265	y = 0.2598x + 486.95	y = 0.1704x + 606.69	y = 0.4035x + 404.25	y = 0.1422x + 618.35
R^2 = 0.2757	R^2 = 0.1234	R^2 = 0.0412	R^2 = 0.1746	R^2 = 0.0404
y = 0.6258x + 358.13	y = 0.0105x + 1101.8	y = -0.0491x + 1195.3	y = 1.1135x – 373.36	y = -0.1327x + 1348.6
R^2 = 0.4663	R^2 = 0.0002	R^2 = 0.0018	R^2 = 0.3854	R^2 = 0.0391
e MTO Mod Hiley Formula	e/e_f MTO Mod Hiley Formula	C MTO Mod Hiley Formula	e/C MTO Mod Hiley Formula	
y = 0.1409x + 696.5	y = 0.4082x + 476.07	y = 0.3197x + 605.89	y = 0.9258x - 63.924	
R^2 = 0.0742	R^2 = 0.1407	R^2 = 0.1313	R^2 = 0.5672	
y = 0.2717x + 478.35	y = 0.3939x + 419.51	y = 0.2365x + 563.51	y = 0.4924x + 335.39	
R^2 = 0.1302	R^2 = 0.1935	R^2 = 0.0905	R^2 = 0.2679	
y = 0.0089x + 1106.2	y = 0.2657x + 735.75	y = 0.232x + 802.83	y = 1.1729x - 423.81	
R^2 = 0.0002	R^2 = 0.0341	R^2 = 0.0538	R^2 = 0.6187	

From the above Figures and Table 56 it is determined that varying the coefficients of the dynamic formulae result in the ENR and MTO modified Hiley formulae producing, on average, the largest coefficient of determination values. The parameters altered are the c coefficient for the ENR formula and the e and C coefficients simultaneously for the MTO modified formula. Varying the $^1/_7$ coefficient of the Gates formula and the 1.75, 100, and simultaneously the 1.75 and 100 coefficients the FHWA modified Gates produces slightly smaller coefficients of determination. The exception to this is for drop hammered piles for which the ENR formula produces coefficient of determination values approximately 10 percent less than those from the Gates, FHWA modified Gates, and MTO modified formulae where e and C are varied simultaneously. Every parameter and combination of parameters altered for the Hiley and MTO modified Hiley formulae result in significantly lower coefficient of determination values. The exception to this is from the simultaneously varied e and C parameters of the MTO modified Hiley formula as discussed above.





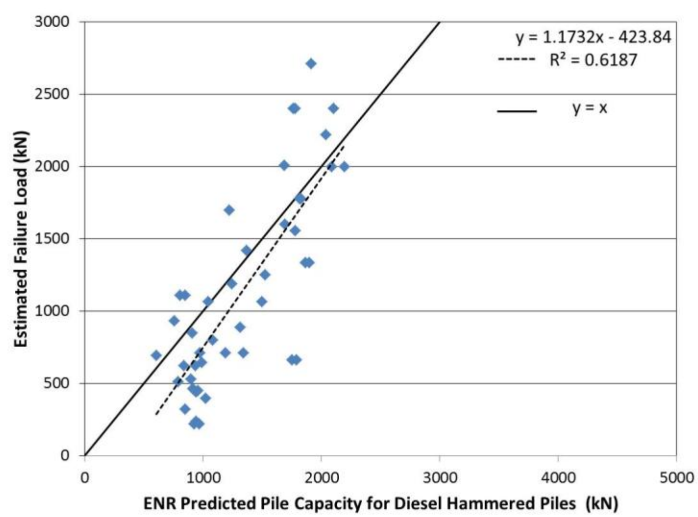
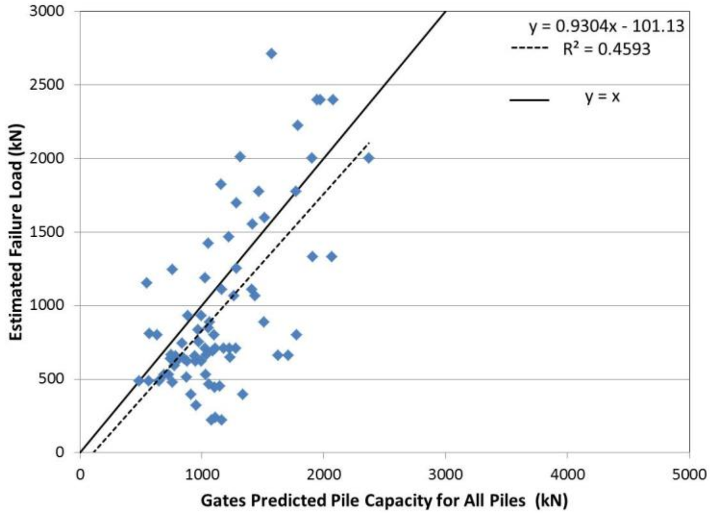
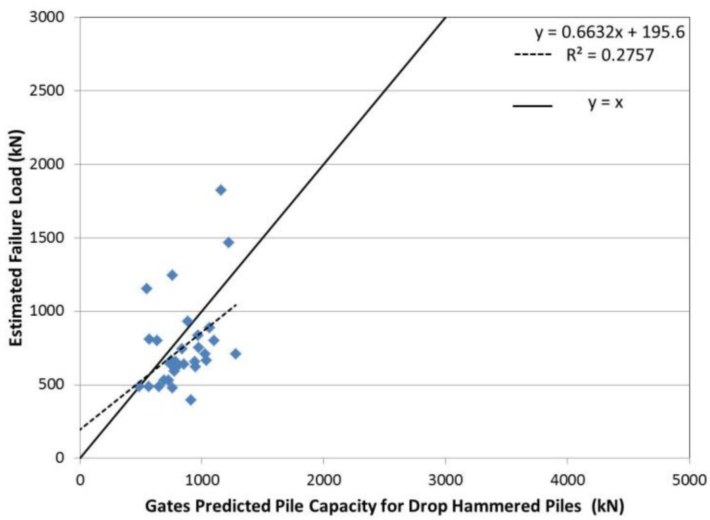


Figure 67: Predicted capacities from the Multi Regression Analysis c coefficient of the ENR formula without safety factors vs. pile test failure loads





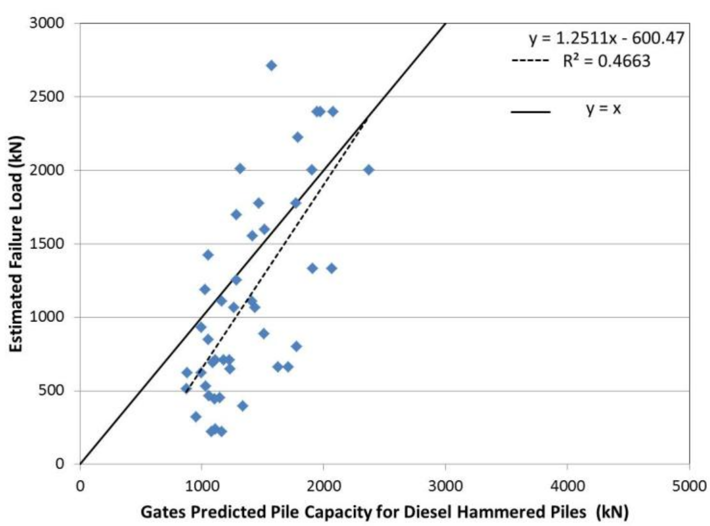


Figure 68: Predicted capacities from the Multi Regression Analysis $^{1}/_{7}{}^{th}$ coefficient of the Gates formula without safety factors vs. pile test failure loads

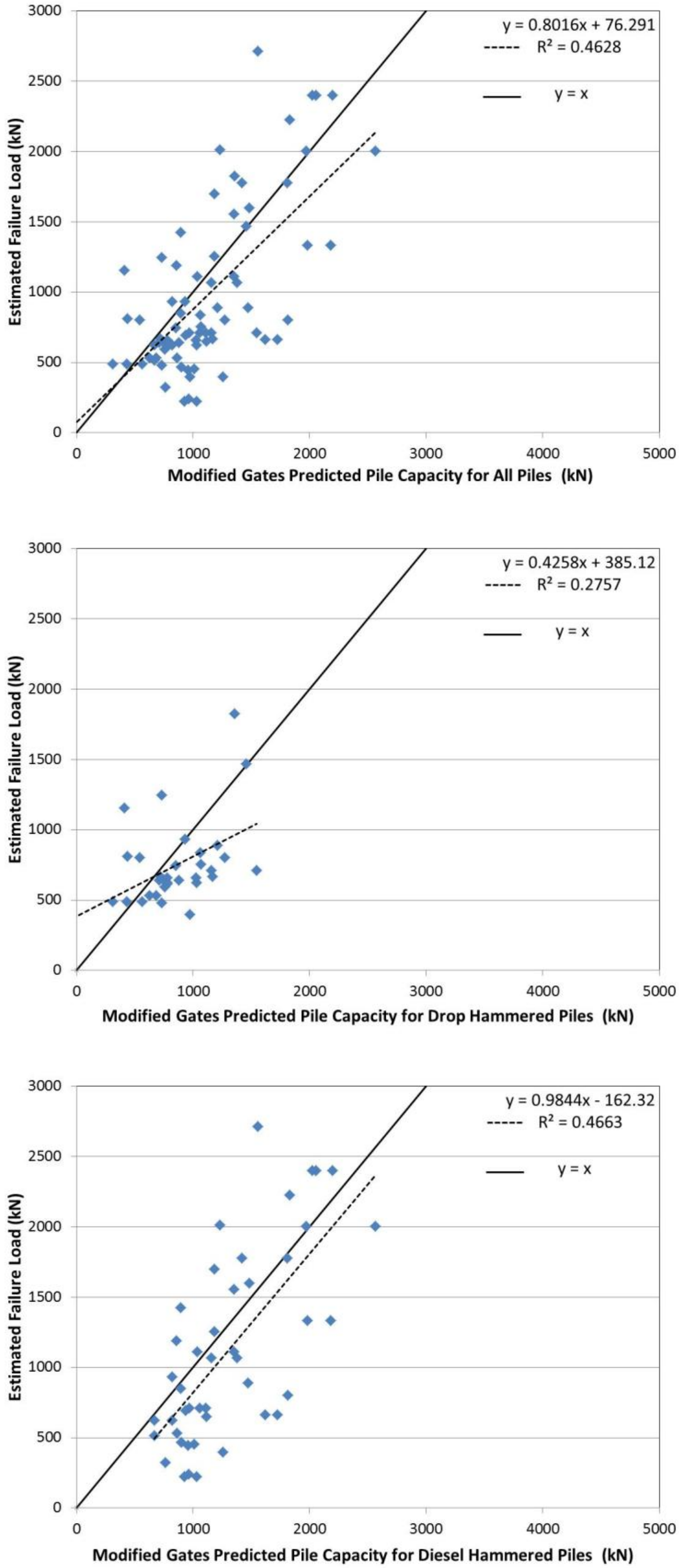


Figure 69: Predicted capacities from the Multi Regression Analysis 1.75 coefficient of the FHWA modified Gates formula without safety factors vs. pile test failure loads

