ISTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

VERIFICATION OF PILE DRIVING PREDICTION USING PDPWAVE

M Sc. Thesis by
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Department : **Civil Engineering**

Programme: **Soil Mechanics and Geotechnical Engineering**

JUNE, 2006

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PDPWAVE YAZILIMI İLE KAZIK ÇAKMA ÖNGÖRÜLERİNİN DOĞRULANMASI

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TABLE OF CONTENTS

ABBREVIATIONS

BS: The British Standard Code of Practice for Foundations
CFA: Continuous Flight Auger
EOM: Equation of Motion
SPT: Standard Penetration Test
CPT: Cone Penetration Test
PMT: Pressuremeter Test
DMT: Dilatometer Test

TABLE LIST

Page No

FIGURE LIST

Page No

SYMBOL LIST

A_b : Cross-sectional area of the pile toe (base)

A_s : Surface area of the pile shaft

A_{si} : Nominal surface area of the pile in soil layer i

C_1 : Temporary elastic compression in the cap

C_2 : Temporary elastic compression in the pile

C_3 : Temporary elastic compression in the soil

c_{avg}: Average undrained shear strength over the depth of the pile shaft

c_u : the undrained shear strength

c_{ub} : Undrained shear strength at the base of the pile

e : Coefficient of restitution (elasticity)

e_f : Mechanical efficiency

E : Rated hammer energy per blow

f_s : Average unit shaft friction on the pile shaft

F : Factor of safety

$F\alpha_p$: Adhesion factor

h : Distance that hammer falls

K : Coefficient of lateral earth pressure

N_c : Bearing capacity factor

N_q : Bearing capacity factor

P_a : Net allowable vertical load capacity

Q_s : Ultimate shaft resistance in skin friction

Q_b : Ultimate resistance of base

$\mathbf{q_b}$ **:** Unit bearing capacity of the pile toe

$\mathbf{q_b}$ **:** Net unit end bearing resistance

$\mathbf{q_s}$ **:** Design ultimate unit shaft friction

$\mathbf{q_{bk}}$ **:** Characteristic value per unit area of base

$\mathbf{q_{sic}}$ **:** Characteristic value of the resistance per unit of the shaft in layer i

s **:** Penetration per blow at the end of driving

$\mathbf{W_r}$ **:** Weight of ram

$\mathbf{W_p}$ **:** Weight of pile

$\boldsymbol{\sigma'_{vo}}$ **:** Average effective overburden pressure

$\boldsymbol{\phi}$ **:** Friction angle

$\boldsymbol{\gamma_s}$ **:** Partial safety factor for shaft friction

$\boldsymbol{\gamma_b}$ **:** Partial safety factor for base resistance

$\boldsymbol{\delta}$ **:** Angle of friction at the pile/soil interface

VERIFICATION OF PILE DRIVING PREDICTION USING PDPWAVE

SUMMARY

Pile foundation design mainly depends on the determination of bearing capacity. In this manner determination of pile capacity by static and dynamic formulas are the most frequently used and the known oldest methods. Although those mentioned calculation methods have wide application areas, they have weaknesses in practical application results due to a great number of different dynamic formulas having several different assumptions and different degrees of accuracy. Also their lack of providing any information about the pile drivability bring out the necessity of usage of more comprehensive solution provided for impact driven piles by the wave equation. Wave equation theoretically based on the one dimensional stress wave theory. In geotechnical engineering one of the application areas of the wave equation analysis is the drivability studies in order to be able to decide hammer selection to prevent possible damages during pile driving. Wave equation analysis considers the hammer cushion-pile-soil system to be able to determine the appropriate hammer size for the given pile dimensions and soil resistance conditions. In the scope of this study, by using PDPWAVE software, pile-soil system is modeled before the installation of a driven pile in the field. Driving resistance against penetration and blow count predictions are performed during driving process depending on two different hammer types and depending on the modeled soil profile condition. The goal of the study is to verify the obtained computer based results with the actual field data in order to be able to test the efficiencies of the hammer types as well as with to determine the efficient driving parameters not to cause any damage to pile during driving. At the end of the study it is observed that the predictions and actual data match with some appropriate calibration. As a result it is concluded that the site investigation in-situ tests defining the actual soil condition must be reliable in order to get an accurate analysis result.

PDPWAVE YAZILIMI İLE KAZIK ÇAKMA ÖNGÖRÜLERİNİN DOĞRULANMASI

ÖZET

Kazıklı temel sistemlerinin istenilen servis gereksinimlerini karşılayabilmeleri bakımından; tasarım aşamasında, oturma analizlerinin yanısıra taşıma gücü kapasitesinin belirlenmesine dayalı hesaplar da büyük önem taşımaktadır. Taşıma gücü hesapları yaygın olarak statik kazık formülleri ile; dinamik analizin gerekli olduğu durumlarda ise çakma kazıklar için dinamik kazık formülleri ile yapılmaktadır. Dinamik formüllerin kendi içlerinde bir çok farklı yaklaşım sergilemesi ve hesap yöntemlerinin kazıkların çakılabilirliği ile ilgili bir bilgi sağlayamıyor olması daha kapsamlı bir hesap yönteminin gerekliliğini ortaya koymuştur. Bu amaçla geliştirilmiş olan dalga denklemi analizi kazıkların çakım sırasındaki dinamik davranışlarını daha doğru bir şekilde tanımlayabilmektedir. Profound tarafından geliştirilmiş olan PDPWAVE bilgisayar yazılımı da bu esasa dayalı olarak çakma analizi gerçekleştirmektedir. Bu çalışma kapsamında PDPWAVE yazılımı kullanılarak arazide çakılmış olan bir kazık için kazık-zemin modellemesi ve çakım simülasyonu gerçekleştirilmiştir. Yazılım kullanılarak, çalışılan zemin koşulları ve iki ayrı çekiç tipi için, çakım esnasındaki çakma direnci ve toplam vuruş sayısı tahmin analizleri gerçekleştirilmiştir. Elde edilen analizler, gerçek çakma verileri ile karşılaştırılarak yapılan tahminin doğruluğu karşılaştırmalı olarak incelenmiştir. Bu sayede en uygun çekiç ve zemin parametleri değerleri belirlenmiştir. Çalışma sonucunda simülasyon sisteminin kalibrasyonunun ardından analiz sonuçları ile gerçek arazi verilerinin uyum içerisinde olduğu görülmüştür. Yapılan değerlendirmeler sonucunda, etkili ve doğru bir analiz sonucunun elde edilmesi için, öncelikle deneyimin ikinci olarak ise, bölgenin zemin koşullarını tanımlayan arazi değerleri sonuçlarının çok güvenilir olması gerekmektedir.

1. INTRODUCTION

Pile foundation design mainly depends on the calculation of bearing capacity. In this manner determination of pile capacity in design stage is commonly done by using static formulas. In special occasions for driven piles dynamic formulas are also used for capacity evaluation. At this point, accurate and reliable calculation is very important for foundation systems to be able to meet the service requirements under loading conditions. Although those mentioned calculation methods have wide application areas, especially the dynamic based calculations have weaknesses in practical application results due to a great number of different formulas having several different assumptions with different degrees of accuracy indicated by high factor of safety. Also their lack of ability in providing any information about the pile drivability brings out the necessity of usage of more comprehensive solutions. In this aspect, one of the most important developments applied for driven piles is the wave equation analysis defining the dynamic behavior of pile driving more accurately. Wave equation analysis is theoretically based on the principles of one dimensional wave propagation theory in a rigid body. In geotechnical applications, the developed test methods and computer programs based on this one dimensional stress wave theory are applied in order to predict pile drivability and to control the integrity of constructed piles. Wave equation analysis based programs model the hammer-pile-soil system to be able to determine the appropriate hammer type for a given pile and soil resistance conditions to prevent possible damages which may occur during driving process. Practical use of stress wave theory on piles in order to analyze pile driving was pioneered by A. L. Smith in the late 1950s. He was the first who developed a mathematical model of the hammer-pile-soil system by formulating a numerical solution using discrete elements methods. Smith's work was followed by significant developments especially in computer programming and testing equipments. Nowadays one of the most commonly used wave equation program is TNOWAVE which is developed by Profound. The software is composed of five different modules having different wave equation applications. One module of this program is called as PDPWAVE which is a kind of pile-soil interaction simulation program providing information about optimized hammer selection, drivability and the maximum energy level for a pile. In the scope of this research, pile-soil system will be modeled by using PDPWAVE software before the installation of a driven pile in the field. Stress level, the blow count and the duration of the driving process predictions will be done depending on the hammer selection for the modeled soil profile condition. The goal of the study is to choose efficient hammer as well as with the efficient driving parameters not to cause any damage to pile during driving. The results will be compared with the actual driving parameters and the accuracy of the prediction will be discussed.

2. PILE FOUNDATIONS

The most important basic unit of an engineering structure is considered as foundation. Every building in existence should rest on a foundation whether formally designed or not (Bowles, 1996).Foundation systems act as an interfacing element between the superstructure and the underlying soil or rock which are having two different materiel characteristics. The system transmits the construction loads to the substratum.

Superstructure loads are brought to the foundation system by concrete or steel columns. They have high value of allowable design compressive stresses range between 140Mpa and 10Mpa. On the contrary, load bearing soil can only support stresses between 200 and 250 kPa which is relatively low. Bearing capacity difference of two materials can only be compensated by spreading the load to the soil without exceeding the tolerable limits of strength and deformations. Spreading depth of load within the ground roughly defines the foundation type. Depending on that definition, foundation types are generally divided into two groups such as shallow and deep foundations. Shallow foundations distribute the load laterally while deep foundations spread it vertically. Bases, spread footings and mats are considered as shallow foundations whereas piles and drilled shafts are considered as deep foundation systems.

Deep foundations are composed of piles which are timber, concrete or steel columnar structural elements. Pile foundations are made up of those structures. The general function of a pile foundation system is to transfer the load which is coming from the superstructure through weak strata onto stiffer or more compact and less compressible soils or onto rock.

Pile foundations are also commonly used in order to resist uplift forces caused by basement mats below the water table, offshore structures and to resist overturning forces caused by winds, waves or earthquake effects especially in the construction of retaining walls and foundations of tall building or tower structures. Piles can resist the lateral loadings by bending. While constructing pile foundation systems of retaining walls, machinery foundations, bridge piers and abutments, they are exposed to the combined effect of both vertical and laterals forces.

Finally pile foundations are chosen in the case of existence of expansive or collapsible soil stratum. Soil profiles having expansive and collapsible characteristics can lead to undesired settlement problems within the structures resting on shallow foundations. Pile foundations remove the negative effects of the defined soil profile on the structure.

Other than pile foundation usage purposes one of the most important points is to determine the load-bearing capacity of a designed pile in order to construct a proper foundation system providing the service requirements. As a primitive rule in the earliest days of piling, allowable load on a pile is determined from its resistance to driving by a hammer of known weight and with a known height of drop. With the developments in construction techniques, the actual pile capacity and design controls are determined by using the dynamic formulas and pile load tests. Theoretical bearing capacity calculations of shallow and deep foundations are based on the principles of soil and rock mechanics but the fundamental theory leads to different reliable results in spread foundations and deep pile foundations. In spread foundations the theory works depending on the physical characteristic of undisturbed soil due to the reason that shallow foundation construction techniques affect shallow depths.

2.1 Classification of Piles

Piles can be classified or analyzed based on five main different categories (Prakash, Sharma, 1990):

- *Pile material*

Piles can be classified as timber, concrete, steel and composite piles depending on their principal own material. Composite piles can probably be the combination of timber and concrete or steel and concrete. Among these ones, the classification method based on pile material has more common and precise properties in the type description of pile which also involves other significant pile characteristics such as pile installation, load transfer and ground disturbance amount in itself providing more detailed and further information. Following the definition, pile types can be classified in five major categories:

i. Timber piles
ii. Concrete piles
iii. Steel piles
iv. Composite piles
v. Special types of piles

- Method of pile manufacturing

The method of pile fabrication is another classification criteria which is briefly defines the prefabrication stage of piles as it is valid for timber and steel piles. Concrete piles can either be precast or cast-in-place.

- Amount of ground disturbance during pile installation

The third step for classification is the amount of ground disturbance during pile installation which is a distinguishing criterion. This classification can be divided into four different sections within itself.

i. Large displacement piles
ii. Small displacement piles
iii. Replacement piles
iv. Composite piles

In large displacement piles while installing piles by driving, jacking, or vibration, the resting soil is disturbed by displacement. The driven types of large displacement piles are as timber, precast concrete, prestressed concrete, steel tube driven with closed end and fluted and tapered steel tube piles. The driven and cast-in-place type of large displacement piles are as steel tube driven and withdrawn after placing concrete, precast concrete shell filled with concrete.

The small displacement piles are also displace the soil at rest but relatively in small amount due to the piles having relatively small cross-sectional areas. Examples to this type of piles are given as steel H- or I- sections, precast and prestressed concrete tubular section driven piles with open end. The terms ‘large’ or ‘small displacement’ used are for qualitative description only, since no quantitative values of displacement have been assigned (Prakash, Sharma, 1990).

Replacement piles do not displace the soil due to the fact that initially a hole is drilled by rotary auger, percussion boring (grabbing) or reverse-circulation methods and then one of the following techniques are applied as placing concrete, tubes filled with concrete, precast concrete sections, or placing steel sections and steel tube into the drilled hole of equal volume of pile.

Those three categories of pile types depending on the amount of ground disturbance mentioned above can be combined in such a way that composite pile is formed. Displacement and replacement piles combination and H-section piles jointed to the lower end of a precast concrete pile form an example to composite pile type.

- Method of pile installation into ground

Piles are installed into ground by three basic methods. These methods are driving, boring (drilling) and the combination of those of both driving and boring. Bored piles are most commonly the cast-in-place concrete piles, on the other hand timber, steel and precast concrete piles are examples to the driven piles.

- Method of load transfer

Piles transfer the structure loadings to the surrounding soil by tip resistance, sleeve friction or by the combination of both bearing forces and finally by lateral loading. Piles transferring load by tip resistance are named as end-bearing piles. In the same manner, piles transferring load by sleeve friction are called friction piles. End-bearing piles are passing through loose stratum reaching to the stiffer soil profile as if gravel, dense sand or rock formation. Friction piles distribute the structure loading along its sleeve to soil profile in which it is driven through. Combined end-bearing and friction piles support the load partly through sleeve friction to the soil around them and the remaining load is transferred to the underlying denser or stiffer stratum (Prakash, Sharma, 1990).