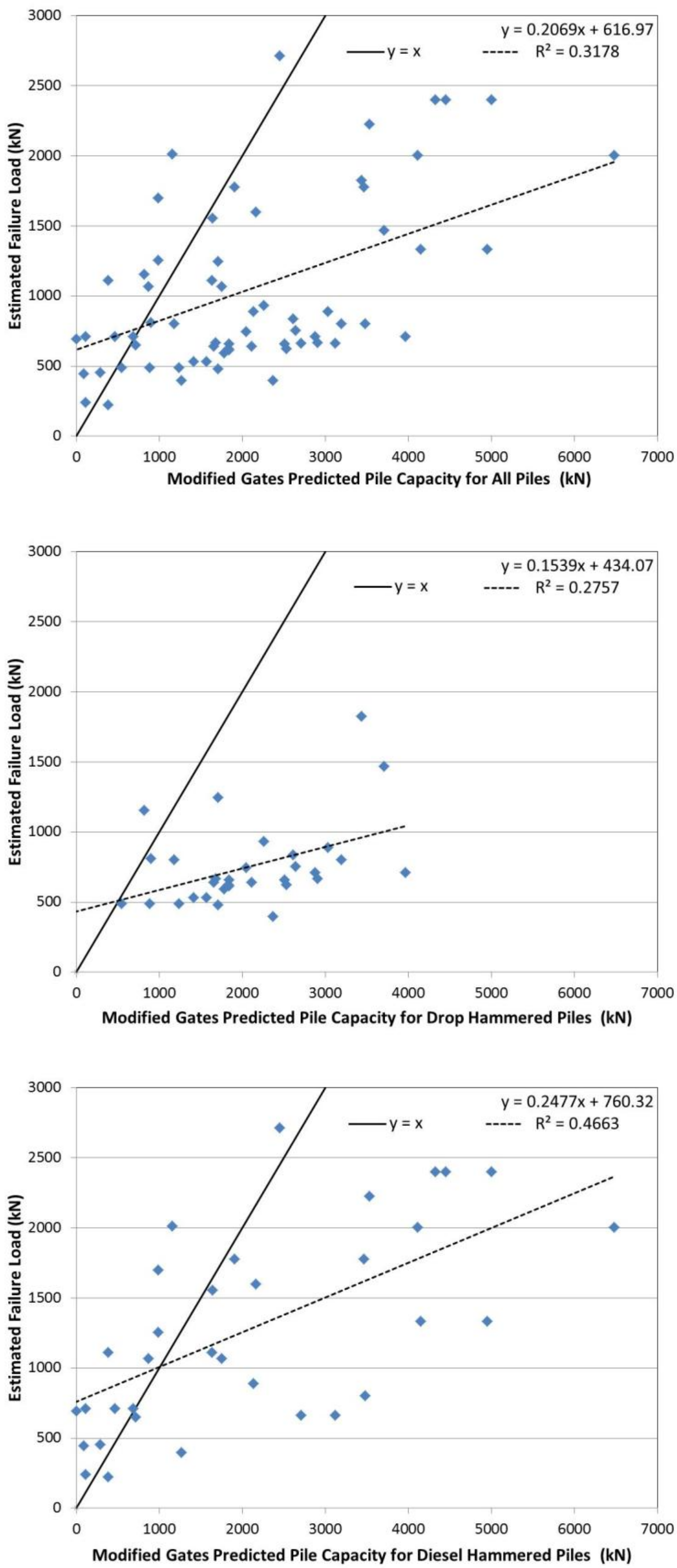


Figure 70: Predicted capacities from the Multi Regression Analysis 100th coefficient of the FHWA modified Gates formula without safety factors vs. pile test failure loads

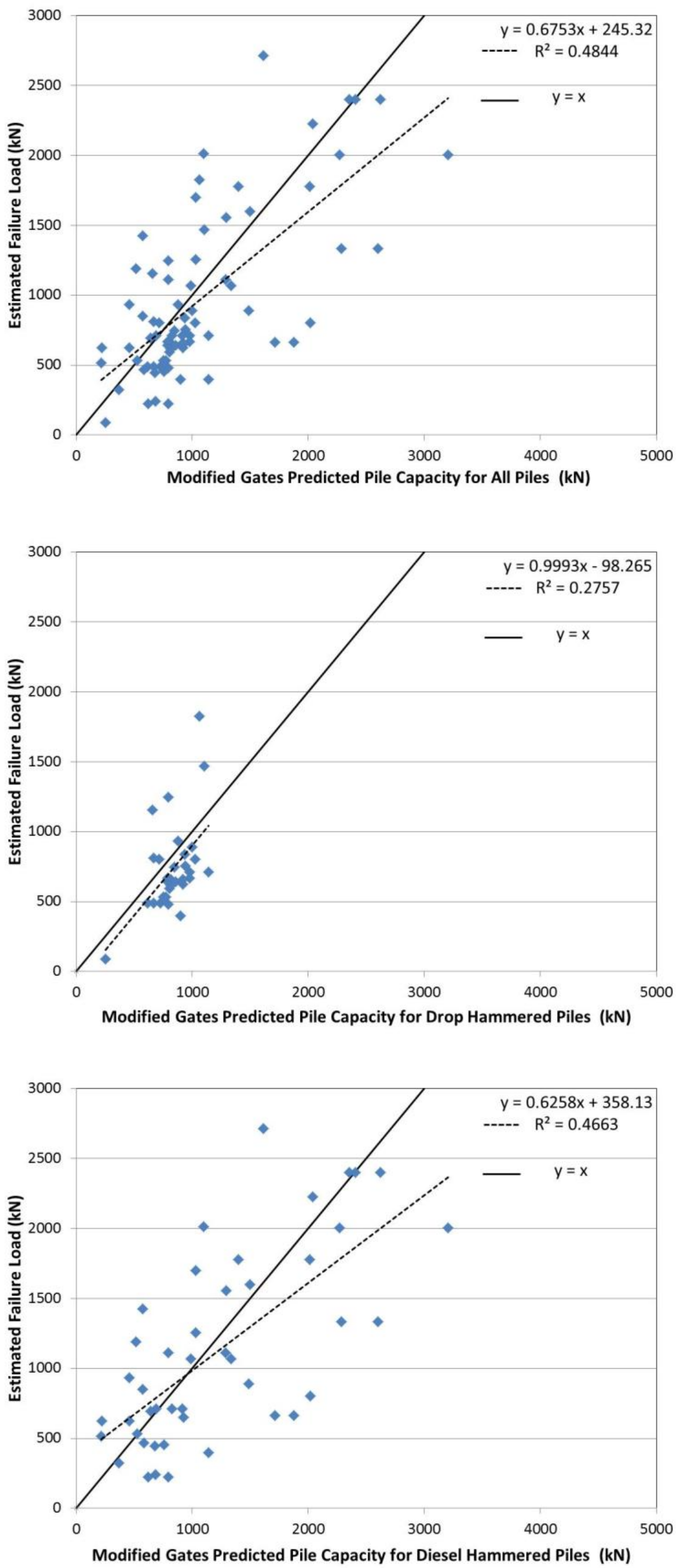


Figure 71: Predicted capacities from the Multi Regression Analysis 1.75 and 100th coefficient of the FHWA modified Gates formula without safety factors vs. pile test failure loads

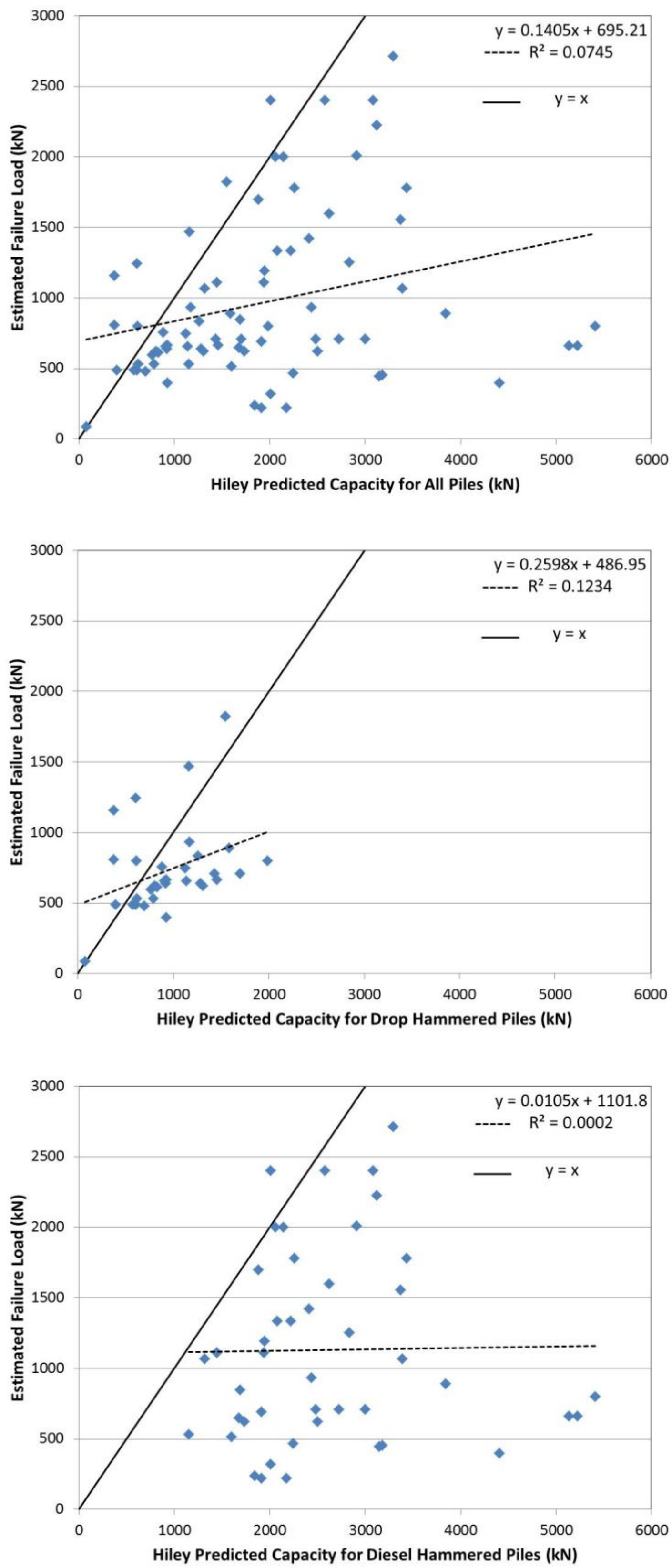


Figure 72: Predicted capacities from the Multi Regression Analysis e coefficient of the Hiley formula vs. pile test failure loads