2.2 Pile Types

Those mentioned classification categories above shows that piles can be analyzed in various titles as pile material, pile fabrication, amount of ground disturbance during installation, installation techniques and finally the load transfer. The British Standard Code of Practice for Foundations the "BS 8004" classifies piles depending on their amount of ground disturbance related with the pile installation methods as follow;

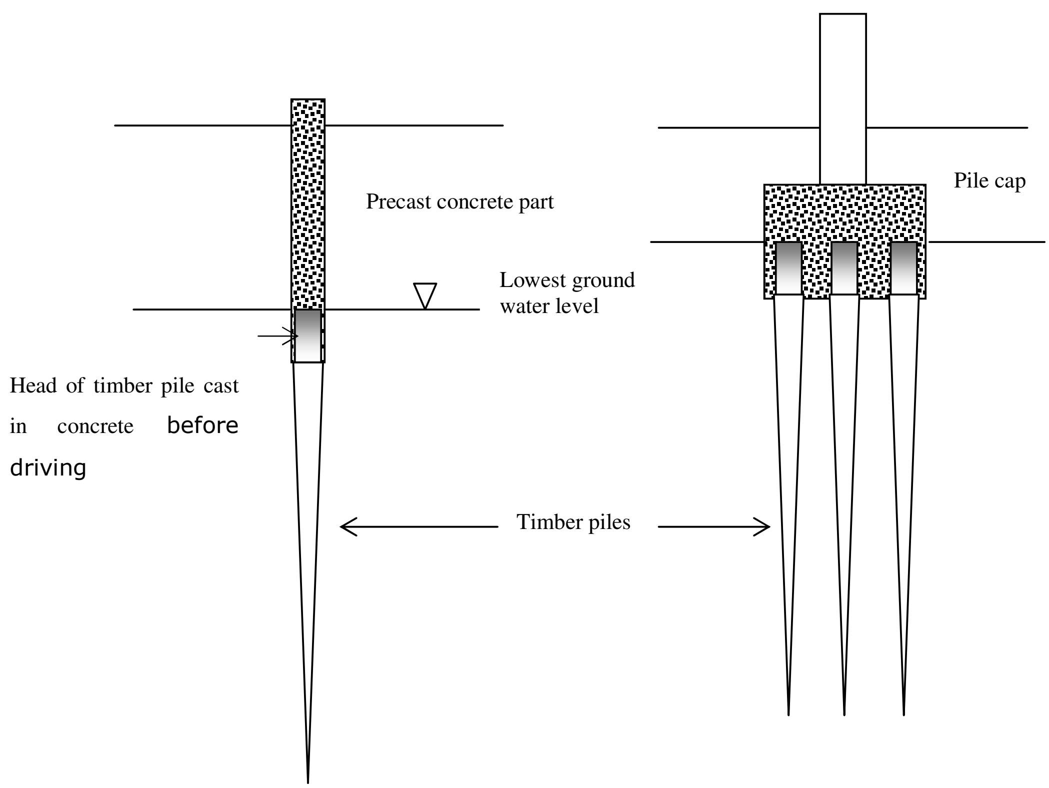
- Driven piles (Displacement type)
- Driven and cast-in-place piles (Displacement type)
- Bored and cast-in-place piles (Replacement type)
- Composite piles

2.2.1 Driven Piles

Driven type of piles consist of solid-section piles or hollow-section piles with a closed end, which are driven or jacked into the ground and thus displace the soil.

2.2.1.1 Timber Piles

Timber piles are advantageous for piling due to their easy handling, readily cut to desired lengths, high durability and they have almost a long lasting life time period in suitable environmental conditions. Timber piles require treatment related to their installation environments. In fresh groundwater level they do not need any treatment but in the case of such as extending above the groundwater level or in marine conditions it is required to be treated by creosote in order to be able to prevent the decay of timber piles. On the other hand, some other prevention methods must be thought although preservative chemicals can extend the life of timber. One of the most common techniques is to cut off timber piles just below the lowest predicted ground-water level and to extend them above this level in concrete(Fig2.1a). If the ground-water level is shallow the pile cap can be taken down below the water level (Fig2.1b).



(a) (b)

Figure 2.1 : Prevention of timber piles – (a) precast concrete section above water level; (b) extended pile cap below water table level (Tomlinson, 1994)

Timber piles can be in two types as round untrimmed logs or as sawed square sections. Square cross-sectional timber piles are prepared by removing the outer sapwood which is absorptive to creosote or some other liquid preservative. Thus, this process has negative effects on piles durability. Timber piles are commonly used as friction piles in all type of soils but they are significantly suitable for coarse grained soils.

In order to prevent possible damages of the pile point against splitting or brooming under hard driving conditions, shoe is required for the pile tip especially for end-bearing timber piles. Cast steel point for pile toe is the common protecting shoe for timber piles. Shoe especially must be used when piles are driven into dense or hard materials.

2.2.1.2 Precast Concrete Piles

Precast concrete piles are cast, cured and stored in a yard before they are installed in the field. They are driven as small or large displacement type of piles. They have circular, square, octagonal or hexagonal cross sectional areas with short or moderate lengths. In order to save weight long piles are generally designed with a hollow in the center. After the driving process is completed, the interior hole is filled with concrete. This process is applied in order to prevent bursting against frost action.

Reinforcement is an important aspect for precast concrete piles. This characteristic can have either positive or negative effects on land structures related to the economical issues. Precast piles must be reinforced in order to be able to withstand the bending and tensile stresses during handling and driving but if piles are only exposed to compressive loading, then this reinforcement becomes useless after the installation of pile into the ground. In that case reinforcement turns into be an economical factor which raises the cost. This necessity of reinforcement causes them to be more costly than other type of concrete piles such as driven-and-cast-in-place type. On the other hand if lateral loading and uplift loads are exists causing bending and tensile stresses respectively reinforcement has an important function to resist. One of the other shortcomings of unjointed precast concrete piles is that they are not readily cut down in order to be able to reach the bearing stratum. They are used both as end-bearing piles in soft subgrades and as friction piles in sand, gravel or clay.

Precast concrete piles are subdivided into two as reinforced precast concrete piles and prestressed concrete piles. Reinforced precast concrete piles are reinforced by longitudinal bars with also including the lateral reinforcement as individual hoops or spiral. They are generally designed for about 12-15m in length. Cracking causing pile deterioration under environmental conditions occurs up to 0.25mm in width. These disadvantages can be prevented by the usage of prestressed concrete piles instead of reinforced precast concrete piles.

Prestressed concrete piles are designed by using steel rods or wires under tension to replace the longitudinal steel used in reinforced concrete piles (Prakash, Sharma, 1990). They meet the high-capacity long pile requirement for different soil conditions. Besides they have some other advantages compared to an ordinary reinforced concrete as if being lighter and longer. Also they are more durable due to the fact that the concrete is under continuous compression which blocks the penetration of harmful chemicals through the concrete mass. Prestressed piles can be also divided into two sections as pretensioned and posttensioned prestressed concrete piles. Posttensioned piles are constructed in sections to be assembled at the site and pretensioned prestressed ones are manufactured in full length at about 40m long.

2.2.1.3 Steel Piles

Steel piles are commonly used as sheet piles but their advantages in handling, driving and resistance over other type of piles make them preferable as load bearing piles. Steel piles are light and resistant to buckling and bending forces due to flexibility of material, they can easily be handled without damage. They can be readily cut down. Also their lengths can easily be adjusted by welding or coupling depending on different depths of bearing soil formation or rock They have ability of carrying high compressive loads when they are driven into firm bearing stratum and they have resistance to hard driving without breaking.

Since steel piles are resistant to lateral and buckling forces, they are widely used in marine structures. Especially steel tubular piles are preferred. Thus circular shape is also advantageous in minimizing drag and oscillation from waves and currents but the portion of the pile above the sea bed in marine structures or in disturbed soils steel piles require protection against corrosion by cathodic methods. In the same manner, in marine conditions welded joints of pile above the sea-bed level must be in high quality against possible high lateral forces and corrosion effects. In land structures, welding is not a critical factor affecting the load carrying capacity of pile where it is supported by the soil.

Three main types of steel piles are H-section piles, box section piles, plain tube or monotube (tapered and fluted tube) piles. Hollow- section piles can be driven with open ends. They need not have to be filled with concrete. Where steel tube or box piles are filled with concrete the load is shared between the concrete and the steel. The working stress in the concrete should not exceed the value normally used for precast concrete piles (Tomlinson, 2001).

Steel H-section piles are mainly characterized as small displacement piles (Fig2.2a). They do not cause ground heave or lateral displacement. These properties make them useful in deep penetrations through loose or medium-dense sands and in situations where ground heaving is undesirable. Due to their small cross sectional areas, H piles do not act as a high resistant end bearing pile in soils or in weak or broken rocks. Special types of H-section steel piles are constructed. Short H-section piles are welded on to the flanges of the piles close to their toes to form 'winged piles' in order to increase their cross-sectional areas in end-bearing without reducing their penetrating ability. The bearing capacity of tubular piles can be increased by welding T-sections on their outer periphery when the increased capacity is provided by a combination of skin friction and end-bearing on the T-sections (Tomlinson, 1994).

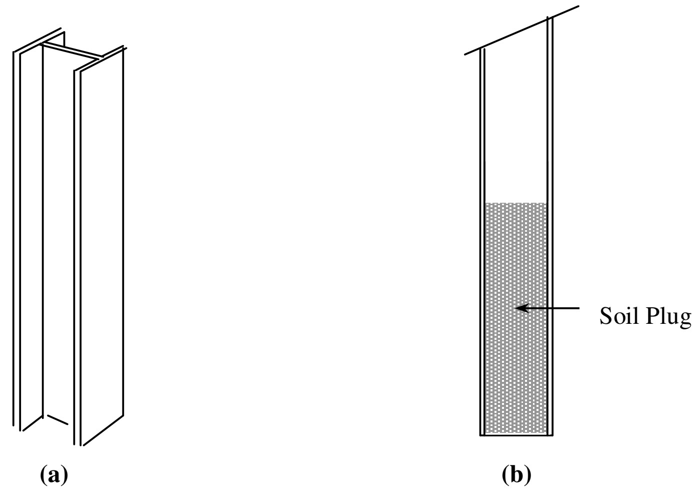


Figure 2.2 : Cross section view of steel piles (a) H-section pile, (b) open ended steel tube piles

On the other hand as a limiting property, H-sections can show bending along their weakest axis. This bending can result in deflection from the vertical with serious curvature during deep penetration drivings. Results of curvature measurements show that working stresses of steel is exceeded before the application of super structure loads. Failure in the section of maximum curvature is due to the plastic deformation of the pile shaft. Concrete as a material does not stress during driving before the application of load. In this aspect, the steel tube piles with their interiors filled with concrete are superior when compared to H-section piles against curvature and working stresses.

Steel tube piles are widely used in the USA, sometimes in the form of ordinary pipe-sections filled with concrete, and also in the form of specially designed fluted sections which are driven to the full depth by ordinary pile hammer and then filled with concrete. (Tomlinson, 2001). Steel tube piles as also called as pipe piles may be driven either with a closed-end or with an open end. A closed-end pipe has a flat steel plate or a conical steel point welded to the bottom. An open-end pipe has nothing at the bottom thus the soil enters the pile interior during driving forming a soil plug (Fig2.2b). Thus, an open-end pipe pile displaces less soil than closed-end pile, but more than an H-pile (Coduto, 1994).

Monotubes tapered and fluted types are hollow steel tubes tapering uniformly. Piles have two sections as uniform and tapered sections. They have standard tip diameters of 203mm and the shaft diameter varies between the values of 305mm, 356mm, 406mm or457mm depending on the pile length.

2.2.2 Driven-and-Cast-in-Place Piles (Displacement Type)

Driven-cast-in-place displacement types of piles are composed of three main components as steel tube, reinforcement element and concrete. Close ended steel tube is driven into the ground to the desired depth with the reinforcement unit is placed after driving and inside of the tube is filled with concrete. As casing element withdrawable steel tubes, thin steel shells or precast concrete shells can be used. The withdrawable tube piles classified as uncased cast-in-place driven piles are the most economical type of pile for land structures (Tomlinson, 1994) but pile rig can be a constraint factor in pile length due to the penetration limits of pulling out the driven tube. So piles are at about 20 or 30m in length for withdrawable tube cast-in place piles.

2.2.2.1 Withdrawable Tube Types

Withdrawable tube is driven into the ground by a drop hammer or by a diesel or vibrating hammer. At the end of the driving of the tube to the desired penetration depth reinforcing cage is lowered down the full length of the tube. Full length of reinforcement prevents possible discontinuities in the pile shaft that might have been occurred during the removal of the tube due to arching and lifting of the concrete. Concrete is then placed into the tube and finally the tube is pulled out by a hoist rope. The length of formed pile is limited by the ability of the rig which withdraws the drive tube. As a result pile lengths reach up to 20-30m. This installation technique of the tube is known as top driving method. Other possibility is to drive it by internal hammer technique. As well as the installation methods, removing the driven tube back after placing the concrete is another important step construction of the cast-in-place piles. The withdrawable tube can be pulled out in stages during placing concrete or after completing the process when it is totally filled with concrete. Permanent light gauge steel lining tube can be used along with the drive tube in order to prevent possible problems that may arise due to the pressure of injected concrete in the surrounding soil (such as soft clays and peat) and also in the pile shafts.

Frankie pile (Fig2.3) which is a special type of cast-in-place pile also called as expanded base compacted pile is an example to both internal hammer installation technique and pulling out of the driven tube in stages. Frankie pile consists of a drive tube having plug material at the bottom tied down by lifting ropes to the pile rig above the ground surface, the internal hammer is dropped on this plug in order to install the tube. Tube is driven until the required toe level controlled by the lifting ropes. Then at the toe level the gravel and dry concrete are compacted in order to obtain a bulb or enlarged base to the pile. Reinforcing cage is set into the tube, then as the drive tube is pulled out in stages concrete is placed. Internal hammer method is relatively a slower process when compared to the top driving method, but in the case of the necessity of high bearing capacity, the enlarged base maintains economical benefits.

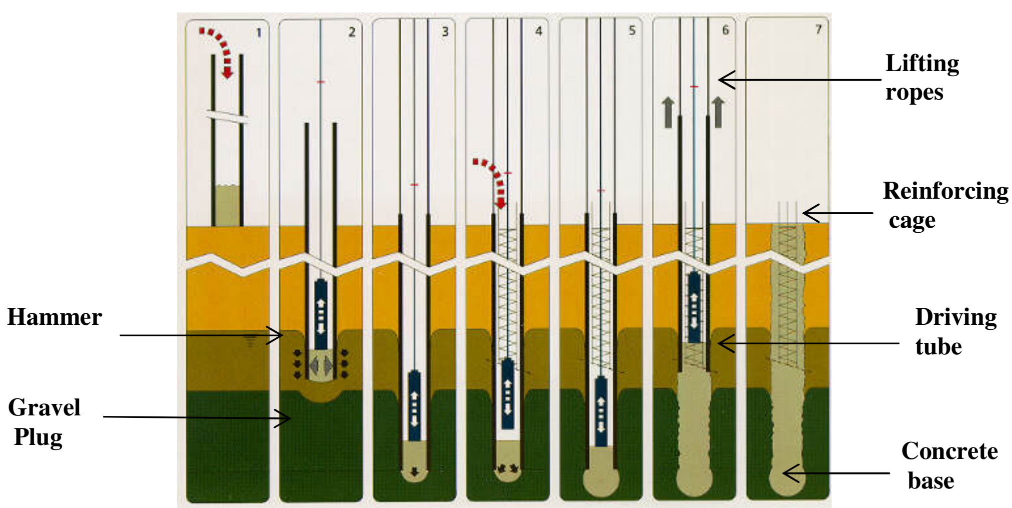


Figure 2.3 : Configuration of a Frankie pile (Tomlinson, 1994)

The Vibrex pile installed in Holland by Verstraeten BV and in Belgium by Fundex PVBA employs a diesel or hydraulic hammer to drive the steel tube which is closed at the end by a loose steel plate. The steel tube is driven, and the reinforcement cage is lowered. Finally the concrete is placed. After placing the concrete, the drive tube is pulled out by the vibrating unit clamped to the upper end of the tube. The vibrex piles are formed in shaft diameters of from 350 to 600mm and in lengths up to 38m.

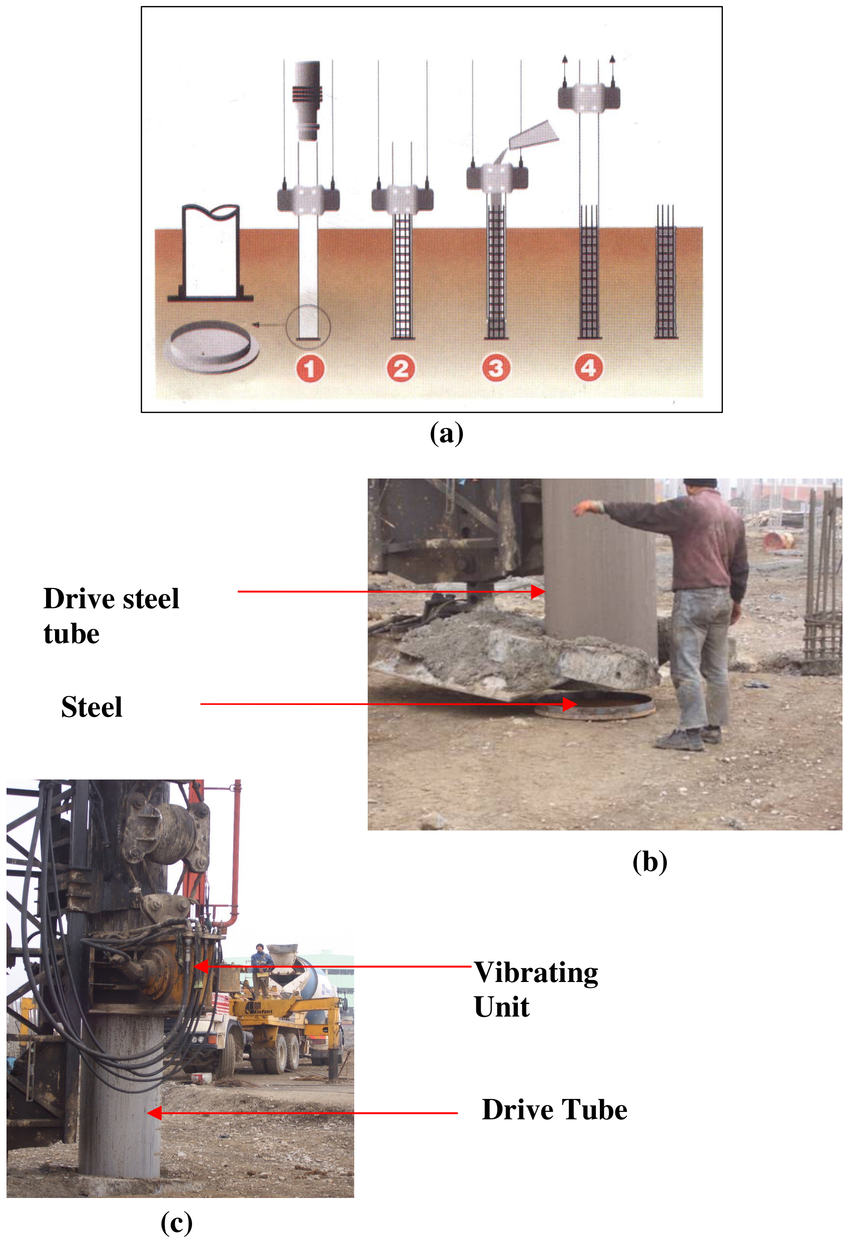


Figure 2.4 : Vibrex pile (a) stages in installing a Vibrex pile (b) Placing the driving tube onto steel plug (c) withdrawing the steel drive tube

2.2.2.2 Shell Types

Cased cast-in-place driven piles are suitable in the case of the necessity of protection of the placed concrete against ground pressures and intrusions. Installation procedure for cased driven piles consists of driving the steel casing, inspecting the casing for damages and filling the driven casing with concrete (Prakash, Sharma, 1990).The permanent lining tube can either be thin corrugated steel or pipe (either open or close ended) or precast concrete shell units. For shell type of driven and cast-in-place piles precast concrete sections can be used instead of steel lining tubes to lower down the temporary drive tube. Bottom of the corrugated steel lining tube is jointed to a steel plate or precast concrete shoe which prevents the uplift of constructed pile during removing the temporary drive tube or mandrel. Mandrel is part of a piling rig attached to the head of the lining tube which transfers the hammer blows to the casing top to drive it. Dropped-in-shell concrete piles, Raymond step-taper piles and finally the West's shell pile are examples to the shell type driven cast-in-place concrete piles.

The West's shell piling system uses short cylindrical concrete shells (Fig2.5). With joining those short units a continuous concrete shaft is obtained. Precast units are placed onto a central steel mandrel and on a precast concrete driving shoe. The completed assembly is driven to a depth limited by the length of the mandrel. When the driving process for the first mandrel section is completed, additional concrete shells are placed onto a second extension mandrel which is attached to the top of the bottom mandrel. The driving process is repeated in this manner up to the required depth is reached.

2.2.3 Bored and Cast in Place Piles (Replacement Type)

Replacement pile term based on the technique that first an equal volume of soil is removed by drilling without disturbing the adjacent soil profile and then cast-in-place pile is constructed by placing concrete or other structural element in the borehole. Replacement piles consist of bored-and-cast-in-place piles and drilled-in tubular piles.

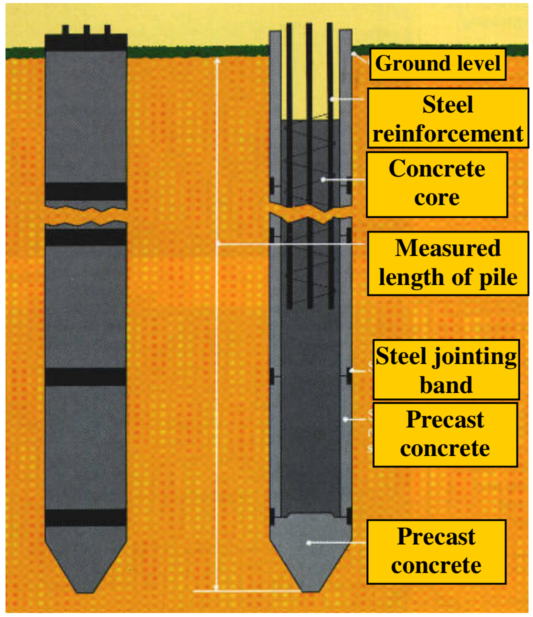


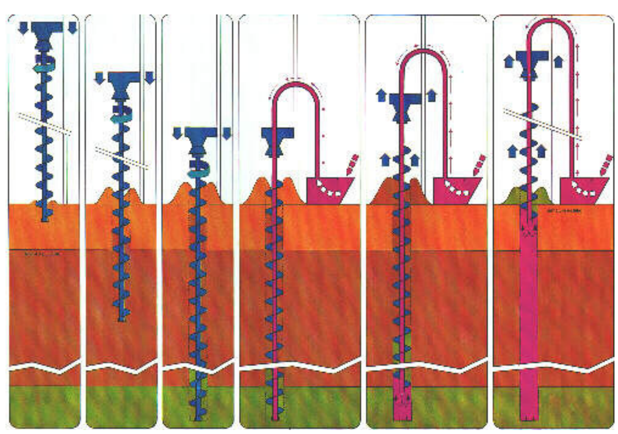
Figure 2.5 : Configuration of West's shell pile showing the driving units and installation.(www.geoforum.com)

2.2.3.1 Cast-in-Place Concrete Piles

In these type of piles a borehole is drilled in the ground by one of the following driving, boring, jetting and coring methods. Bored piles can be constructed by different drilling methods. Hand and mechanical auger techniques can drill pile shafts up to 355mm in diameter and 5m in depth. These techniques are commonly preferred for light structures. Mechanical spiral- plate, bucket augers, grabbing rigs (Tomlinson, 1994) can drill boreholes of 7.3m in diameter and depth can be reached up to 120m with larger rotary auger machines. Then concrete is placed inside the hole to form a cast-in-place concrete pile. In designing a functional pile system, predetermination of load factors which would have effects during and after installation on pile, in addition the predetermination of pile length in order to adjust field requirements have a very important role.

For precast-concrete piles, due to the driving and lifting stresses piles are subjected to lateral and uplift loads and pile design is based on those calculations. One of the advantages of cast-in-place piles is that they are designed only depending on the service loads. They do not require casting and storage yards. Also not like the unjointed precast piles, for cast-in place piles, pile lengths can be designed appropriate to the field conditions.Therefore predetermination of pile length is not critical (Prakash, Sharma, 1990).

By using rotary continuous-flight auger drilling technique, continuous-flight auger or auger injected piles also known as CFA piles are constructed (Fig2.6). At suitable ground conditions where the drilling depth is above the ground water level and where the soil can hold itself, first the auger equipment is removed and then the sand-cement grout is pumped. In unstable or water-bearing soils a flight auger is used with a hollow stem closed at the bottom by a plug (Tomlinson, 1994). Reaching the desired penetration borehole depth cement-sand mixture or concrete is placed inside the drilled shaft through the hollow stem while the auger is removed in stages with or without rotation (Fig2.6a–f). The bearing capacity and settlement behavior of CFA piles is to a large extent influenced by the equipment used and the experience of the operator. The significance of these two aspects is often underestimate or overlooked at the design stage, but it plays an important role for the performance of CFA piles. Great attention must be given to every phase of the field installation procedure, including the drilling of the hole, the casting of the shaft, the extraction of the auger and the placement of the reinforcement.



(a) (b) (c) (d) (e) (f)

Figure 2.6 : Stages for installation of CFA piles (www.geoforum.com)

For unstable soils with conventional boring techniques temporary casing must be used in order to support the soil during drilling process. Temporary casing usage shows the necessity of full length reinforcement in piles. Generally bored-and-cast-in-place piles do not require reinforcement but in the existence of uplift loads piles need to have reinforcing cage. Casing should be removed during or after placing the concrete for economical factors and also to increase the skin friction on the shaft. During removing the casing required attention must be paid in order to prevent lifting of concrete. The CFA piles become advantageous at this point when compared to the conventional boring technique for unstable ground conditions due to not requiring a temporary casing and not being affected by water conditions of soil profile.

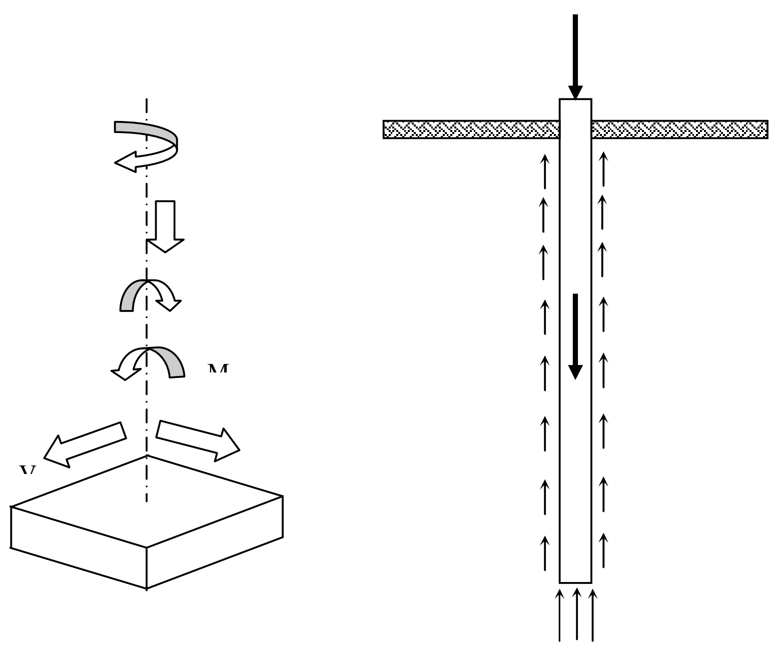
Method of load transfer of conventionally bored piles and CFA piles are also different. CFA piles are much likely to be considered as friction piles because of the fact that there is not enough data defining the soil or rock at the toe of drilled shaft. The characteristic of the bottom structure can only be distinguished by the increase in torque during driving of flight auger. In conventional boring technique the soil profile at the required depth can be analyzed by drill cuttings or by probing. If CFA piles are used as end-bearing piles, a conservative value for the allowable end-bearing pressure must be chosen (Tomlinson, 1994). The integrity of the constructed pile must be kept under control due to pumping of concrete.

2.2.4 Composite Piles

Probable problems that may arise due to site or ground conditions which is not suitable for only a one kind of pile material can be achieved by optional solutions. Composite piles have this advantage of using two different material characteristics by joining sections. Various combinations can be used such as bored piles with driven piles also such as concrete and timber piles as well as concrete and steel piles. Other combinations can also be used. Timber piles can be combined with the precast concrete pile in order to prevent the decay of timber or a cased borehole is drilled and then the timber pile is driven to the desired depth after that the rest of the borehole is filled with concrete. For marine conditions, a composite pile can be made by joining sections of precast concrete and a steel H-pile avoiding corrosion caused by sea-water. The joints between the different elements must be rigidly constructed.

3. BEARING CAPACITY OF PILES

Structures transfer normal, shear, moment and torsion loads to the designed foundation systems which they rest on (Fig3.1a). For pile foundations these mentioned loadings are classified as axial and lateral loads. Applied axial structural loadings are transferred to the bearing stratum and the surrounding soil in two ways such as skin friction and tip resistance (Fig3.1b).



(a) (b)

Figure 3.1 : Foundation systems (a) Transferred loadings to the foundation

(b) Forces for pile foundation

The basic difference between displacement and non-displacement piles requires a different approach to the problems of calculating bearing capacity. The bearing capacity and settlement calculations of a pile has two important parameters in the design. Load-settlement relationship graph shows the elastic and permanent deformation of pile under gradual loading (Figure 3.2b). At the early stages of loading (point A), the settlement is relatively small and the elastic behavior is observed for the pile and for the soil surrounding the pile. At this loading step the pile head will return in its initial position if the load is removed. The applied load is fully carried by the skin friction on the upper part of the shaft (Figure 3.2b).