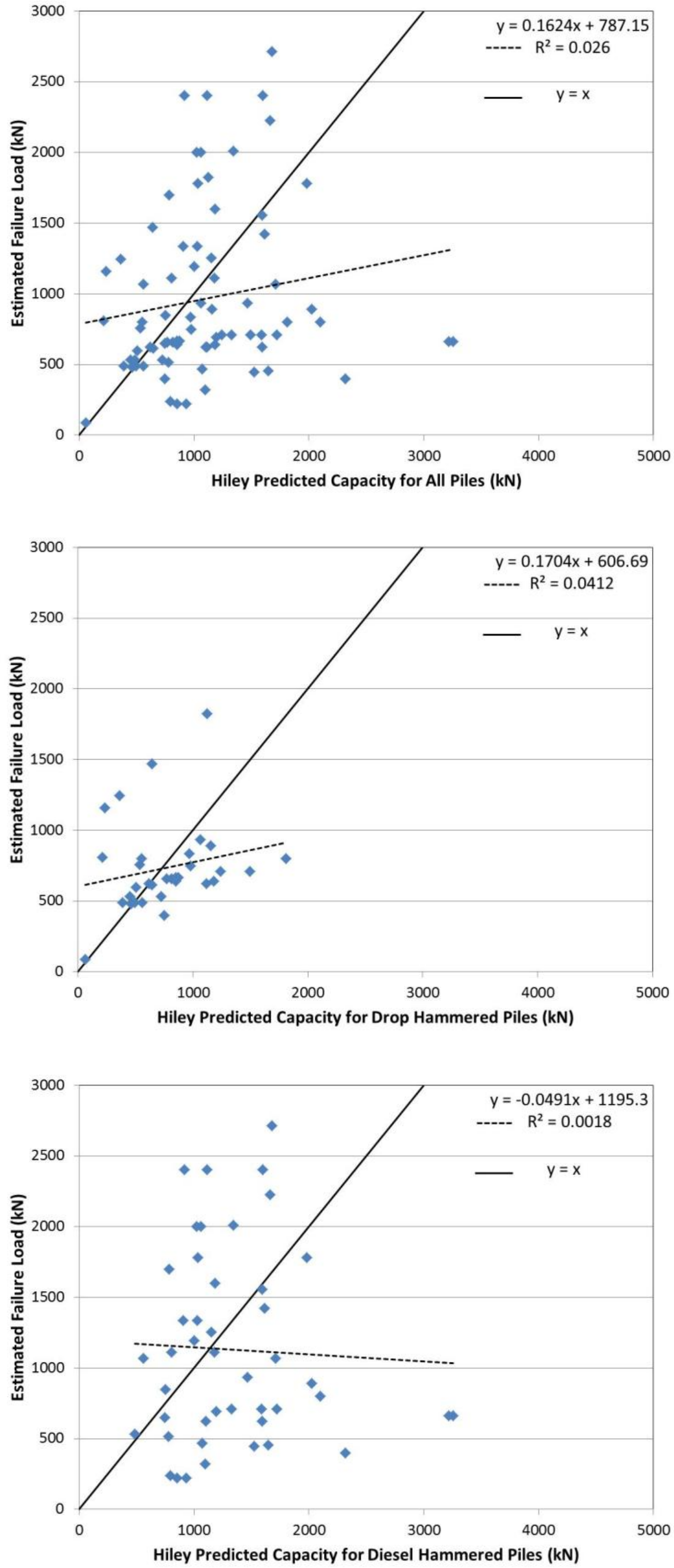


Figure 73: Predicted capacities from the Multi Regression Analysis C3 coefficient of the Hiley formula vs. pile test failure loads

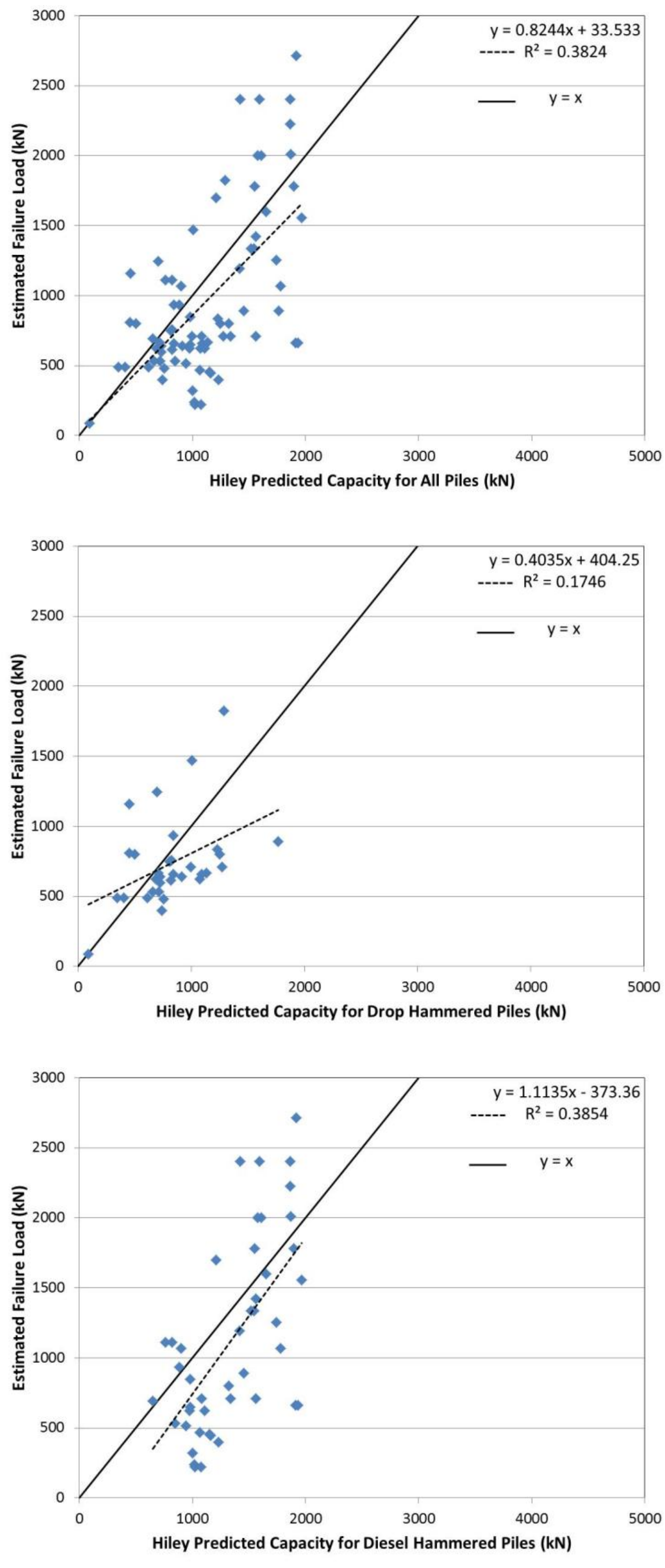


Figure 74: Predicted capacities from the Multi Regression Analysis e and C3 coefficient of the Hiley formula vs. pile test failure loads

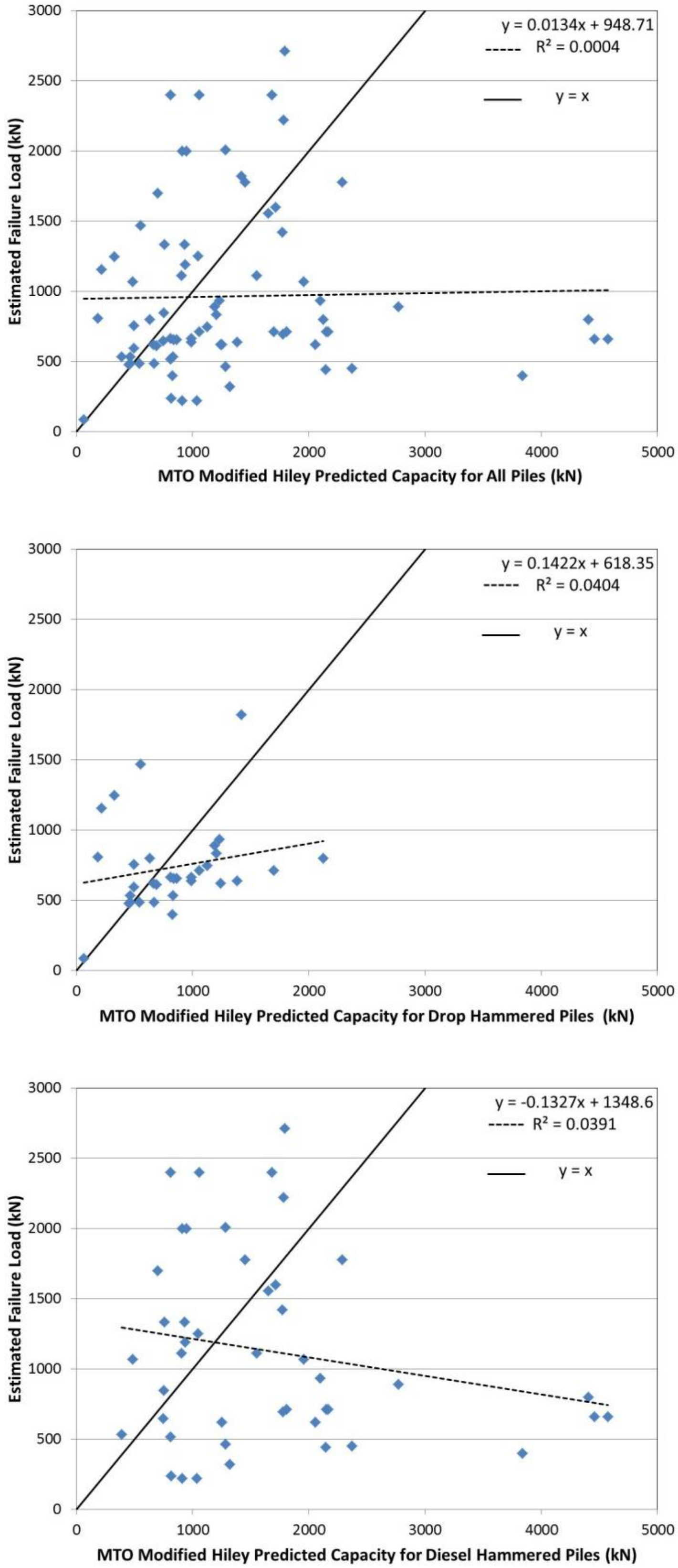


Figure 75: Predicted capacities from the Multi Regression Analysis e_f coefficient of the MTO modified Hiley formula vs. pile test failure loads