As the load is increased, on the settlement-load graph reaches point B (Fig 3-2a). Relatively a large settlement (deformation) is observed at that point when compared to the previous loading stage. Some part of the observed deformation returns back due to little elastic behavior but this time majority portion of the settlement is occured as permenant deformations at the pile head. For this condition the applied load is not only carried by the skin friction, end-bearing also contributes to the load bearing. The skin friction resistance is fully developed after only 5-10mm of downward displacement. Much more displacement is required to fully mobilize the end bearing capacity approximately about 10% of the pile diameter (Coduto, 1994).

For the failure condition of the pile at point C, the settlement increases rapidly with little further increase of load. A large proportion of the ultimate load is finally carried by end-bearing. As it is seen in Figure 3-2b load carried in end-bearing value is close to the loading value applied at the top of the pile for point C situation.

Pile loading tests are performed in order to determine axial load (bearing) capacity more accurately. Such load-settlement curves (Fig 3.2) representing pile behavior under loading conditions are obtained at the end of the test. Loading test methods give more reliable results in determining the pile capacity when compared with static or dynamic methods. However, load tests are also much more expensive and thus must be used more judiciously (Coduto, 1994). Nevertheless, engineers judge the accuracy of all other those mentioned methods by comparing them to full scale loading tests.

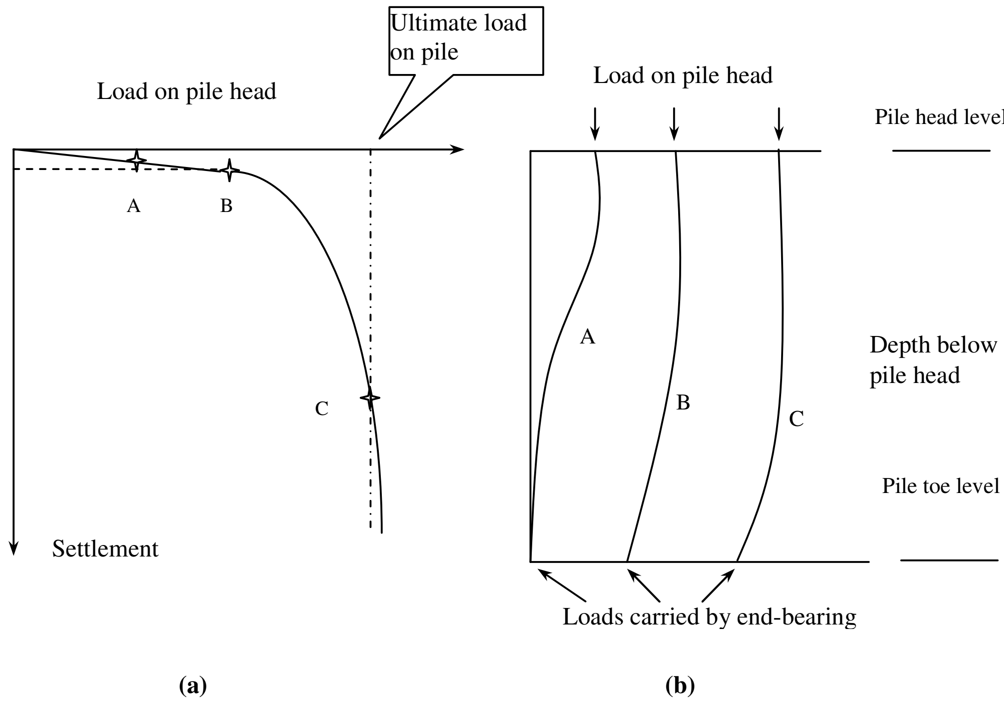


Figure 3.2 : Load distribution for different loadings along the pile shaft

3.1 Static Methods

Determination of bearing capacity methods depending on the soil properties or on the results of in-situ tests as the standard penetration test or cone penetration test are known as static methods (Coduto, 1994). Soils are roughly classified in two broad categories as cohesionless and cohesive soils. Static method is differently applied to those two types of soils due to their different behaviors under loading. Those formulas require soil parameters such as the friction angle "ϕ", or the undrained shear strength, "c_u".

Shaft friction and end-bearing carrying capacity proportion depend on the shear strength and elasticity of the soil. Elastic movement characteristic of the pile shaft transfers the applied load to the surrounding soil via the mobilized skin friction along the pile shaft. Both the skin friction and end bearing resistance have contribution to the calculation of ultimate bearing capacity of a pile at the failure state (Eq 3.1).

$$Q_p = Q_s + Q_b - W_p \quad \textbf{(3.1)}$$

Where;

Q_s : Ultimate shaft resistance in skin friction

Q_b : Ultimate resistance of base

W_p: Weight of pile

W_p is usually so small compared to Q_p, so it is often neglected.

$$Q_u = Q_b + Q_s = q_b A_b + f_s A_s \quad \textbf{(3.2)}$$

q_b : Unit bearing capacity of the pile toe

A_b : Cross-sectional area of the pile toe (base)

f_s : Average unit shaft friction on the pile shaft

A_s : Surface area of the pile shaft

This equation mathematically defines the movement of the pile shaft and pile toe with respect to the surrounding soil under the applied load.,

$$R_{cd} = \frac{R_{sk}}{\gamma_s} + \frac{R_{bk}}{\gamma_b} \quad \textbf{(3.3)}$$

γ_s : Partial safety factor for shaft friction

γ_b : Partial safety factor for base resistance

These values are given as follows;

γ_s =1.3 for driven and bored piles

$$\gamma_b = \begin{cases} 1.3 & \text{------ for driven piles} \\ 1.6 & \text{------ for bored piles} \\ 1.45 & \text{------ for flight auger piles} \end{cases}$$

The shaft resistance is;

$$R_{sk} = \sum_{i=1}^{\pi} q_{sik} A_{si} \qquad \textbf{(3.4)}$$

q_{sic} : Characteristic value of the resistance per unit of the shaft in layer i

A_{si} : Nominal surface area of the pile in soil layer i

and the base resistance is;

$$R_{bk} = q_{bk} A_b \qquad \textbf{(3.5)}$$

q_{bk} : Characteristic value per unit area of base

A_b : Nominal area of the pile base

Ultimate pile load capacity is determined by loading tests. At the end of the applied test two components of bearing capacity (as shaft resistance and end bearing) can either be measured instrumentally or be obtained by graphical interpretation. At this point Eq. (3.3) is used to calculate the design bearing resistance (R_{cd}).

Allowable pile capacity is another important parameter including the calculated bearing capacity of pile. This parameter keeps the calculations within the conservative limits. It is a capacity which takes into account the pile's bearing capacity, the materials from which the pile is made, the required load factor, settlement, pile spacing, down-drag, the overall bearing capacity of the ground beneath the piles and other relevant factors. The allowable pile capacity indicates the ability of the pile to meet the specified loading requirements and is therefore required to be not less than the specified working load" (Tomlinson, 2001).

3.1.1 Bearing Capacity Calculation of Piles for Cohesionless Soils

Driven piles displace the coarse grained soils during their installation by hammering or jacking. This process can be thought as some kind of compaction in loose coarse-grained soils. For very loose soils compaction causes depression at the ground surface around the pile as a result of driving. During driving, the relative density is increased close to the pile due to vibrations and lateral displacement of soil. The increase in relative density increases the load capacity of a single pile and pile groups. The pile type also affects the amount of change in relative density. Piles with large displacement characteristics such as closed-end pipe and precast concrete increase the relative density of cohesionless material more than small displacement steel H-pipes or open-end pipe piles.

For dense coarse grained soils ground heaving can be observed due to very little further of compaction. Heaving is the result of the shear failure but the shear resistance is very high for dense soils. This high resistance characteristic of soil bring out the necessity of heavy driving during penetration in order to be able to reach the desired depth. Heavy driving decrease shear resistance of the soil beneath the pile toe by deformation of soil particles. Besides it is not advantageous to choosing driven piles in dense coarse-grained materials due to the possible damages to pile itself. Their compaction effects in loose or medium-dense coarse grained soils leads to high end-bearing resistance different from the bored pile types. Boring causes to loosen of coarse material which results in loss of end-bearing capacity. The shaft frictional resistance of piles in coarse soils is small compared with the end resistance. The full unit skin friction capacity, “q_s”, develops when the shear stress along the soil-pile interface exceeds the shear strength (Coduto, 1994). As mentioned before this condition is possible with 5-10mm of pile displacement. The unit shaft resistance q_s in Eq. 3.6 is given by the basic equation;

$$q_s = K\sigma'_{vo} \tan \delta \qquad \textbf{(3.6)}$$

Where;

K : Coefficient of lateral earth pressure

σ'_{vo} : Average effective overburden pressure

δ : Angle of friction at the pile/soil interface

$\tan\delta$: Coefficient of friction between the soil and the pile

The coefficient of lateral earth pressure, K, is generally not equal to K_o which is the coefficient of lateral earth pressure in the ground before the installation of pile. This difference between the K values depends on the degree of displacement, soil consistency and special construction techniques. The largest possible value of K is the coefficient of passive earth pressure, K_p.

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) \qquad \textbf{(3.7)}$$

It is difficult to determine the value of K_o due to the difficulties in determining whether the soil is normally or over consolidated. The value of δ is independent of soil density and can be obtained from laboratory shear box tests. K_o is not constant over the depth of the pile shaft. It depends on the relative density of the soil as well as the displaced volume of soil.

The base resistance component q_b in Eq. 3.5 is given as;

$$q_b = N_q \sigma'_v \qquad \textbf{(3.8)}$$

Where;

N_q : Bearing capacity factor

Bearing capacity factor is related to the peak angle of shearing resistance "ϕ'" of the soil and the slenderness ratio of the pile which is defined as the ratio of length of pile to diameter of pile (L/R).

The ultimate bearing capacity is calculated and is divided by a safety factor to obtain the allowable pile capacity. Safety factor value varies depending on the pile diameter, soil compressibility and thus settlement limitations. A pile having a shaft diameter of not more than 600mm has a settlement limitation value of 15mm. This criteria is provided with the safety factor of 2.5.

In cohesionless soils, the ultimate bearing capacities of piles can also be determined by in-situ test results of Standart Penetration Test (SPT) and static Cone Penetration test (CPT). Relative density, angle of shear resistance ϕ' is determined by the SPT or the CPT in-situ tests. The CPT is the most reliable method due to the fact that it is analogous to a pile driven into the ground. Also it is not affected by the drilling disturbance. CPT apparatus with the measurement technique can be assumed as a model pile simulating the soil displacement to some extent although the volume of displaced soil is not exactly the same. The CPT can lead to erroneous results in coarse grained soils having cobbles, boulders and gravel. In these conditions SPT is chosen. For sands and gravels, the end bearing resistance is defined depending on the corrected SPT N values as well as the pile dimension aspect ratio.

In the calculation of bearing capacities using static formulas there are some points to be careful about depending on the type of pile. Driven and cast-in-place piles are formed by driving a tube into the ground. After reaching to desired penetration depth the driven tube is filled with concrete or the tube is withdrawn and the pile shaft is filled with concrete. For piles which the tube is left in ground and for precast concrete piles shaft resistance is calculated by the static formulas given above in Eq(3.6), but for the pile types in which the driven tube is withdrawn, it is hard to be able to determine whether the pile shaft is in contact with the loose or dense material in order to choose the correct soil parameters. Withdrawal of the tube and compaction of concrete during placement affect the skin friction value.

End-bearing calculations for closed end driven tubes are made depending on the base area at the pile tip. Bulb effect which is formed at the base of the pile bring out difficulties in calculations. Bulb sizes varies depending on the soil type.

Bored piles are constructed by drilling a pile shaft supported by temporary casing with one of those mechanical auger, cable percussion or grabbing rig methods. Concrete is then placed while the casing is withdrawn. Bored pile construction technique shows the fact that drilling in coarse-grained soil results in loosening of the soil even it is in dense or medium-dense condition. Eq.3.6 is used in order to calculate the shaft resistance assuming K_s to be 0.7 to 1.0 times K_o which is at rest earth pressure coefficient and ϕ value will be represantative of loose conditions. Again for the determination of N_q value peak angle of shearing resistance value is chosen as suitable for loose conditions. "Research by Fleming and Sliwinski in 1977 appeared to show that the shaft friction on bored piles concreted under a bentonite slurry could be calculated on the assumption that the ϕ value would correspond to undisturbed soil conditions (Tomlinson, 2001). Loose condition approach in calculation of bearing capacities of bored piles in coarse grained soils shows that the ultimate bearing capacity of bored piles are lower than that of driven type of piles.

3.1.2 Bearing Capacity Calculation of Piles for Cohesive Soils

The bearing capacity of piles driven into cohesive soils as clay and clayey silts is equal to the sum of the shaft resistance and end bearing resistance. The end bearing capacity of piles in cohesive soils should be based on the undrained strength of the soil due to the reason that the cohesive soil characteristic does not permit the dissipation of excess pore water pressure in a short time. This property leads to the fact that the piles develop their full bearing tip capacity after some time but the applied live loads with the dead loads cause development of new excess pore water pressures so calculations must be done depending on the undrained conditions for conservative results.

The end resistance is given by the equation;

$$Q_b = q_b A_b = N_c c_{ub} A_b \tag{3.9}$$

N_c : Bearing capacity factor
c_{ub} : Undrained shear strength at the base of the pile
A_b : Base area of the pile toe
q_b : Net unit end bearing resistance

N_c value is taken as 9 provided that the pile is driven at least five diameters into the bearing stratum. The undrained shear strength is taken as the undisturbed shear strength.

Although the drained strength conditions control the skin friction capacity, performed analyses are based on the empirical correlations with the undrained strength; c_u. These correlations implicitly "convert" the undrained strength to the drained strength (Coduto). This technique is widely preferred due to the reason that the determination of c_u by the application of triaxial or by shear test is inexpensive and easy when it is compared to the determination of the parameter K_o the coefficient of lateral earth pressure which is required for the calculations in drained conditions. This type of undrained condition calculation of skin friction is known as the α method since defining the unit skin friction resistance by the adhesion factor, α. The skin friction on the pile shaft is given by the equation;

$$Q_s = q_s A_s = F\alpha_p c_{avg} A_s \tag{3.10}$$

Where;

q_s: Design ultimate unit shaft friction
$F\alpha_p$: Adhesion factor

c_{avg}: Average undrained shear strength over the depth of the pile shaft

A_s: Surface area of shaft over the embedded depth within clay

The adhesion factor α can either be determined by site pile load tests or by empirical graphical relations as a function of c_u . The working load for all pile types is equal to the sum of the base resistance and the shaft friction divided by a suitable safety factor. As a result allowable bearing capacity is obtained;

$$Q_a = \frac{Q_b + Q_s}{F_s} \tag{3.11}$$

where F_s value can be taken as 2.5 but the allowable bearing capacity should not be more than the value given by the equation;

$$Q_a = \frac{Q_b}{3} + \frac{Q_s}{1.5} \tag{3.12}$$

3.2 Dynamic Methods

Dynamics of pile driving is also a major category in determination of pile bearing capacity. Dynamic analysis evaluate the static load capacity depending on the effort required to drive the pile. Generally the piles that are more resistant to driving should have a greater static load capacity (Coduto,1994).