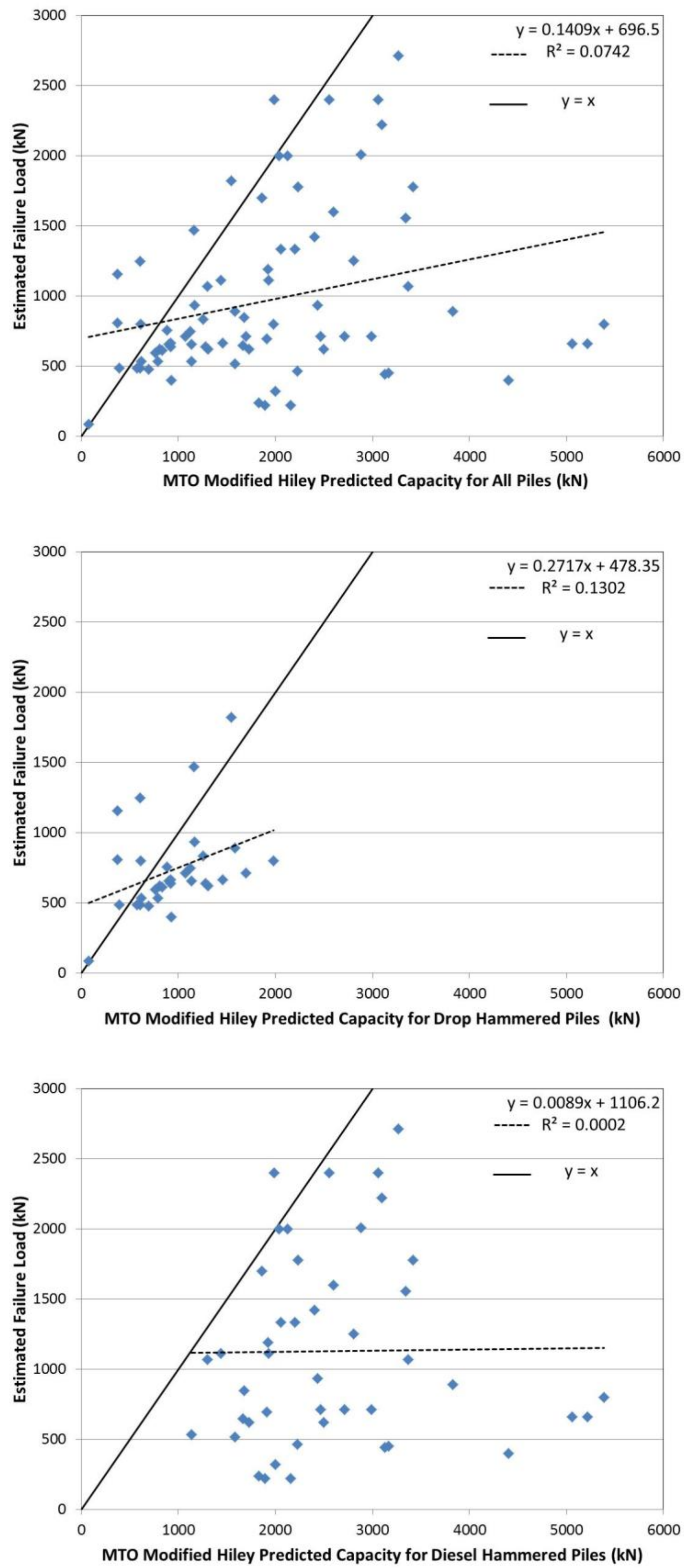


Figure 76: Predicted capacities from the Multi Regression Analysis e coefficient of the MTO modified Hiley formula vs. pile test failure loads

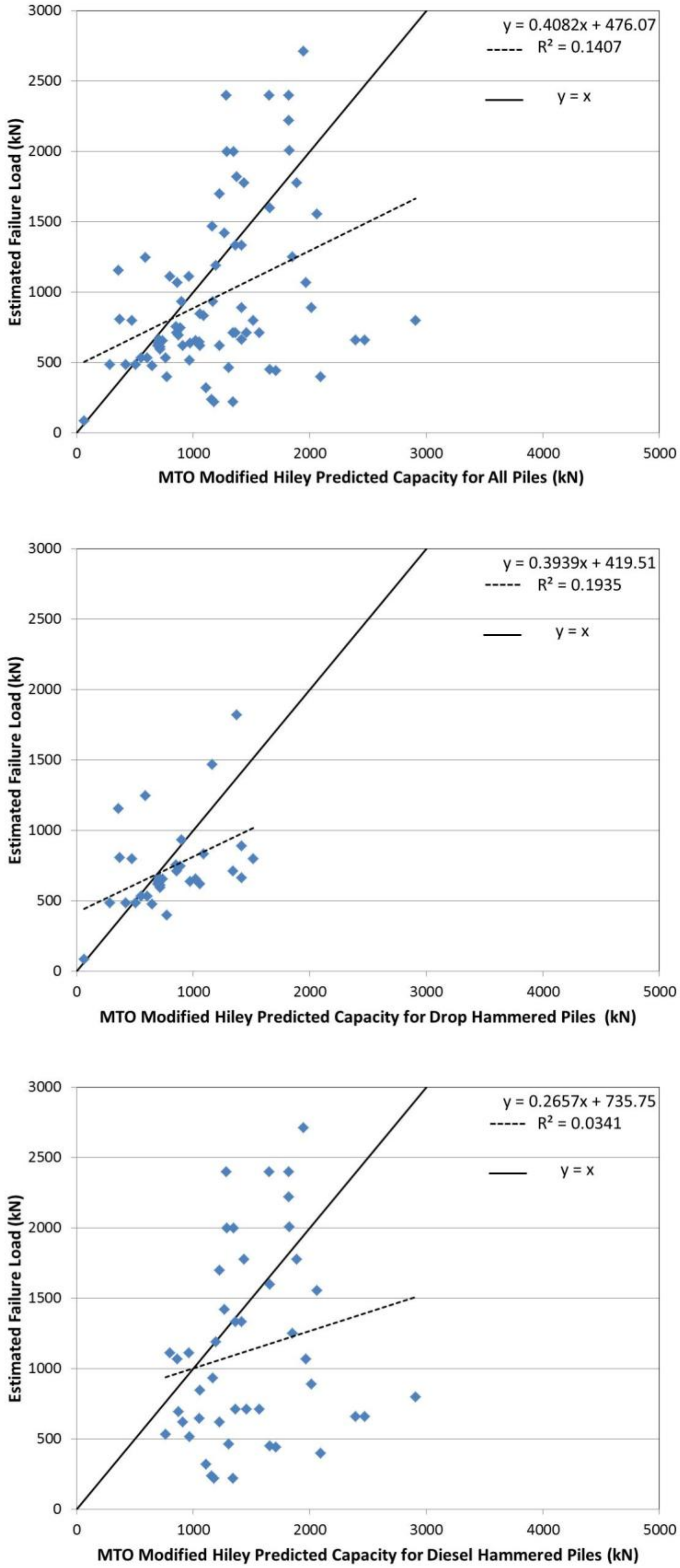


Figure 77: Predicted capacities from the Multi Regression Analysis e and e_f coefficient of the MTO modified Hiley formula vs. pile test failure loads

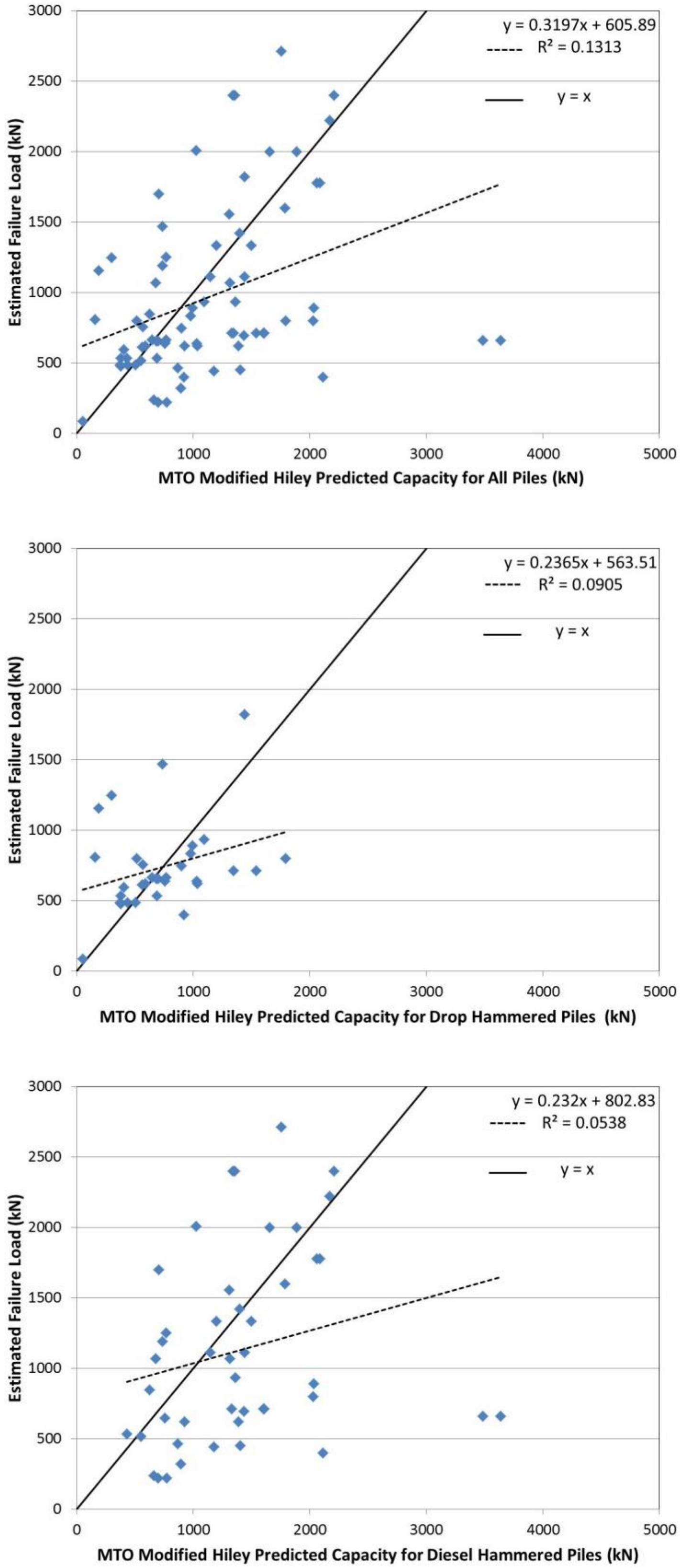


Figure 78: Predicted capacities from the Multi Regression Analysis C coefficient of the MTO modified Hiley vs. pile test failure loads

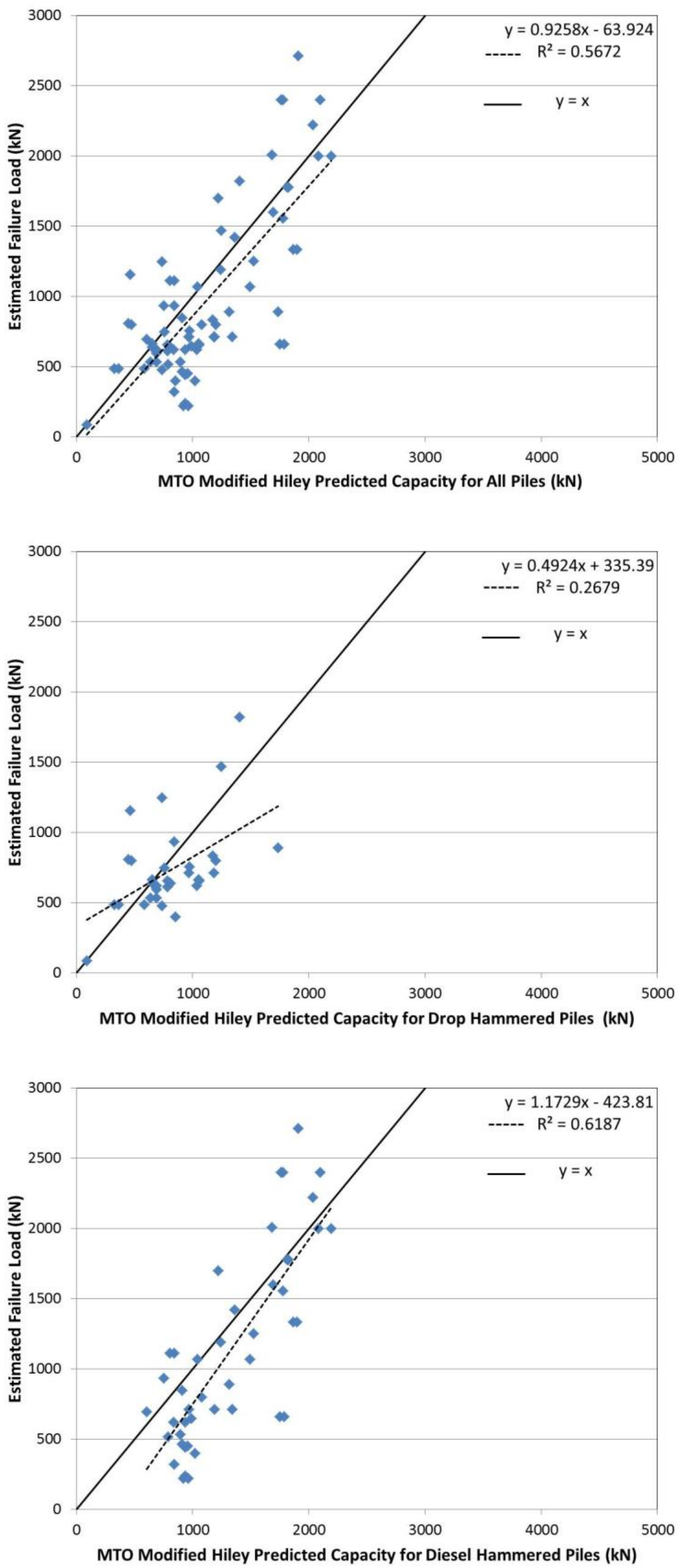


Figure 79: Predicted capacities from the Multi Regression Analysis e and C coefficient of the MTO modified Hiley factors vs. pile test failure loads

Table 57: Ratio of MRA Dynamic Predicted to Field Estimated Pile Capacities without Safety Factors

Hammer	c ENR Formula	$^1/_7$ Gates Formula	1.75 FHWA Gates Formula	100 FHWA Gates Formula
Maximum				
Drop	2.335	2.275	2.430	5.926
Diesel	4.339	5.240	4.656	4.711
Minimum				
Drop	0.479	-0.699	-6.090	-20.418
Diesel	0.704	0.580	0.573	-2.166
Average				
Drop	1.238	1.149	0.999	2.197
Diesel	1.532	1.691	1.543	1.059
All	1.410	1.466	1.317	1.532
1.75/100 FHWA Gates Formula	e Hiley Formula	C3 Hiley Formula	e/C3 Hiley Formula	e_f MTO Mod Hiley Formula
Maximum				
2.841	2.476	2.259	1.989	11.723
3.576	11.020	5.795	4.837	9.597
Minimum				
0.569	0.323	0.201	0.390	0.074
0.356	0.835	0.381	0.592	0.337
Average				
1.285	1.387	1.116	1.193	1.133
1.294	3.470	1.775	1.649	2.261
1.290	2.604	1.501	1.460	1.792
e MTO Mod Hiley Formula	e/e_f MTO Mod Hiley Formula	C MTO Mod Hiley Formula	e/C MTO Mod Hiley Formula	
Maximum				
11.723	11.723	11.723	11.723	
11.001	6.040	5.487	4.339	
Minimum				
0.074	0.074	0.074	0.074	
0.827	0.535	0.416	0.704	
Average				
1.147	1.141	1.126	1.152	
3.449	1.933	1.713	1.532	
2.492	1.604	1.469	1.374	

Examining the trend lines produced from the formulae show that varying the c, $^1/_7$, 1.75, e and C3, and e and C parameters of the ENR, Gates, FHWA modified Gates, Hiley, and MTO modified Hiley formula, respectively result in slopes near unity when diesel and combined drop and diesel hammered piles are considered. The only combination of varying parameters which results in a trend line slope near unity for drop hammered piles is when the 1.75 and 100 parameters of the FHWA modified Gates formula are altered simultaneously. However, for diesel and combined drop and diesel hammered piles altering the 1.75 and 100 parameters does not produce trend line slopes near unity for diesel and combined drop and diesel hammered piles.

Table 58: Percent Difference of MRA Predicted to Field Estimated Capacities without Safety Factors

Hammer	c ENR Formula	$^1/_7$ Gates Formula	1.75 FHWA Gates Formula	100 FHWA Gates Formula
		Maximum		
Drop	133.455	127.463	142.988	492.565
Diesel	333.910	424.044	365.565	371.072
		Minimum		
Drop	-52.112	-169.938	-709.038	-2141.811
Diesel	-29.587	-42.000	-42.688	-316.565
		Average		
Drop	23.798	14.914	-0.123	119.732
Diesel	53.183	69.089	54.303	5.945
All	40.971	46.575	31.685	53.233
100/1.75 FHWA Gates Formula	e Hiley Formula	C3 Hiley Formula	e/C3 Hiley Formula	e_f MTO Mod Hiley Formula
		Maximum		
184.067	147.636	125.884	98.353	1072.308
257.595	1001.971	479.486	383.675	859.732
		Minimum		
-43.089	-67.694	-79.927	-61.037	-92.569
-64.357	-16.546	-61.915	-40.810	-66.262
		Average		
28.466	38.654	11.593	19.319	13.260
29.436	246.951	77.480	64.921	126.079
29.033	160.386	50.099	45.969	79.193
e MTO Mod Hiley Formula	e/e_f MTO Mod Hiley Formula	C MTO Mod Hiley Formula	e/C MTO Mod Hiley Formula	
	Maximum			
1072.308	1072.308	1072.308	1072.308	
1000.138	503.987	448.729	333.910	
	Minimum			
-92.569	-92.569	-92.569	-92.569	
-17.300	-46.498	-58.384	-29.587	
	Average			
14.662	14.067	12.577	15.179	
244.874	93.291	71.335	53.183	
149.202	60.367	46.916	37.389	

Table 59: Standard Deviation of MRA Predicted to Field Estimated Capacities without Safety Factors

St. Dev.	c ENR Formula	$^1/_7$ Gates Formula	1.75 FHWA Gates Formula	100 FHWA Gates Formula
Drop	0.439	0.492	1.368	4.292
Diesel	0.892	1.088	0.948	1.579
All	0.749	0.926	1.165	3.046
1.75/100 FHWA Gates Formula	e Hiley Formula	C3 Hiley Formula	e/C3 Hiley Formula	e_f MTO Mod Hiley Formula
0.442	0.565	0.517	0.420	2.158
0.785	2.701	1.395	1.054	2.099
0.661	2.329	1.159	0.876	2.182
e MTO Mod Hiley Formula	e/e_f MTO Mod Hiley Formula	C MTO Mod Hiley Formula	e/C MTO Mod Hiley Formula	
2.161	2.160	2.158	2.163	
2.687	1.449	1.275	0.892	
2.718	1.809	1.710	1.551	