The pile driving formulas depend on the emprical relationship between the hammer weight, blow count and other factors with the static capacity. Various pile driving formulas had been derived base on different approaches. Altough these formulas have different formats and all share a common methodology of computing the pile capacity based on the driving energy delivered by the hammer (Coduto, 1994). By using pile driving formulas, it is possible to calculate the pile capacity just only by considering the final blow count. They have wide application areas but on the contrary their accuracy is another important discussion topic.

For all pile driving formulas, the common assumptions that have been mentioned just in the previous paragraph can be defined mathematically as follow if it is assumed that no impact or elastic losses occured, and the mechanical efficiency of the hammer were hundred percent (Chellis) :

$$P_a = \frac{W_r h}{sF} \tag{3.13}$$

Where:

P_a : Net allowable vertical load capacity
W_r : Hammer ram weight
h : Distance that hammer falls
s : Penetration per blow at the end of driving
F : Factor of safety
The Sanders formula proposed in 1851 was obtained by applying a factor of safety of 8 to the general formula where Merriman used the same terms with a purported factor of safety of 6.

- Sanders Formula

$$P_a = \frac{W_r h}{8s} \tag{3.14}$$

- Merriman Formula

$$P_a = \frac{W_r h}{6s} \tag{3.15}$$

Since in actual driving conditions, there are losses due to efficiency, impact, and elastic compressions of the cap, pile and soil (Chellis). Due to the inefficiencies in the driving system the various losses of energy are determined from the pile load tests. These effects are reflected to the formulas through different emprical correction factors (Coduto). The assumptions base on those correction factors for the energy loss definition brings out several different pile driving formulas.
These actual energy losses are added to the general formula to obtain:

- Hiley formula

$$P_a = \frac{e_f W_r h}{s + 1/2\left(C_1 + C_2 + C_3\right)} \frac{W_r + e^2 W_p}{W_r + W_p} \quad \textbf{(3.16)}$$

Where:

W_r : Weight of ram
W_p : Weight of pile
e_f : Mechanical efficiency
e : Coefficient (or degree) of restitution (elasticity)
C_1 : Temporary elastic compression in the cap
C_2 : Temporary elastic compression in the pile
C_3 : Temporary elastic compression in the soil

As mentioned before, different assumptions lead to different dynamic analysis formulas. These assumptions can be given as follow;

If it is assumed that there are no elastic losses in the cap or soil quake and the hammer is mechanically hundred per cent efficient thus e_f =1; and solve for P_a which is the net allowable vertical load capacity The Universal or Stern formula is obtained. If the impact is assumed to be perfectly inelastic instead of semielastic, which means the “e” value is equal to zero (e = 0), the Redtenbacher formula is obtained. But for the case in which the impact loss is entirely neglected then the Weisbach formula is formulated. Pacific Coast Uniform Building Code formula bases on the following statements as the hammer is assumed to be mechanically hundred percent efficient, twice the average elastic loss is used taking into account the full length of the pile appointed and finally fixed values are selected for e.

By taking the Hiley formula and assuming that the mechanical efficiency is hundred percent (e_f = 1.0), that the impact is perfectly inelastic (e=0) and that there are no elastic losses in the cap, pile or soil the Dutch formula is obtained (Chellis):

- Dutch formula

$$P_a = \frac{W_r h}{s} \frac{W_r}{W_r + W_p} \frac{1}{F} \qquad \textbf{(3.17)}$$

Most commonly used pile driving formula is Engineering News Formula which was derived by Wellington in 1888 (Chellis):

- Engineering News formula

$$P_a = \frac{W_r h}{F(s + c)} \qquad \textbf{(3.18)}$$

If in the Hiley formula (Eq3.16) the impact loss is entirely neglected, the mechanical efficiency taken as hundred percent. If the elastic losses in the cap, pile, and soil represented by a constant term of 1.0, The Engineering News formula is obtained. In this formula the safety factor is taken as 6. Wellington had been derived his formula based on timber pile data driven with drop hammers. After the formula has started to be used widely, it has been modified for other types of piles and hammers. The modified Engineering News formula (Eq.3.19) depending on the pile driving tests performed by the Michigan Highway Department in 1961 (Coduto).

$$P_a = \frac{0.0025E(W_r + e^2 W_p)}{(s + 0.1)(W_r + W_p)} \quad \textbf{(3.19)}$$

Where:

E : Rated hammer energy per blow

W_p : Weight of pile and driving appurtenances

W_r : Weight of hammer ram

e : Coefficient of restitution (elasticity)

Other earlier modifications of the Engineering News formula can be given as follows: Vulcan Iron Works formula, United States Steel formula, Bureau of Yards and Docks formula and finally the Benabencq formula.

3.2.1 Accuracy of Pile Driving Formulas

The dynamic pile driving formulas are widely used in practice applications for centuries by engineers in order to determine the pile capacity. In spite of their common use, the accuracy of formulas has been discussed for years. Cummings was the first in 1940 who described the shortcomings of them. Basically, the dynamic formulas are inaccurate due to their over-simplicity in modeling the hammer, driving system, pile, and soil. In fact, most of the foundation engineers agree with the fact that dynamic formulas are dangerously unreliable today (Likins, Rausche,Hussein). In the following years, Terzaghi and Peck (1942) had also emphasize the weaknesses of driving formulas and based their claims upon the comparisons between pile load tests and capacities predicted by pile driving formulas by which the inaccuracies have clearly been demonstrated.

The fundamental application of analysis of pile driving (dynamic analysis) is based on the Newton's theory of rigid-body impacts and the principle of conservation of energy. This approach has been accepted and used widely with practitioners due to its simplicity of application. The main shortcoming of the formulas are the difficulties in considering the energy losses accurately in a real pile driving process (Coduto, 1994) as well as with the application of Newton's theory to impacts produced by pile driving hammers. Pile driving is not a simple problem of impact that may be solved directly by Newton's laws. Rigid-body assumption considers a pile structure as a rigid body at all. Thus the flexibility of pile is neglected.

Pile driving is a complex process involving the use of various types of hammers, cap blocks, pile caps, cushion blocks along with the use of different pile types and elastic-plastic behavior of the ground associated with other problems in soil mechanics. As a result of these difficulties, all pile-driving formulas are partly empirical and consequently can be applied only to certain types or lengths of piles (Smith, 1960). The pile, hammer, and soil type combination used in the generation of dynamic formulas can be different from the one as those in the field where it is applied. This is the major probable reason for the inaccuracies in the original Engineers News Formula due to the reason that the formula designed only to serve for timber piles driven with drop hammers.

The second reason that contributes to the inaccuracy of the formulas is the freeze effect that is not considered. Briefly, the freeze effect can be defined as the temporary loss of bearing capacity piles driven into saturated clays because of the produced excess pore water pressure. After some time as the dissipation of the excess porewater pressure occurs, bearing capacity of pile returns. This process is known as freeze or setup. Thus the dynamic formula gives the prefreeze capacity of piles. Besides these, the hammers do not always operate at their rated efficiencies also the energy absorption properties of cushions can vary significantly. As mentioned before the formulas do not account for flexibility in the pile and there is no simple relationship between the static and dynamic strength of soils as it is considered to be so. Other most important point is the high factor safety values used in formulas bearing suspects about their accuracies. Finally they do not provide any information about the drivability of pile. As a result of these many difficulties, the necessity of an alternative dynamic analysis method is turned out which is the wave equation analysis.

4. WAVE EQUATION ANALYSIS

As described in detailed in the previous section, pile driving formulas (dynamic formulas) assumes the pile as a rigid body and apply the classical Newtonian physics in defining the pile behavior under dynamic forces along with the inadequate modeling of soil as it interacts with the pile. These weaknesses and others were not as apparent when wood piles installed using drop hammers since the formulas had been derived for a specific type of pile and hammer system. With the introduction of concrete and steel piles these weaknesses become critical especially when cracking occur in concrete piles at both the top and the bottom during pile driving. This phenomenon can not be determined or quantified with the use of dynamic formulas.

This assumed model for dynamic analysis is poor due to the reason that the impact energy from the hammer actually travels down the pile as a stress wave. Thus pile driving is a problem in longitudinal wave transmission that can be analyzed in a general way by the wave equation (Smith, 1960). Application of wave motion theory into pile driving process makes it possible to answer the following questions which are very important for pile design and construction control (Goble, Rausche, Likins,1980);

- What is the static driving resistance of the pile during or after driving based on the pile driving records? (static capacity back analysis)
- Can the pile be driven for a complete description of Pile-Soil-Hammer properties? (driveability)
- Is the pile structurally sound? (pile integrity)
- What are the stresses in the pile during driving?
- What is the efficiency of the driving system?

Wave equation analysis is based on the theory of propagation of one-dimensional stress-wave in a long slender rod. For simple boundary conditions as it is in a free rod situation the wave equation can be solved by analytical methods (theoretical methods) but for more complex boundary conditions as it is in the real life applications numerical solution is required in order to solve the wave motion.

4.1 Brief History of the Application of Stress-Wave Theory to Piles

In the 1860's a Frenchman, A.J.C Barre de Saint Venant was the first who had applied the principles of conservation of mass and momentum to the water flow in an open channel. The application had been resulted in two quasi-linear differential equations. Saint Venant again had been the first who derived a theoretical solution which is known as the method of characteristics.

The method had started to be applied to pile driving studies in the 1930's around the world. Isaacs the Australian engineer was the first who used the wave equations for modeling the pile driving analysis (Middendorp,Verbeek, 2004). In the following years in 1940 Cummings published a paper defining the superiority of the stress-wave analysis over the dynamic analysis based on dynamic formulas derived with the Newtonian law of impact energy. Karl Terzaghi, in 1943 provides an extensive discussions on the "phenomena of wave propagation which occurs in a pile after it has been struck by a falling hammer" and the application of "the theories of longitudinal impact on piles" for rational analysis (Hussein, Goble).

Wave equation solution based on the method of characteristics originally has been developed for the propagation of one dimensional stress wave through a free rod. Method does not take account of the friction and resistance where it is valid for real physical conditions. But for a real pile driving situation the shaft friction or toe resistance factors are in question.

Between the years from 1956 to 1974 different solution models have been proposed in order to incorporate the toe resistance and the characteristics method have also been extended by formulating the theoretical solution for the shaft friction. As the boundary conditions get complex; as getting far from the ideal conditions; analytical (theoretical) solution methods become insufficient. If the shaft friction for piles is assumed to be depending on the velocity or displacement, then there occurs the necessity of numerical integration of the differential equation.

In 1960's Edward A. L. Smith produced the first general solution for the practical application of stress wave theory to piles. Smith has been interested in finding a way to predict pile drivability more effectively than the currently used methods. Smith formulated a numerical solution using discrete elements with the finite difference equations. For this purpose the pile have been modeled by the series of point masses, springs and dash–pots in order to represent the pile, shaft friction and toe resistance combination. This model is known as the Smith's Model or as the Lumped Model. Model details will be discussed in the following section as the theoretical background. Smith's work is generally considered to be the first application of digital computers in a civilian engineering application, and his landmark paper in 1960 is among the classics in the engineering literature. Smith's model, methodology, and terminology as a whole called as the wave equation are still the basis for modern wave equation analysis.

In the same years different approaches were developed for the analysis of a pile under impact. Alternatively to the Smith's discrete model, Donnell-de Juhasz have developed a graphical method solution of the wave equation. The Juhasz's method of characteristics is more exact for ideally elastic, continuous systems. However, it is more difficult to apply to the pile capacity problem due to the difficulty of including a realistic soil, hammer and driving system model.

Starting in the 1950's, the application of stress-wave analytical methods for the analysis of pile driving started to be used in the petroleum industry for large fixed offshore platforms in deep water. Thus, the wave equation computer analysis programs were initially applied to evaluate the drivability of offshore piles. Today, pile dynamic pile testing is routinely performed as an integral part of offshore pile installation for assessments of hammer driving system performance, pile driving stresses and structural integrity as well as with the soil resistance and pile load bearing capacity (Hussein, Goble).

One dimensional elastic wave studies leads back to 1910’s when the early experimental investigators devised simple apparatus in order to be able to analyze the stress-time duration of traveling waves. In 1914 Bertram Hopkinson was the first who had designed the Hopkinson’s Bar which consists of a cylindrical steel bar several feet in length an d about one inch in diameter suspended in a horizontal position by threads (Hussein, Goble). In 1948, the electronic type of this bar is invented by R.M Davies for study of stress-wave propagation. Davies’ experiments measured the shape and duration of the traveling stress-wave and confirmed predictions based on the earlier theories of elasticity (Hussein, Goble).

In 1938 W.H. Glanville in England, had measured the strains occurred in concrete piles by using the strain measurement device (bonded wire strain gages) during driving. In this way, tension cracking occurred on concrete piles had been studied. As such Glanville is the pioneer of the Pile Driving Analyzer (PDA). With the development of the bonded resistance strain gage in the years between 1938 and 1957 several measurements had been performed. In 1960 studies concerning the pile driving hammer performance measurements was performed. The main purpose was to evaluate hammer performance by measuring the energy transmitted to the pile (Hussein, Goble). Force and acceleration measurements were taken at the pile top. It was the first time that the acceleration, force and motion was measured during pile driving.

The improved measurement techniques covering the force and acceleration measurements at the pile top made possible the development of different analysis methods. The first developed analysis technique assumes the pile and the total driving system as a single rigid body and Newton’s Second Law can be applied at the instant of zero velocity but in order to be able to evaluate the soil response and the pile capacity the original method had been improved to represent the pile as an elastic rod. Elastic rod assumption takes into consideration the propagation of stress-wave along the total length of the driven pile. This solution could be applied and solved in real time for each hammer blow by special purpose analog electronic equipment. Results had been obtained, giving instantaneous answers for the capacity at the time of testing. By the mid 1970's with the advent of digital computation, improved analysis methods became available to improve the accuracy of the capacity prediction. While the initial goals were to evaluate pile capacity, it became apparent that pile driving stresses, pile integrity and hammer performance questions could be assessed by using stress-wave theory based analysis methods. Finally the application of stress-wave theory to piles becomes complete with the low-strain integrity testing methods.

4.2 Theoretical Background

Wave equation assumes the pile as a prismatic elastic rod where the generated wave propagates with a constant velocity depending on the rod or pile material. The wave equation concept mainly depends on the principle that the generated shock wave advances in the elastic rod like a wave in a cable. Entire material does not feel the impact force immediately. Its effect is carried by the traveling wave.

During driving a pile each hammer blow loads the pile and a wave action occurs within the pile. Pile discontinuities and interactions with the surrounding soil s to this wave action. Stress-wave propagation within the pile body and associated reflected waves can be explained mathematically with the one-dimensional wave theory because of the large dimension of the length of the pile compared to its diameter.

One dimensional wave equation can be derived mathematically with the combined application of Hooke's and Newton's laws on the equilibrium of forces acting on an infinitesimal piece of bar or pile section (Fig 4.1).