Table 60: Correlation of MRA Predicted to Field Estimated Pile Capacities without Safety Factors

Hammer	c ENR Formula	$^1/_7$ Gates Formula	1.75 FHWA Gates Formula	100 FHWA Gates Formula
Drop	0.444	0.525	0.525	0.525
Diesel	0.787	0.683	0.683	0.683
All Piles	0.740	0.678	0.680	0.564
1.75/100 FHWA Gates Formula	e Hiley Formula	C3 Hiley Formula	e/C3 Hiley Formula	e_f MTO Mod Hiley Formula
0.525	0.351	0.203	0.418	0.201
0.683	0.015	-0.043	0.621	-0.198
0.696	0.273	0.161	0.618	0.021
e MTO Mod Hiley Formula	e/e_f MTO Mod Hiley Formula	C MTO Mod Hiley Formula	e/C MTO Mod Hiley Formula	
0.361	0.440	0.301	0.518	
0.013	0.185	0.232	0.787	
0.272	0.375	0.362	0.753	

Table 61: Coefficients of Variation of MRA Predicted to Estimated Capacities without Safety Factors

C of V	c ENR Formula	$^1/_7$ Gates Formula	1.75 FHWA Gates Formula	100 FHWA Gates Formula
Drop	0.355	0.428	1.370	1.953
Diesel	0.582	0.644	0.615	1.491
All	0.531	0.631	0.885	1.988
1.75/100 FHWA Gates Formula	e Hiley Formula	C3 Hiley Formula	e/C3 Hiley Formula	e_f MTO Mod Hiley Formula
0.344	0.408	0.463	0.352	1.906
0.606	0.779	0.786	0.639	0.928
0.512	0.894	0.772	0.600	1.218
e MTO Mod Hiley Formula	e/e_f MTO Mod Hiley Formula	C MTO Mod Hiley Formula	E/C MTO Mod Hiley Formula	
1.885	1.893	1.917	1.878	
0.779	0.749	0.744	0.582	
1.091	1.128	1.164	1.129	

By examining the ratios of predicted to field estimated bearing capacities it is seen that on average varying the 1.75 and 100 parameters of the FHWA modified formula simultaneously results in predicted bearing capacities being closest to the tested field capacities. For drop and diesel hammered piles, varying the 1.75 and 100 parameters, respectively of the FHWA modified formula results in ratio closes to unity. The least accurate prediction, on average, is when the e coefficient of the Hiley formula is varied. For drop and diesel hammered piles, varying the 100 coefficient of the FHWA modified Gates formula and the e coefficient of the MTO modified Hiley formula results in the least accurate predictions, respectively.

The percent differences between the predicted and field estimated pile capacities follow the same trend as that of the ratio of predicted to field estimated bearing capacities in the paragraph above. This implies that in general the FHWA modified Gates formula results in more accurate predicted bearing capacities compared to the other four formulae examined while the Hiley formula results in the least accurate predictions.

The correlation values indicate that the Gates and FHWA modified Gates formulae equally result in the most accurate predicted pile capacities for drop hammered piles. For diesel hammered piles, varying the c coefficient of the ENR formula and the e and C coefficients simultaneously of the MTO modified Hiley formula equally result in the most accurate predications when compared to all five dynamic formulae. Comparing all the MTO installed piles, the MTO modified Hiley formula results in the most accurate pile predictions when the e and C coefficients are varied concurrently. The smallest correlation values are from the Hiley formula, specifically when the e_f coefficient is varied.

Examination of the coefficients of variation indicates that varying the 1.75 and 100 coefficients simultaneously of the FHWA modified Gates formula results in predicted pile capacities containing the least amount of dispersion for drop hammered piles and drop and diesel hammered piles together. For diesel hammered piles the least amount of dispersion is equally obtained when predicting the pile capacities by altering the c coefficient of the ENR formula and the e and C coefficients simultaneously of the MTO modified Hiley formula. Varying the 100 coefficient of the FHWA modified Gates formula results in the highest coefficients of variation when predicting pile capacities for all installed piles; drop, diesel, and drop and diesel hammered piles together.

5.3.6 Comparison of All Dynamic Formulae by Analysis

To objectively determine which version of the multiple dynamic formulae used best predicts pile bearing capacity each one was compared to the other with the most accurate being summarized in Table 62 below. For example, of the five versions of the ENR formula used; original without any modifications, revised one where the c coefficient is taken as the average of the required c values to exactly match predicted to estimated pile capacities, multi regression analysis of the c coefficient, the dynamic formula omitting the built in safety factor, and multi regression analysis of the c coefficient omitting the built in safety factor, it is determined that the revised formula produces the most accurate predictions. The same analysis is conducted for the Gates, FHWA modified Gates, Hiley and MTO modified Hiley formulae. The parameters examined to aid in determining which formula is the most accurate are the trend line slopes and y intercepts, the coefficients of determination, the average ratios of predicted to field estimated pile capacities, the percent differences, the standard deviations, the coefficients of variation, and the correlation values between the predicted and field estimated pile capacities.

Ideally, the most accurate dynamic formula will result in a slope of unity, a y intercept of zero, an R^2 value of unity, an average ratio of predicted to estimated bearing capacity of one, as well as an average percent difference, a standard deviation and coefficient of variation of zero, and a correlation value of one, between the predicted and estimated bearing capacity values.

Table 62: Summary of the Most Accurate of Pile Capacity Analysis divided by Dynamic Formula

Dynamic Formula	Parameter	ENR	Gates	FHWA modified Gates		Hiley	MTO modified Hiley
Varied Coefficient(s)		revised c	revised $^1/_7$	revised		e/C3 multi regression analysis	e/C multi regression analysis
Original Coefficient Value	Drop	0.1	$^1/_7$	1.75	100	0.5 / variable	0.25, 0.32, 0.55 / variable
	Diesel	1	$^1/_7$	1.75	100	0.5 / variable	0.25, 0.32, 0.55 / variable
Revised Coefficient Value	Drop	0.05	2.29	1.80	119	1 / 0.58	1 / 21.66
	Diesel	0.21	3.53	1.17	276	1 / 1.55	1 / 55.66
Drop Hammered Piles	Slope	0.5966	0.8014	0.4613	0.4587	0.4035	0.4924
	y – intercept	526.69	195.64	390.25	442.55	404.25	335.39
	R^2	0.2176	0.2758	0.2846	0.2706	0.1746	0.2679
	Avg Ratio	0.486	0.951	0.894	0.708	1.193	1.152
	Avg % Diff.	-51.440	-4.929	-10.639	-29.162	19.319	15.179
	St. Dev.	0.303	0.408	1.330	1.452	0.420	2.163
	CV	0.623	0.429	1.488	2.05	0.352	1.878
	Correlation	0.466	0.525	0.533	0.520	0.418	0.518
Diesel Hammered Piles	Slope	1.3081	2.1148	1.1074	0.7431	1.1135	1.1729
	y – intercept	117.15	-599.59	-107.26	312.27	-373.36	-423.81
	R^2	0.5062	0.4660	0.4661	0.4662	0.3854	0.6187
	Avg Ratio	0.861	1.000	1.304	1.174	1.649	1.532
	Avg % Diff.	-13.881	-0.023	30.388	17.425	64.921	53.183
	St. Dev.	0.507	0.644	0.796	0.704	1.054	0.892
	CV	0.589	0.644	0.610	0.600	0.639	0.582
	Correlation	0.711	0.683	0.683	0.683	0.621	0.787
All Installed Piles	Slope	1.075	1.7129	0.9118	0.7145	0.8244	0.9258
	y – intercept	322.65	-333.02	86.365	316.55	33.533	-63.924
	R^2	0.4930	0.4292	0.4556	0.4867	0.3824	0.5672
	Avg Ratio	0.705	0.979	1.133	0.981	1.460	1.374
	Avg % Diff.	-29.490	-2.062	13.338	-1.936	45.969	37.389
	St. Dev.	0.470	0.555	1.063	1.095	0.876	1.551
	CV	0.667	0.567	0.938	1.116	0.600	1.129
	Correlation	0.702	0.655	0.675	0.698	0.618	0.753

From examining Table 62 and using the guidelines above, it is seen that for drop hammered piles the Gates formula with a revised coefficient of 2.29 results in values which most closely satisfy the conditions compared than the other formulae.

For diesel hammered piles it is seen that the ENR formula with a revised c value of 0.21 results in values which most closely satisfy the conditions compared than the other formulae. The MTO modified Hiley formula with revised e and C coefficient values of 1 and 55.66 result in values which satisfy conditions such as a slope value near unity, the largest R^2 and correlation values, and the smallest coefficient of variation, however, it also results in some of the largest ratio, percent differences and standard deviation values.

Considering all piles simultaneously, the revised Gates formula with a coefficient of 2.29 for drop hammered piles and 3.53 for diesel hammered piles results in values which best satisfy the condition with respect to the average ratio and percent difference of predicted to field estimated pile bearing capacities and low standard and coefficient of variation values. However, the revised Gates formula also results in the largest slope and offset y intercept value. Whereas, the revised MTO modified formula, with e and C values of 1 and 21.66 and 1 and 55.66 for drop and diesel hammered piles, respectively results in a slope value which is closest to unity, a relatively small y intercept, and the largest R2 and correlation values when compared to the other dynamic formula. However, the revised MTO modified formula also results the largest standard deviation and coefficient of variation values and a high average ratio of predicted to field estimated pile capacity when compared to the other four dynamic formulae.

For complete calculations of the mutli regression analyses, see Appendix F. The findings from the above analyses are further investigated in section 6: Discussion and Conclusions.

6.0 DISCUSSIONS AND CONCLUSIONS

From the analyses performed in sections 5.2 to 5.3, it is determined that the comparison of the dynamic formulae predictions to the measured static field tested pile bearing values of the MTO publications matched the results from similar studies conducted in the United States as well as Canada.