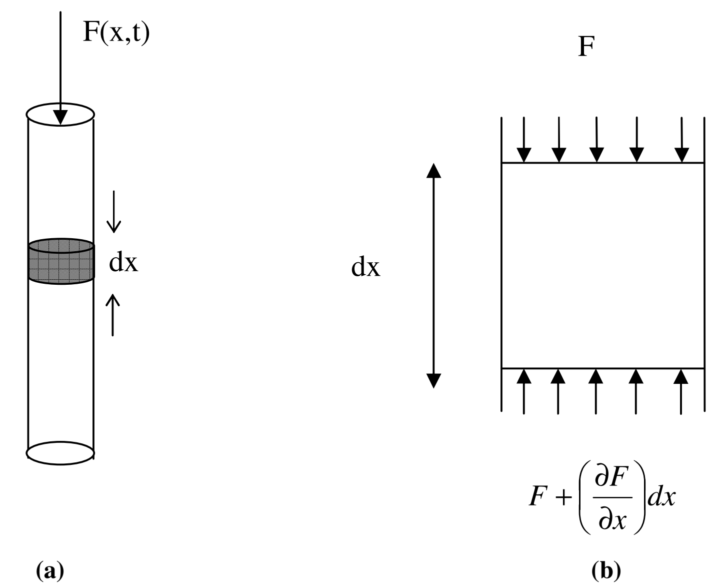


Figure 4.1 : (a) Cylindrical rod loaded at one end, **(b)** Enlarged infinitesimal piece of the elastic rod.

Displacement u_x due to compression for the infinitesimal rod section positioned at the distance x enlarged in (Fig. 4-1b) is the function of dimension and time.

$$u_x = u_x(x,t) \qquad \textbf{(4.1)}$$

From the Newton's Second Law ;

$$F = ma \qquad \textbf{(4.2)}$$

The Equation of Motion (EOM) for an infinitesimal piece of bar, dx, is given by (Figure 4-1):

$$F - \left[F + \left(\frac{\partial F}{\partial x}\right)dx\right] = \rho A dx \frac{\partial^2 u}{\partial t^2} \qquad \textbf{(4.3)}$$

$$-\frac{\partial F}{\partial x}dx = (\rho A dx)\frac{\partial^2 u}{\partial t^2} \qquad \textbf{(4.4)}$$

Where;

F : the axial force in the rod [N]

x : referential coordinate on the rod

u: displacement

t : time [s]

dx : infinitesimal thickness of pile [m]

ρ: mass density of the rod or pile material [kg/m^3]

$\frac{\partial^2 u}{\partial t^2}$: acceleration of the considered section [m/s^2]

The force can also be related to the strain (ε) from the Hooke's Law:

$$F = -EA\varepsilon = -EA\frac{\partial u}{\partial x} \qquad \textbf{(4.5)}$$

$$\frac{\partial F}{\partial x}dx = \left(\frac{\partial u}{\partial x} + \left(\frac{\partial^2 u}{\partial x^2}\right)dx\right)AE - AE\frac{\partial u}{\partial x} \qquad \textbf{(4.6)}$$

Where;

A : cross section [m^2]

$\varepsilon = \frac{\partial u}{\partial x}$

ε : strain

$E = \frac{\sigma}{\varepsilon}$

E : Elasticity modulus of the pile material [Pa]

From the equations; (4.4) and (4.6) the fundamental one-dimensional wave equation of propagation in an elastic medium is obtained:

$$EA\frac{\partial^2 u}{\partial x^2} = \rho A\frac{\partial^2 u}{\partial t^2} \quad \textbf{(4.7)}$$

or

$$\frac{\partial^2 u}{\partial x^2} - \frac{1}{c}\frac{\partial^2 u}{\partial t^2} = 0 \quad \textbf{(4.8)}$$

Where;

c is the wave propagation velocity

$$c = \sqrt{\frac{E}{\rho}} \quad [\mathrm{m/s}] \quad \textbf{(4.9)}$$

The general solution for Eq. 4.7, a partial differential equation in displacement and time, is:

$$u = u_1(x - ct) + u_2(x + ct) \quad \textbf{(4.10)}$$

That is, the general solution consists of two traveling waves propagating in the rod (the pile), with constant velocity +c (downward wave) and –c (upward wave), but in opposite directions along the lines ($x \pm ct$), called characteristics (Fig4-2). The velocity is determined by the material mechanical properties while the form of the waves defined by the constant values of u_1 and u_2 are determined from the initial and boundary conditions.

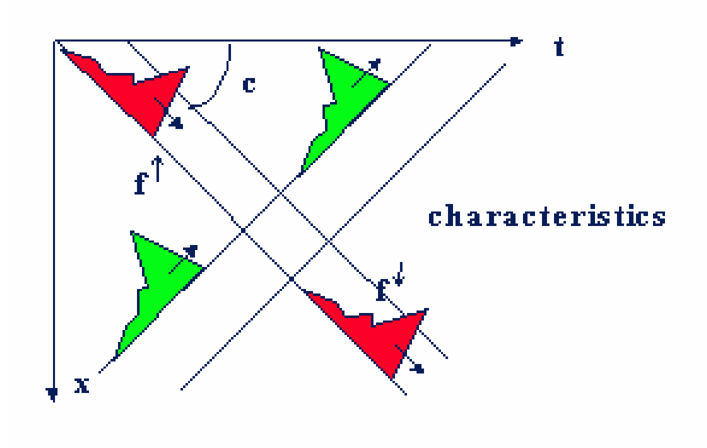


Figure 4.2 : Two fundamental forms of traveling waves in opposite directions along the characteristic lines (TNOWAVE manual)

The functions u_1 and u_2 can be used to describe any behavior such as force or pile particle velocity propagating in the pile respecting the elastic propagation assumptions. Superposition of both forms respects the general wave equation.

For the particle velocity v and the axial force F, the following equations can be derived:

$$v = \frac{\partial u}{\partial t} = \frac{\partial u_+}{\partial (x-ct)}(-c) + \frac{\partial u_-}{\partial (x+ct)}(+c) = v_+ + v_- \quad ^{*} \qquad \textbf{(4.11)}$$

$$F = -EA\frac{\partial u}{\partial x} = -EA\left(\frac{\partial u_+}{\partial (x-ct)} + \frac{\partial u_-}{\partial (x+ct)}\right) = F_+ + F_- \qquad \textbf{(4.12)}$$

Downward traveling wave particle velocity v (+) and axial force F(+) and similarly upward traveling wave particle velocity v(-) and axial force F(-) can be combined in Eq. 4.11 and Eq. 4.12 to define a new parameter called Impedance (Z) which is defined as the ratio of the driving force to the associated velocity.

---

[*] + : downward direction
 - : upward direction

$F_+ = Z.\ v_+$ and $F_- = -Z.\ v_-$ **(4.13)**

where;

$$Z = \frac{EA}{c} = A\sqrt{E\rho} \quad \textbf{(4.14)}$$

and

Z : impedance of the rod

4.3 Analysis Methods

As mentioned in the previous section, St Venant was the first who introduced the analysis of one-dimensional wave propagation into pile driving applications. Depending on the one dimensional wave propagation theory in an elastic rod he had derived the differential equation defining the wave motion and he had presented its solution. His solution is known as the method of characteristics which forms the basis for analytical methods. Analytical method solutions work for the limited cases of boundary conditions thus has only limited success because of difficulties in describing a real hammer-pile-soil system. With the advent of the digital computer, discrete solutions of the wave equation became practical. Smith developed the original model (Goble, Rausche, Likins, 1980).

4.3.1 Analytical Method

Analytical method principal is based on solving the differential equations by using conventional solving methods developed in mathematics. Method of characteristic in other words the d'Alembert method is one of those solving techniques. It has been commonly used for solving the second degree differential wave equation for pile analysis studies. In the Method of Characteristics where no pile-soil interaction exists, time is subdivided into discrete time intervals, Δt, and the pile is subdivided into elements of length, $\Delta l = c \times \Delta t$. Thus, the length of the element depends on the wave speed and, hence, the modulus of elasticity and density. Furthermore, discontinuities in the pile impedance are allowed to be exist only on element borders. Within each element, the pile properties are constant. In Figure 4.3, a pile section is shown defining the downward and upward traveling waves within an individual pile element.

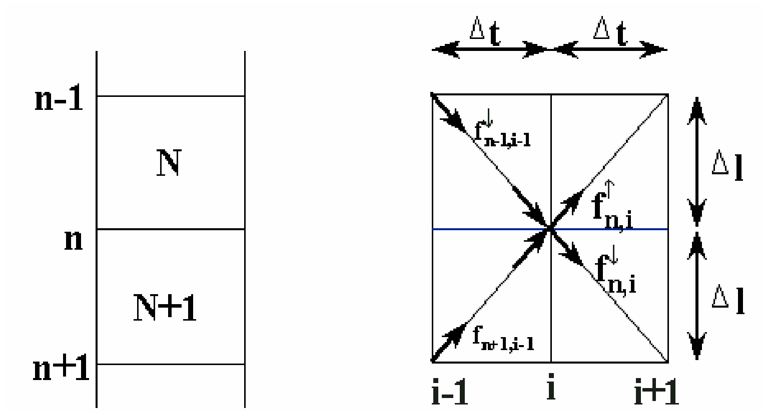


Figure 4.3 : Downward and upward traveling waves through pile (TNOWAVE manual)

Notations used in Figure 4.3 can be explained as follow:

N : pile element number	$f_{n-1,i-1}$	:	downward traveling incident wave
n : element level	$-f_{n+1,i-1}$	:	upward traveling incident wave
Δl : element length	$f_{n,i}$	:	downward traveling transmitted wave
i : time step	$-f_{n,i}$	:	upward traveling transmitted wave
t : time			

At time t (time step i) waves are arriving at level n inside the pile from the former time step t - Δt and levels (n –1) and (n +1). Incident waves are calculated in the former time steps then at the intersection point of elements the transmitted waves (resulting waves) are calculated based on equilibrium and continuity conditions.

For all piles in soil, a complex interaction exists between the pile and the soil. Soil is a complicated material, with cohesion, friction, damping, elasticity, water pressures, and so on. In method of characteristics the pile-soil interaction is modeled by springs, dampers, and added masses. The general formula for the interaction force W between the pile and soil is given by :

$$W = W_u + W_v + W_a \tag{4.15}$$

The functions W_u, W_v, and W_a represent the interaction forces due to displacement, velocity, and acceleration of the pile. the pile-soil interaction forces are distributed over the entire contact area between pile and soil. Method of Characteristics interaction forces are assumed to be acting only at the element boundaries as shown in the figure below.

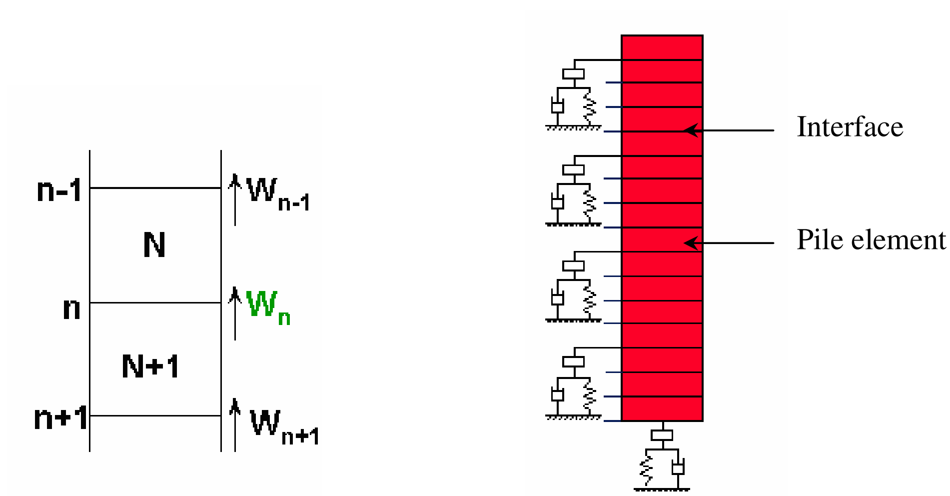


Figure 4.4 : Schematic demonstration of the pile-soil interaction forces

(TNOWAVE manual)

4.3.2 Numerical Method

Development of electronic computers makes it possible to solve the wave equation with the numerical integration method. Before that, in order to be able to solve the equation, real condition defining parameters had to be simplified and which decrease the accuracy of the solution technique.

In 1960, E. A. L. Smith presented the numerical integration method as a solution for the wave equation propagation in defining the pile driving problem. In his model, Smith divides the pile, hammer and other driving accessories such as the cap block, pile cap into discrete elements as a series of point masses and springs which is known as the Smith's model (Fig 4-5). Model also divides the total wave action time into small time intervals. Each small time interval represents the individual movement of the entire continuous pile driving motion. The smallest the time interval value leads to the more accurate modeling.

In Smith's model the time interval value is chosen as 1/4000 sec. For any defined time interval velocity, force and displacement values are assumed to be fixed and finally the velocities, forces and displacements for each interval is computed so as to differ from those existing in the preceding interval by just enough to represent the change occurring during one interval. The action of each weight and each spring is then calculated separately in each and every time interval and in this way probable stresses occurred during driving, pile penetration, permanent set per blow against any amount or kind of ground resistance can be determined mathematically (Smith,1960).