6.1 Original Dynamic Formula Analyses Summary

Examining Tables 24 to 27 with Table 29, it becomes evident that the original analysis performed on the MTO driven piles from dynamic formulae matches the trends found when comparing them to the results from the WSDOT database. The major difference is that the slope of the trend line describing the data is larger for the WSDOT piles, by factors of 1.48 to 2.23. The difference in slopes may seem large; however, this is most likely due to the difference in the y intercept of the best fit trend lines as well as the fact that the 2005 WSDOT study included piles driven by drop, diesel, sir/steam, and hydraulic hammers while the piles from the MTO database are driven by drop and diesel hammers only. When the best fit lines are normalized, by forcing to intercept to zero, the slopes only differ by factors of 1.32 to 1.91. Even though the slopes differ; the coefficients of determination and thus the correlation values of the data, from predicted to measured pile capacity only differ by approximately one and half to slightly greater than ten percent, with a mode value near six percent. The ratios of predicted to field estimated bearing capacities from pile load tests are all larger for the MTO database than those from the WSDOT study by factors of 1.12 to 1.92 times. The percent differences in predicted to measured bearing capacities between the two databases, in general, are similar and differ by a factor of 1.49, 1.36, and 11.85 for the ENR, Gates, and FHWA modified Gates formulae, respectively.

6.2 Revised Dynamic Formula Analyses Summary

Back calculating the predicted bearing capacities from the MTO database to exactly match the measured capacities from field tests by solving for specific variables from each dynamic formula and using the averaged values as the basis for re-analysis causes the trend line slopes for all piles examined (drop, diesel, and combined drop and diesel hammered piles together) to become closer to unity. However, the revised coefficients cause some formulae which previously under predicted pile capacities to now over predict them and vice versa as well. Even though the coefficients of determination increases for almost all formulae using the revised coefficients; it is only by -0.086 to

19.4 percent for diesel hammered piles and 3.9 to 115.9 percent for combined drop and diesel hammered piles. When considering drop hammered piles solely, the coefficients of determination changes from approximately -3.06 to 389 percent; even though this seems significant the actual coefficient of determination values range only from 0.07 to 0.28.

The revised ratios of predicted to estimated pile capacities range from 0.705 to 1.133 being smallest for bearing capacities being derived from the ENR formula and largest for bearing capacities being derived from the 1.75 revised coefficient of the FHWA modified Gates formula. This is a relatively large improvement over the values from the original dynamic analysis which on average under predicted and over predicted pile capacities by 56.8 to 99.0 percent, respectively.

The average percent difference from the revised formulae ranges from -29.5 to 13.3 percent from the ENR formula to the 1.75 revised coefficient of the FHWA modified Gates formula. Compared to the original analysis based on piles in the MTO database, the revised coefficients result in a smaller spread of values by approximately 110 percent. Similarly, the correlations improve by values of 0.013 to 0.258 from using the revised formulae when compared to the original dynamic analysis of piles from the MTO database.

6.3 Multi Regression Analysis Summary

The performed multi regression analysis involved adjusting two variables for each dynamic formula examined in order to minimize the amount of error between the predicted and measured pile capacities from piles given in the MTO database by using the solver function of Microsoft Excel. The variables are modified according to installation method as well as pile type for the analysis of the Hiley and MTO modified Hiley formulae.

Similar to the other analyses, the equation of the trend lines and coefficients of determination from the graphs of predicted versus measured pile capacities are compared using the results from the original dynamic formulae to those from the revised versions. The optimal coefficient values are determined in four stages; first, one variable is kept constant as its original value while the other is varied and vice versa. The second analysis involves solving them simultaneously to result in the minimum amount of error possible. An additional analysis is completed by varying the coefficients such that the corresponding trend line slope is as near to unity as possible. The final analysis is completed whereby a term is added to the end of the dynamic formula to ensure that the offset of the trend line equation is as small as possible. These analyses are conducted for all piles according to installation method and pile type; drop, diesel, and both combined together. For the Hiley dynamic formula the same methodology is used as for all piles within the MTO database as well as specifically on timber, precast concrete, H, and concrete filled steel tube piles. Based on the coefficients of restitution used by the MTO; the analysis is performed on predictions from the MTO modified Hiley formula and are done for all piles within the MTO database, then specifically for timber piles, piles driven with a steel cushion, and piles driven without a steel cushion, despite actual pile type.

During the ENR analysis it is observed that using Excel to solve for the c and modified energy coefficients does not produce a best fit line with a slope near unity except for diesel hammered piles. However, when all piles are examined together the values of c and modified energy which are determined as optimal are 0.4 and 6, respectively. This is interesting, since the ENR formula contains a built in safety factor of 6, thus performing calculations without the safety factor may result in more accurate pile predictions. The c and modified energy coefficient values which result in the best fit line having a slope near one for drop hammered piles are 0.4 and 2 and for diesel hammered piles are 0.3 and 1.85, respectively.

The Gates formula produces the most accurate predictions when an energy modifier of 0.78 to 0.96 is used and the $^{1}/_{7}$ coefficient at the beginning of the formula is varied from 0.54 to 0.55. However, when the energy term is left unchanged and the $^{1}/_{7}$ coefficient is changed to 0.35 for drop hammered piles and 0.60 for diesel hammered piles the slope of the best fit line approaches unity. It is found that the predicted values which cause a trend line to most closely match the 1:1 line occurs when examining all piles simultaneously and the $^{1}/_{7}$ coefficient is changed to 0.419.

For the analysis of the FHWA modified Gates formula the coefficients at the beginning and end of the equation are varied to produce the smallest amount of error between predicted and measured values as well as values which give a best fit line with slopes as near to unity as possible. Unlike the other dynamic formulae studied, the values of the coefficients which meet the two objectives are the exact same. To meet these criteria the 1.75 coefficient is varied between 0.8 and 1.3 and the 100 coefficient is varied between -66 to 86 for all piles examined together, drop hammered, and diesel hammered piles. The values of the coefficients of determination from the FHWA modified Gates formula are very similar to those of the original Gates formula, however when comparing the best fit line equations the offset from the Gates formula is smaller than that from the FHWA modified Gates formula.

During the analysis of the Hiley formula, it is determined that for timber and steel H piles no combination of soil compression (C_3) and coefficient of restitution (e) values result in best fit lines with an average slope of unity. For precast concrete, concrete filled steel tube, and all pile types examined simultaneously to obtain slopes of one, the coefficient of restitution in most cases is set to one, however for precast concrete piles it ranges between 0.5 and 1 depending upon installation method and for concrete filled steel tube piles a value of 0.8 is needed for a slope of unity but only when examining diesel hammered piles by themselves. The soil compression values ranged from 0.86 to 21.5 inches, in general the compression values are larger for drop hammered piles, decrease for diesel hammered piles, and vary when all piles are examined simultaneously, where some are in between the values of drop and diesel hammered piles while others are the lowest and highest values determined.

For pile prediction analysis based on the MTO modified Hiley formula, timber and steel piles installed without a cushion could not achieve a trend line with a slope of one. Only six piles within the MTO database are installed without a cushion and thus the results from statistical analysis based on such a small sample set may not be applicable. Examining all pile types simultaneously and steel piles installed with a cushion yielded predictions which could result in best fit lines with slopes of one once the required coefficients of restitutions and hammer efficiencies are chosen. In all cases, for all pile types and installation methods the optimal predictions occur when the coefficient of restitution is set to one. Hammer efficiencies which are originally taken as 0.75 for drop hammers and 1.00 for diesel hammers are found to give the optimal results when they are reduced to 0.55 to 0.64 for drop hammered piles and 0.28 to 0.48 for diesel hammered piles. When all piles are examined together, the optimal hammer efficiencies, those which result in the lowest amount of error between predicted and measured bearing capacities, range between 0.34 and 0.49. To achieve best fit lines with slopes of one, pile efficiencies are modified to range from 0.18 to 0.48 for diesel hammered piles, 0.10 to 0.22 for drop hammered piles, and 0.14 to 0.33 when examining all piles simultaneously.

Despite the fact that best fit lines with slopes of unity can be obtained for predictions based on the multi regression analysis of Hiley and MTO Hiley formula variables, the coefficients of determination and correlation values are significantly lower than those given from the multi regression analysis of ENR, Gates, and FHWA modified Gates formulae.

Of the three formulae which best predict pile capacity from the multi regression analysis, in general, the FHWA modified Gates formula produced the smallest amount of error. When considering drop hammer piles solely, on average, the ENR formula produces pile predictions which under predict pile capacity and gives the largest amount of error while the Gates formula produces slightly more accurate prediction however they are still underestimated, the FHWA modified Gates formula produces the most accurate results with the trend line lying over top the 1:1 slope and gives the least amount of error.

The coefficients determined from the multi regression analysis for diesel hammered piles resulted in predictions whose best fit lines overlay the 1:1 slope for capacities derived from the ENR and FHWA modified Gates formulae; the FHWA modified formula also gives the lowest amount of error. The Gates formula overpredicted pile capacities and resulted in the largest amount of error between predicted and measured pile capacities.

6.4 Dynamic Formulae Omitting Safety Factors Summary

Removing the safety factors from the dynamic formulae, in general, causes the predicted capacities to be greater overestimated, with an average of 4.9 to 6.4 and up to 24 times the actual estimated failure loads for the ENR and FHWA modified Gates formulae. This translates into an average difference of 390 to 537 and as high as 2346 percent above the estimated capacities from pile load tests. The only dynamic formula whose predictive accuracy increases is the Gates formula which when examining all installed piles results in a trend line with a slope of 0.98 and an y intercept of -12.7. The average ratios of predicted to estimated bearing capacities range from 0.93 to 1.51 for drop and diesel hammered piles, respectively.

The standard deviations from the three formulae are greatly increased from 2 to 6 times when compared to the same dynamic formulae used to predict pile capacities when the safety factors are present. In general, the correlation values between the two sets of formulae, with and without safety factors, are almost identical without any one method being obviously superior to the other. The coefficients of variation are also almost identical between the two types of dynamic formulae, with the exception for drop hammered piles which slightly decrease for drop hammered piles.

6.5 Revised Dynamic Formulae Omitting Safety Factors Summary

Removing the safety factors and modifying select parameters results in slopes very close to unity for predictions derived from the ENR, Gates, FHWA modified Gates (when the 1.75 coefficient is varied), the Hiley (when the e and C3 coefficients are varied simultaneously), and the MTO modified Hiley (when the e and C coefficients are varied simultaneously) formulae when compared to the field estimated bearing capacities, when considering drop and diesel hammered and diesel hammered piles solely. The resulting slopes range from 0.80 to 0.93 with y intercepts from -101 to 76 for all hammered piles and slopes of 0.98 to 1.35 with y intercepts from -600 to -162 for diesel hammered piles. None of the varied coefficients result in predictive trend line slopes near unity for drop hammered piles except for the FHWA modified Gates formula when varying the 1.75 and 100 coefficients simultaneously. However, for drop and diesel hammered piles and diesel hammered piles solely, the trend line produced have slopes of 0.63 to 0.68, respectively. Since the trend lines are comparatively low, only the five formulae above will be discussed below.