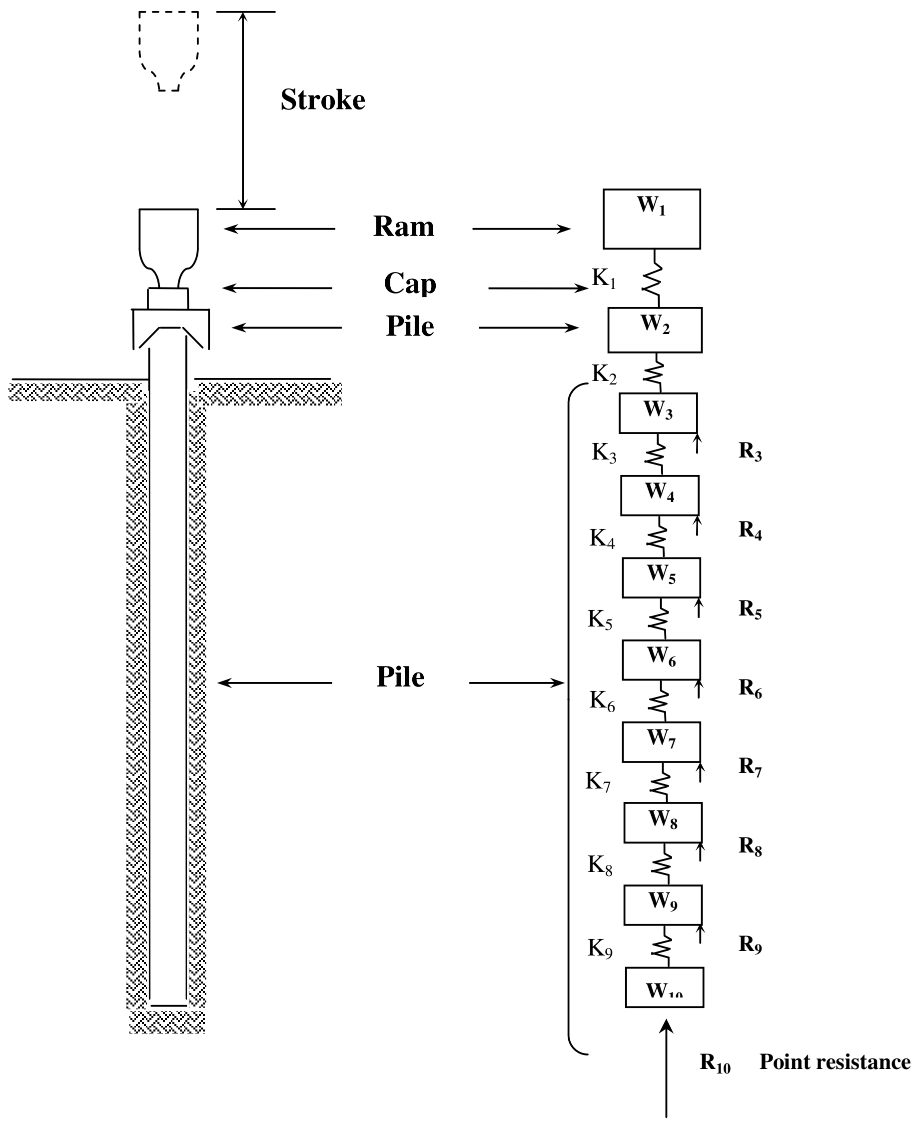


Figure 4.5 : Smith's model for numerical integration method for calculation of wave equation (Smith, 1960)

In Smith's model (Fig 4.5) the driving accessories such as the hammer ram, cap block, pile cap and other parts are represented with the individual point masses without elasticity due to their short and heavy structural properties. W_1 and W_2 point masses represent the ram and pile cap respectively. Cap block unit is relatively light when compared to other driving units therefore it can be represented by the spring K_1.

Different from the other elements pile has elasticity due to its length. Therefore, during driving pile system does the wave action with the propagating longitudinal waves as a result of the hammer blow. This wave action can be analyzed mathematically by dividing the pile into discrete unit elements. Weight of each divided element along the pile is represented by the point masses from W_3 to W_{10} and the associated elasticity of each discrete element is represented by an individual spring K_2 to K_9 in Figure 4.5.

While constructing the Smith's numerical integration method the choice of lengths of each individual discrete element and also the choice of time interval values is an important point of consideration. The smaller the unit lengths chosen, the smaller must be the time interval (Smith, 1960). In determination of the unit section lengths the wave length of the impact wave must be considered. The section lengths should not be less than the occurred impact wave length. Otherwise the pile motion would not catch with the stress wave in mathematical modeling. On the other hand, the time interval should not be unnecessarily small, because this would use an unnecessarily large amount of time in computation with little or no increase in accuracy (Smith, 1960).

In constructing a mathematical model in order to make a pile calculation depending on the wave equation, some boundary conditions must be defined to be able to solve the system. As mentioned throughout the whole section the complexities of the boundary conditions require an analysis technique different from the analytical method. The boundary conditions and the physical characteristics of the entire driving system must be analyzed in order to be able to understand the calculation method and the obtained results. Smith's model is based on the some particular way of behavior of the soil system depending on the soil mechanics principles. The interaction between the pile and soil results in two different external response. They are the resistance at pile tip and the shaft resistance occurred alongside the pile. Pile tip resistance is calculated to be able to take into consideration the dynamic soil parameters released during driving.

These parameters are:

- Quake, Q , the elastic ground compression
- Ultimate ground resistance, R_u
- Viscous damping based on a damping constant, J

These parameters can be explained by the help of Figure 4.6:

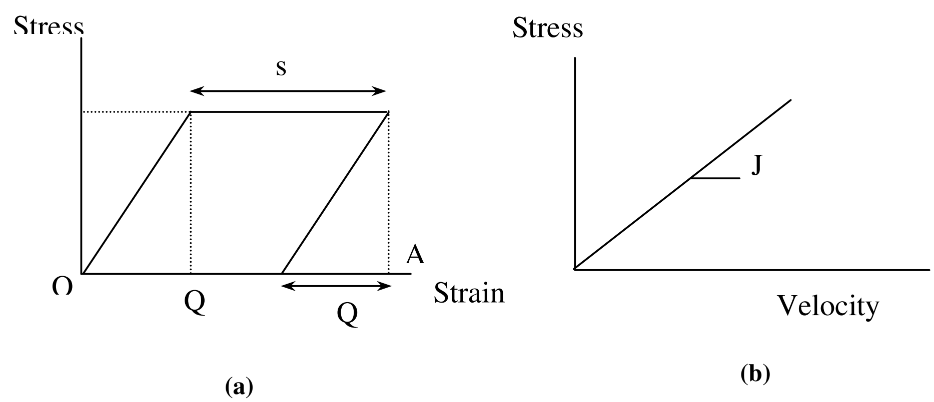


Figure 4.6 : Smith's model for the soil-pile tip interface (a) Spring model (b) Dash-pot model

With the first hammer blow starting from the zero point, pile tip displace up to a value of Q which is the quake value. As the pile tip is displaced the soil at the toe of the pile compresses elastically reaching up to ultimate ground resistance R_u. Up to this point soil behaves elastically beyond this point plastic failure occurs. When the pile tip reaches to a displacement value at point A elastic rebound occurs and driving is completed thus the permanent set of pile, s, can be determined.

Viscous damping parameter is used in order to consider the penetration time of pile tip related to velocity in order to consider the ground resistance. Thus it is obvious that the springs in Smith's model represent resistance to driving as a function of displacement and the dash-pots represent the resistance as a function of velocity (Fig 4-6).

Shaft resistance alongside the pile is calculated by using the damping factor $J^{'}$ instead of damping factor J which is used in the tip resistance calculation. As the pile is driven the displacement of the soil under the pile tip is relatively much than the soil alongside the pile. Thus the $J^{'}$ value should be smaller than the J value of damping factor which affects the viscous damping parameter. J' is applied for the shaft resistances represented with R_3-R_9 and J value is applied to the point resistance R_{10} illustrated in Figure 4.5.

In Figure 4.5 springs K_3 to K_9 can transmit tension, but springs K_1 and K_2 cannot because the ram, the pile cap and the pile itself are all separate pieces. In this aspect, the physical characteristics of the driving equipments must be considered in order to perform pile driving studies. Driving equipments briefly consist of the hammer, cap block, pile cap or follower. For precast concrete piles also pile cushion is needed to protect the concrete from breaking.

In conclusion, a numerical method permits calculation of pile driving action under any specified set of conditions, gives permanent set per blow as well as instantaneous stresses, displacements and velocities. It also makes it possible to check and analyze field test results and is used to determine the driving characteristics of various types of piles and hammers. Finally, the wave equation analysis is commonly used in drivability studies. Drivability prediction studies provides the selection of best driving equipments and procedure as well as with the correct determination of reinforcement required for a concrete pile in order to avoid any possible damages.

4.3.3 Brief Comparison Between Analysis Methods

The Smith's model for the numerical integration method and the method of characteristic model techniques differ from each other in modeling the pile-soil system and in analysis. These significant differences can be summarized as follow (Fig 4 .7):

Method of characteristics assumes the mass and stiffness of pile structure to be continuous. Different from that, Smith's model represents the pile body with concentrated point masses and with concentrated springs representing the stiffness.

In continuous model, initial wave propagation is calculated for each time step Δt and the resulting transmitted waves are calculated depending on the equilibrium and continuity conditions at the pile element intersection planes. On the other hand, in Smith's model a set of equation of dynamic equilibrium representing the wave action of each discrete point masses is solved for each and every time step, Δt.

Final difference common to either methods is the definition of shaft friction resistance occurred as a result of pile-soil interaction. In continuous pile system method (method of characteristics) friction forces are assumed to be acting at the element intersection but in discrete model they are acted in the centers of gravity of the point masses as shown in Figure 4.7.

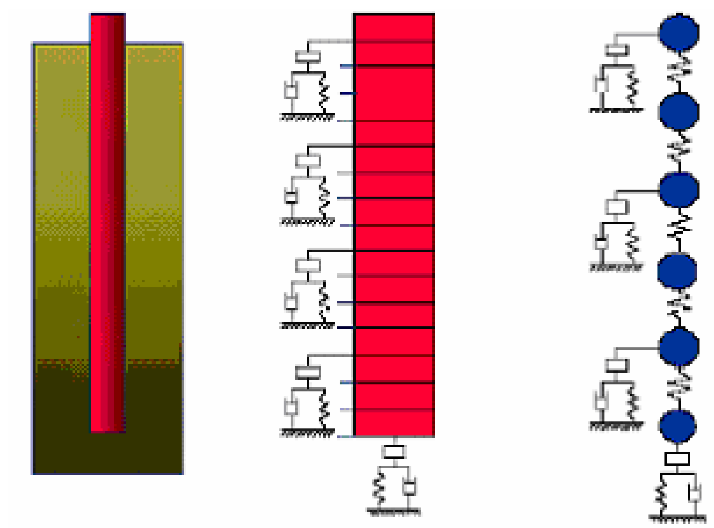
When for the Lumped Model the elements are too long the numerical integration method may cause inaccuracy in the results. Reflection may appear which are not real. When the elements are too long the resistances are concentrated at too few points.

Some Computer programs based on the continuous Model (Method of Characteristics) are:

- ADIG (Ifremer, France)
- CAPWAP-Co (GRL, USA)
- IHCWAVE (IHC, the Netherlands)
- KWAVE (Kanazawa University, Japan)
- PILEWAVE (HBG, the Netherlands)
- TNOWAVE (Profound, the Netherlands)

And Lumped Model based ones are:

- Dynpac (Heerema, The Netherlands)
- GRLWEAP (GRL, USA)
- WEAP (Texas A&m University, FHWA, USA)



(a) (b) (c)

Figure 4.7 Comparison of analysis methods (a) 3 dimensional continuous soil and pile, (b) 1 dimensional continuous pile discrete soil model, (c) 1 dimensional discrete pile discrete soil model (TNOWAVE Manual)

5. PILE DRIVING PREDICTION BY THE WAVE EQUATION

During the early design stages of a pile foundation construction the following points should be considered: the pile material and length, suitable construction methods and equipment, acceptable and tolerable service requirements depending on the performed loading tests and finally the construction quality. There are different types of methods and equipments for installation of piles. Many factors affect the choice but the result obtained always should be a structurally sound pile system capable of developing the design loads also supported by the soil or rock.

During the past few years the use of the wave equation to investigate the dynamic behavior of piling during driving has become more and more popular. Widespread interest in the method had been started in 1960 by E.A.L Smith who used a numerical solution to investigate the effects of driving accessories; such as ram weight, ram velocity, cushion and finally the pile properties; and the dynamic behavior of the soil during driving on drivability studies.

Drivability for a specific pile-soil system is the maximum soil resistance to which a driven or driven-and cast-in-place pile can be installed without damage. In this aspect it is clear that the developed soil resistance is a function of the pile dimensions and soil profile. There are two main topics defining the drivability. First one is the determination of pile type and properties for a given soil profile. Second one is the strength and stiffness of the pile. While the depth, pile and soil profile control the ultimate soil resistance and bearing capacity, on the other side, the pile stiffness, structural strength and driving system (hammer and driving accessories) control the depth to which a particular pile can be driven. Purpose of the wave equation analysis is providing all these considerations without giving any damage to pile during driving. Proper hammer selection considering the pile type and soil condition forms the basis of the wave equation analysis. A hammer that is too small may not be able

to drive the pile to the required capacity and can result in excessive number of hammer blows. On the other hand a hammer that is too large may damage the pile.
Proper hammer-cushion-pile-soil system can be constructed before the field application by using the wave equation analysis. In this manner possible damages or other problems that may occur can be prevented. This analysis technique is also named as the pile driving prediction studies. As it is mentioned before the pile driving equipments form one of the most important branches of the analysis. In this aspect they must be studied in detail by defining their properties before going through the analysis technique.

5.1 Pile Driving Equipments – Hammers and Driving Accessories

There are a variety of pile driving hammers with different working principles and with different manufacturers. Each has its own individual advantages and disadvantages. The contractor can select either type of hammers depending on the project needs, availability and economical choice.

Piles are driven by two different techniques as;

- Impact
- Vibration

Impact hammers install the driven piles by hitting with the use of a ram. Vibratory hammers do it by vibration. The most commonly used hammer types can be classified as follows:

- Drop hammers
- Diesel hammers
- Hydraulic hammers
- Air/Stem Hammers
- Vibratory Hammers

Driving equipment system completed with the additional driving accessories such as:

- Anvil
- Striker Plate
- Hammer cushion (Cap block)
- Helmet (Pile Cap)
- Pile Cushion

Driving accessories are placed between the hammer and the pile in order to transmit the impact force from the impact hammer to the pile. They prevent the pile and hammer individually from getting damaged with the effect of impact. Hammer and other driving accessories for impact hammers are shown schematically in Figure 5.1 in detail. Before defining the hammer types, the driving accessories should be mentioned which complete the entire driving system and also are included as hammer parts.

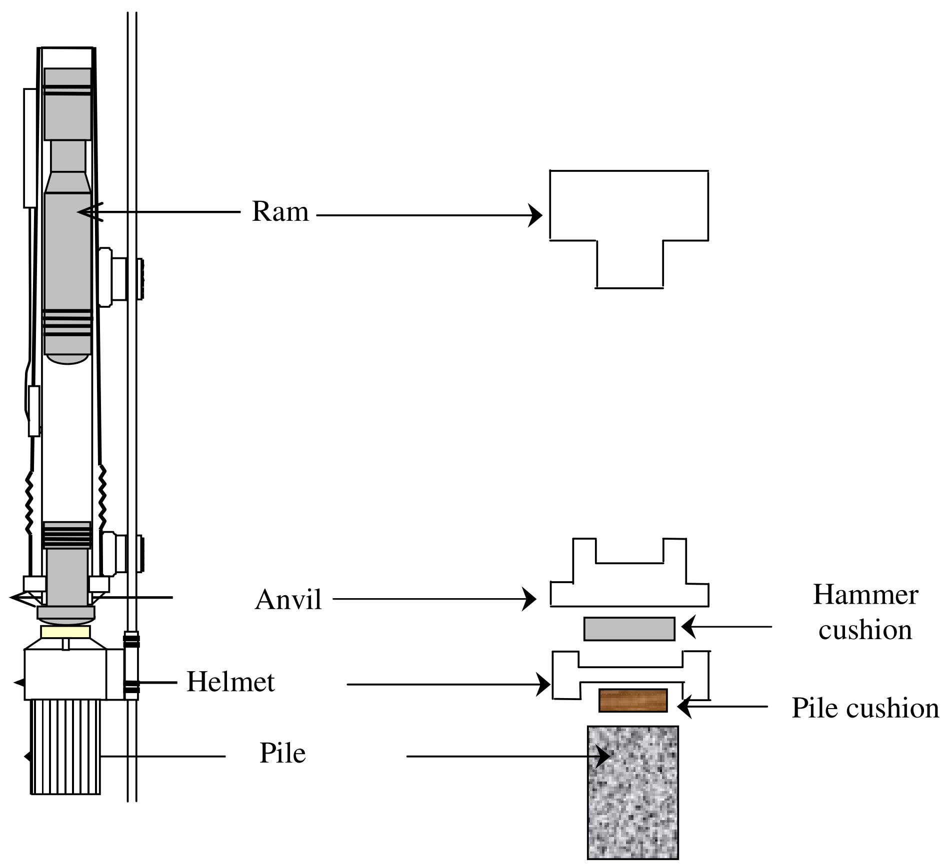


Figure 5.1 : Schematic cross-sectional view of hammer and driving accessories

5.1.1 Driving Accessories

- **Anvil**

They are used in the hammers in order to build the pressure by trapping the combustible mixture.