Comparing the revised predictions to those of the original formula it is noted that the coefficients of determination increase by 0 to 58 percent, with the largest changes being from the Hiley and MTO modified Hiley formula and the smallest from the Gates and FHWA modified Gates formulae. The absolute values of coefficients of determination range from 38 to 62 percent for all piles installed and those installed from diesel hammers only. For drop hammered piles, the coefficients of determination range from 17 to 28 percent.

The average ratios of predicted to field estimated capacities from the revised formulae without safety factors range from 1.32 to 1.47, 1.00 to 1.24, and 1.53 to 1.69 for all installed piles, drop hammered piles, and diesel hammered piles, respectively. This is an improvement from the original formulae which under and over predict pile capacities by an average of 0.17 to 3.06 times, respectively.

The subsequent correlation values improved by factors of 1 to 18 times compared to those of the original analysis. The largest improvements are from predicted values derived from the Hiley and MTO modified Hiley formulae, but the largest correlation values come from the FHWA modified Gates formula when the 1.75 coefficient is varied. In general, the coefficient of variations decreased by 0 to 46 percent when examining predicted capacities from all piles derived by all revised dynamic formulae; the only exceptions to this are for drop hammered piles while using the MTO modified Hiley formula and diesel hammered piles while using the ENR formula which increase by 280 and 16 percent, respectively. Despite the decreased coefficients of variation, the standard deviations for the ENR and Gates formulae derived predicted capacities increased for drop, diesel, and drop and diesel hammered piles by factors of 1.16 to 3.3. For the FHWA modified Gates, Hiley, and MTO modified Hiley formulae the standard deviations decreased by 5 to 67 percent; the only exceptions are for the drop hammered piles using the FHWA modified Gates and MTO modified Hiley formulae which increase by 4 and 337 percent, respectively.

6.6 Summary Of All Dynamic Formula Analyses

Based upon section 5.3.6, the Gates formula, with a revised coefficient of 2.29 rather than $^{1}/_{7}$, results in pile bearing capacity predictions which most accurately match field estimated values form pile load tests for drop hammered piles.

Examining diesel hammered piles demonstrates that the ENR formulae with a revised coefficient of 0.21 instead of 0.1 and the MTO modified Hiley formula using multi regression analysis to solve for e and C values of 1 and 55.66, respectively, results in the most accurate bearing capacity predictions when compared to all other dynamic formulae by all other types of analyses. The average ratio of predicted to estimated pile capacities are slightly under predicted (0.86) when calculating values with the ENR formula and are over predicted by a factor of 1.5 when the MTO modified Hiley formula is used. The average percent difference between predicted and estimated bearing capacities along with the standard deviations are less for the ENR formula than the MTO modified Hiley formula as well; -14 and 0.6 compared to 53 and 0.9. However, the MTO modified Hiley Formula results in predictions with a slope closer to unity (1.17 compared to 1.31), larger coefficient of determination values (0.62 compared to 0,51) and larger correlation values (0.79 compared to 0.71) than the ENR formula.

When both drop and diesel hammered are considered together, the ENR and MTO modified Hiley formulae with revised coefficients from multi regression analysis result again in the most accurate pile bearing capacity values with the same trends between them observed when examining diesel hammered piles only.

7.0 RECOMMENDATIONS AND FUTURE RESEARCH

The following recommendations are given based upon the findings discovered during this study:

Based upon the findings in this study, it is determined that without performing any adjustments to the dynamic formulae the FHWA modified Gates formula is the most accurate of the five studied. It results in best fit with slopes closest to unity and minimal y-intercept values. It also results in the highest correlation values and data points which are the most centered about the 1:1 line and the smallest percent difference between predicted and measured pile capacities for drop hammered piles as well as the ratio of predicted to measured capacity closest to unity for drop hammered piles.

The Engineering News Formula gives, on average, gives the smallest percent differences and ratios closest to unity for predicted to measured pile capacities as well as data points which center the best, compared to the other dynamic formulae, about the 1:1 line for diesel hammered piles.

Using the revised coefficients to predict pile capacity results in the FHWA modified Gates formula producing data points which centers the best about the 1:1 line when varying both the 1.75 and then the 100 coefficients at the beginning and end of the formula, respectively. It is determined that changing the 1.75 coefficient resulted in a smaller spread of data and best fit line slopes closer to unity with offsets closer to zero than when altering the 100 coefficient.

During the multi regression analysis, the ENR and FHWA modified Gates formulae are found to give the most accurate pile predictions. Depending upon the criteria selected, such as coefficients of determination, the ENR formula seems better suited to determine pile predictions while the FHWA is more accurate when looked at the amount of error between predicted and measured pile values.

It was determined that the removing the safety factors of the ENR, Gates and FHWA modified Gates formula did not increase their predictive capacity, except in the case of the Gates formula for drop hammered and combined drop and diesel hammered pile analysis. However the coefficient of variation and standard deviation values were still relatively high.

Removing the safety factors and using mulit regression analysis to solve for select coefficients of the ENR, Gates and FHWA modified Gates improved their predictive capacities more so than simply using multi regression analysis alone. However, using multi regression analysis on the e and C3 and e and C coefficients of the Hiley and MTO modified Hiley showed the most amount of improvement.

Thus, it is not advised to use any of the dynamic formulae in their original form but rather with the new coefficients from the revised or multi regression analyses. This is due to the increased accuracy that the new values provide. Because of the ease of calculations it is recommended that when predicting the bearing capacity of drop hammered piles, that the Gates formula with a revised coefficient of 2.29 is used. When predicting the bearing capacities of diesel hammered piles that both the revised ENR formula with a c coefficient of 0.21 and MTO modified Hiley formula and e and C coefficients of 1and 55.66 are used and the more conservative estimate is taken.

If the Hiley or MTO modified Hiley formulae are used, then field investigations and observations should record the unit weight of the soil being driven into, the unit weight of concrete or timber being used if applicable, the material that the pile cap is composed of and the condition of the pile cap in addition to the usual data collected in order to minimize the amount of uncertainty in parameters such as the coefficient of restitution and ultimately the predicted capacity. Once the uncertainty is minimized, the formulae can be re-examined to determine if the predicted capacity can be improved.

Whenever future piles are driven, the total amount of compression that the soil – pile system undergoes during hammer strikes should be recorded so that it can be inputted into the MTO modified Hiley and Hiley formulae to aid in determining the true extent of how accurate the formula is and if changing parameters, such as the coefficient of restitution, improves bearing capacity predictions. This is especially true for high capacity piles since it has been previously shown that changing the C3 or C coefficients values for all drop and diesel hammered piles has drastic effects on the predicted values of large bearing capacities and causes less variation when bearing capacities are relatively small.

For each borehole record compiled the static water level, unit weights, effective cohesions, shear strengths, unconfined shear strengths, unconfined compressive strengths, effective angles of internal friction, preconsolidation pressures, over consolidation ratios and soil densities of each stratigraphic unit the pile may be driven into should be determined.

If computer programs are used, then soil parameters such as those above should be obtained in addition to specific parameters used by the analytical code such as soil stiffness, damping factors, and shear and elastic moduli.

Because of the high correlation values between predicted and measured capacities from wave equation analysis from both the WSDOT study and the MTO publication, the relatively low expense of instrumenting piles, and the convenience and computing powers of laptops as many piles as practicably possible should be instrumented with strain gauges and accelerometers to record force and velocity measurements at the moment of striking.

Based on the literature review and review of the data available during this study the following avenues for future research are suggested:

Pile predictions should be performed based on the theoretical static equations, assuming the necessary parameters are available, to determine how accurate bearing capacities derived from them are when compared to measured values determined from pile load tests.

Research should also be conducted in determining and improving pile predictions from wave equation analysis and if found superior to dynamic formulae to eventually replace them as indicators of pile bearing capacity.

The next phase of research should be inputting borehole and pile driving data into computer programs to allow them to predict pile capacities. Since these programs are based on soil properties and possibly preliminary pile driving observations there may be some time required between the reported results and further pile driving activities. This may aid in allowing more accurate bearing capacity predictions but most likely program parameters need to be calibrated to site specific data such as pile load test results which may then be used to aid future pile driving operations on the same site or into similar soils. Additionally, the wait time between the first piles driven to gather data for computations and the remaining piles after the results are analyzed may be too great to be practical.

Much research has been conducted upon analysis of single piles, however in practice most piles are utilized in groups of three or more. Because of the high bearing capacity once in groups more often than not load testing them to failure is not practically possible. However if in future jobs it is found to be feasible, then load tests should be conducted on pile groups to determine if the actual settlements are similar to the predicted values and if group efficiencies given by geometric formulae are accurate or not. If not then they can then be altered to produce more accurate predictions for future sites.

Although LRFD is relatively new to the field of geotechnical engineering it has already been studied by AASHTO, FHWA, and academia in the United States and appears to be intricately involved in the future of pile design. As such, the MTO should perform a study based on piles driven in Canada to quantify the uncertainty in pile behaviour and improve pile predictions by using probabilities from previously installed piles.

Alternatives to deep foundations such as mid foundations, soil improvement techniques, and micro piles should also be examined to determine if they can provide feasible substitutions for conventional piles. Ideally, this results in creating the required bearing capacity and save both time and money on construction projects while also providing reliable designs which lasts the intended project lifetime.

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APPENDIX A

STATIC THEORETICAL FORMULAE

To view the contents of Appendix A please see the attached CD in the hard copy of this thesis found at the Graduate Studies Office in the Civil Engineering department or contact Prof. Leo Rothenburg

APPENDIX B

MTO EXCEL SHEETS

ORIGINAL DATA

To view the contents of Appendix B please see the attached CD in the hard copy of this thesis found at the Graduate Studies Office in the Civil Engineering department or contact Prof. Leo Rothenburg

APPENDIX C

MTO EXCEL SHEETS

DYNAMIC FORMULAE PREDICTIONS

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APPENDIX D

EXCEL DATA SHEETS

COMPARISON TO WASHINGTON STATE STUDY

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APPENDIX E

MTO EXCEL SHEETS

REVISED COEFFICIENT PREDICTIONS

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APPENDIX F

EXCEL DATA SHEETS

MULTI REGRESSION ANALYSIS

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