- **Helmet**

This unit is used in order to fit the hammer with the pile (Fig 5.2). Helmet distributes the blow uniformly from the hammer to the pile and as a result the damage caused to the pile is reduced. An appropriate helmet should fit loosely around the pile top to provide the control of pile in case when the pile tends to rotate during driving. However, the fit shouldn't be so loose in order to be able to provide alignment of the hammer and pile.

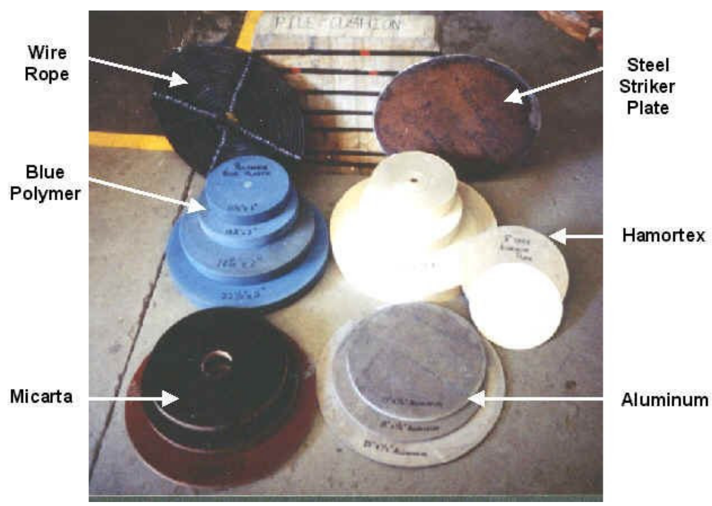
Figure 5.2 : Helmet

- **Hammer cushion**

While the helmet tends to protect the pile by distributing the blow, the hammer also requires protection from the shock wave reflected back to it. Besides the hammer cushion also serves to protect the helmet and the pile. In other words; it is an element stands for two different functions. Therefore, impact hammers have hammer cushion element between anvil and helmet which receives the impact of the hammer. In this way the striking units is protected from damage. It increases the blow efficiency. The actual hammer cushion and its configuration changes due to the hammer configuration and the material used for the cushion. Finally it is placed inside the helmet. Commonly used hammer cushion materials are hardwoods, plywoods, steel wire, laminated micarta and aluminum discs, and plastic laminated discs (Fig 5.3a).

- **Pile Cushion**

Pile cushion unit is generally used while driving concrete piles. It is placed between the pile top and the helmet. Pile cushions are not used on steel piles or timber piles. They are usually made of plywood. They are placed on top of the piles(Fig 5.3b).



(a)

(b)

Figure 5.3 : (a) Different hammer cushion materials, (b) Pile cushion (http://www.fhwa.dot.gov)

5.1.2 Hammer Types

1. **Drop Hammers**

The drop hammer is the simplest and the oldest type of impact hammer. It consists of a ram that is lifted to a specified height and released. Drop hammers can damage the pile head if driving stresses are not controlled by limiting the stroke distance and supplying a cushion material between anvil and ram. The drop hammer is a comparatively simple device which is easily maintained, portable, relatively light. It is mostly suitable for driving relatively small, lightweight timber, steel or aluminum piles.

2. **Air /Steam Hammers**

This type of hammer is divided into two types as single acting steam/air hammers and double-acting steam/air hammers.

- **Single –Acting Steam or Air Hammers**

Single acting steam or air hammers employ pressure from steam or compressed air in order to raise the ram, then automatically releases the pressure allowing the ram to fall freely and strike the drive cap. It has a blow rate of 40-60 blows per minute. When it is compared with the drop hammer type has shorter stroke distance, higher ram weight and operates at higher speeds. A hammer cushion usage within the drive cap mainly depends on the manufacturer's directions. Hammer efficiency can be controlled by observing the stroke height and blow rate.

- **Double – Acting Steam or Air Hammers**

These types of hammers consist of two valves as upper one and lower one. They are also employ pressure from steam or compressed air to raise the ram. Different from the single acting one, in the downstroke stage the steam or compressed air provided from the upper valve operate to supply additional energy to the ram. The compressed air is allowed through the inlet and to upper valve and finally into the steam/air cylinder. Additional pressure on the downstroke stage and the short stroke height results in a high operating rate of 90-150 blows per minute. It is 1.5 to 2 times the operating rates of the single acting hammers thus they deliver higher impact energies proportional to this ratio. Higher impact energy results in higher impact velocities which can cause pile head damages in piles of low compressive strength while on the other hand it is beneficial to the production. A hammer cushion material is not used between the ram and helmet. This hammer type can be used in any soil type.

3. Diesel Hammers

- **Open-End Diesel Hammers**

The "open ended" term represents the situation that the top of the hammer is open and the movement of ram as going up and coming down can be observed as it delivers the blow on pile. These are impact type of hammers and blow count measurements is the general method of inspection. Fuel is introduced into the cylinder, the ram drops due to the gravity setting off an explosion which moves the ram up and this process is repeated over and over. In Figure 5.4, the operation principle can be examined in more detail. Operation steps:

(a) While the ram drops it activates the fuel valve thus fuel is injected into the cylinder.

(b) As the ram closes the exhaust opening, pressure start increasing inside the chamber. As a result compression impact occurs. When the ram strikes the anvil the fuel is atomized and explodes thus forces the anvil down against the pile and the ram up automatically. At this instance the pile is driven. This supplies energy to the pile in addition to the induced by impact of the ram

(c) As the ram goes up, the exhaust opening is opened letting the exhaust gases out thus the pressure inside the cylinder turns to it normal position.

(d) As the ram keeps going up, the fresh air fills the chamber through the opening.

Open end diesel hammer efficiency depends on the pile resistance which means that harder the driving the greater the efficiency. Diesel hammers can be adjusted such that the fuel injected into the cylinder can be controlled and varied thus in that way the transmitted energy can also be controlled. An open end diesel hammer requires a hammer cushion between the anvil and the helmet. Operating rate is about 40-50blows per minute which is somehow slower than the single acting air/steam hammer. As the driving resistance increases, the stroke height increases but the operating rate decreases. They are best suitable for the medium to hard driving conditions. They do not tend to operate efficiently in soft soils because of the required driving resistance for compression.

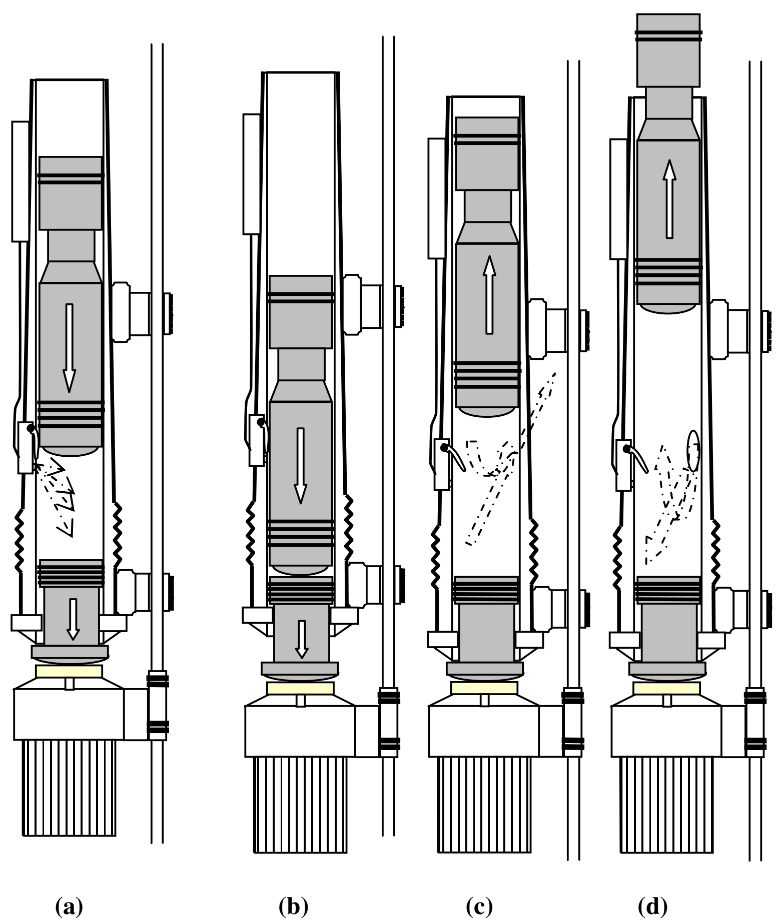


Figure 5.4 : Open end diesel hammer operation principal

- **Closed-End Diesel Hammers**

The closed-end diesel hammer, also known as the double-acting diesel hammer is similar to the open-end hammer, except that a closed top and air tank placed on the upper part of the cylinder. The stroke height is shortened when compared to the open-end hammer because of the air tank. This results in operating speeds of about 80 blows per minute. Some closed-end hammers are convertible to the single acting mode.

4. **Hydraulic Hammers**

The hammer energy on these can be controlled with precise pressure settings. In fact, rather than recording stroke height during driving, the Inspector records the pressure introduced on pressure gauges, on the hydraulic pump. The Inspector can also record stroke height by marking increments on the slide bar.

Like the air/steam hammers, these also require support equipment. A big drawback to these hammers is the need for a dedicated person to operate the hydraulic power unit and the need for experts when repairs are required. Hydraulic hammers have the following advantages such as controllable variable stroke, high efficiency blow, low impact velocity, light weight and finally the quite running.

5. **Vibratory Hammers**

Vibratory hammers operate via the rotating eccentric weights by electric or hydraulic motors producing vertical vibrations. It is important that a rigid connection be maintained between the hammer and the pile. Vibratory hammers are most efficient in driving non-displacement type of piles in sand. These hammers are not very effective in penetrating obstacles, large cobbles or stiff clays. They are not generally suitable for the installation of concrete and timber piles. When used for the right combination of pile and soil, they can install production piles at a rate much faster than any type of impact hammer.

5.2 Pile Driving Studies (Drivability Studies)

Effective design of constructed pile foundations requires pile driving studies. Drivability studies consist of evaluating different alternative pile types by considering the different installation techniques and equipments while also analyzing their effects on pile and soil capacities. As frequently mentioned in the scope of this study, the old practices of drivability studies are based on the use of empirical dynamic formulas, but in recent years pile driving industry start using of wave equation analysis as the means for a designer to evaluate pile drivability, hammer selection and penetration limits in pile driving studies. While the wave equation method provides superior analytical techniques, engineering experience and judgment are still very much of necessity. A review of pile installations for similar sites and structures can be extremely valuable in that aspect. Analytical predictions obtained with the drivability studies can be verified in the field by driving and static load tests as well as with the dynamic analyzer.

5.2.1 Wave Equation Analysis in Drivability Studies

Based on the previous sections, to sum up again; a wave equation analysis provide two main results for drivability analysis:

- Selecting appropriate sized driving equipment thus providing a pile to be driven to final grade without exceeding the allowable driving stresses.
- Determination of the penetration rate.

Analysis is based on a specific type and length of pile, and also a driving system for a defined soil profile. Accuracy and reliability of the analysis results extremely depends on the agreement of the assumed and the actual field parameter values. The results are applicable only to the assumed system and should only be used for the length of pile investigated. Hammer, drive cap, pile and soil resistance input parameters require an expert engineering judgment in order to obtain sensitive and accurate results. In the scope of this study, as it will be mentioned in detail in the preceding chapter, the wave equation computer program "TNOWAVE – PDPWAVE (Pile Driving Prediction)" is used in order to perform a case study depending on the prediction of pile drivability for given equipment and field conditions.

Wave equation based driving studies and the PDPWAVE software can be performed for different hammer selections. Then it is clear that the hammer selection can be considered to be the most important aspect of pile installation. Evaluation of hammer selection must consider the ability of driving the pile without structural damage or reducing the soil capacity, the ability to obtain penetration rates. Besides the size selection for a particular hammer must consider the pile's anticipated driving resistance, ultimate capacity, pile stresses expected during driving and pile set-up.

5.3 PDPWAVE (Pile Driving Prediction Software with Impact Hammers)

PDPWAVE is one of the options of the wave equation program TNOWAVE. The TNOWAVE program was originally developed by TNO Building and Construction Research in Netherlands in 1972 under the supervision of Mr. Middendorp. In 1999 the rights of the program have been transferred to the company Profound. TNOWAVE is a group of wave equation application programs which are designed to analyze the pile behavior in all aspect.

TNOWAVE applications are based on the one dimensional stress wave theory and its algorithm is based on the method of characteristics. It has the following application modules:

- PDPWAVE for impact and vibratory hammer drivability studies
- SITWAVE for the determination of local pile defects from sonic integrity testing signals by signal matching
- DLTWAVE for the determination of pile capacity from a dynamic load test by signal matching
- STNWAVE in order to simulate soil and pile behavior conducting a statnamic load test

PDPWAVE program is used in order to perform professional drivability studies. Performed analysis make it possible to optimize the selection of the impact hammer, increase the efficiency of the pile driving system with determination of an optimal combination of hammer, cushion, anvil, etc. to prevent any possible damage or soil refusal. In this way, precautions can be taken to avoid damage to the piles during driving as a result of unexpected compression or tensile stresses exceeding the pile material strength.

5.3.1 Input Data

Program requires the following data as an input in order to set up the driving system for a given field conditions.

- Hammer type
- Driving accessories (anvil, helmet, hammer cushion, pile cushion)
- Pile type and dimensions
- Soil profile

1. **Hammer data and driving accessories**

Hammer type and driving accessories are chosen from the hammer library provided within the program itself. TNOWAVE includes a number of types of pile driving hammers in a library containing all major hammer manufacturer brands. The parameters for the modeling of steam hammers, air hammers, diesel hammers, hydraulic hammers and vibratory hammers are available in the library with belonging cushions and anvils (Bielefeld, Middendorp, 1992). Hammer properties are loaded into program as an input automatically. In drive set- up window (Fig5.5), driving accessories properties are entered as well with the hammer type and properties. Setting up the hammer model is an important factor for the accuracy of the prediction results.

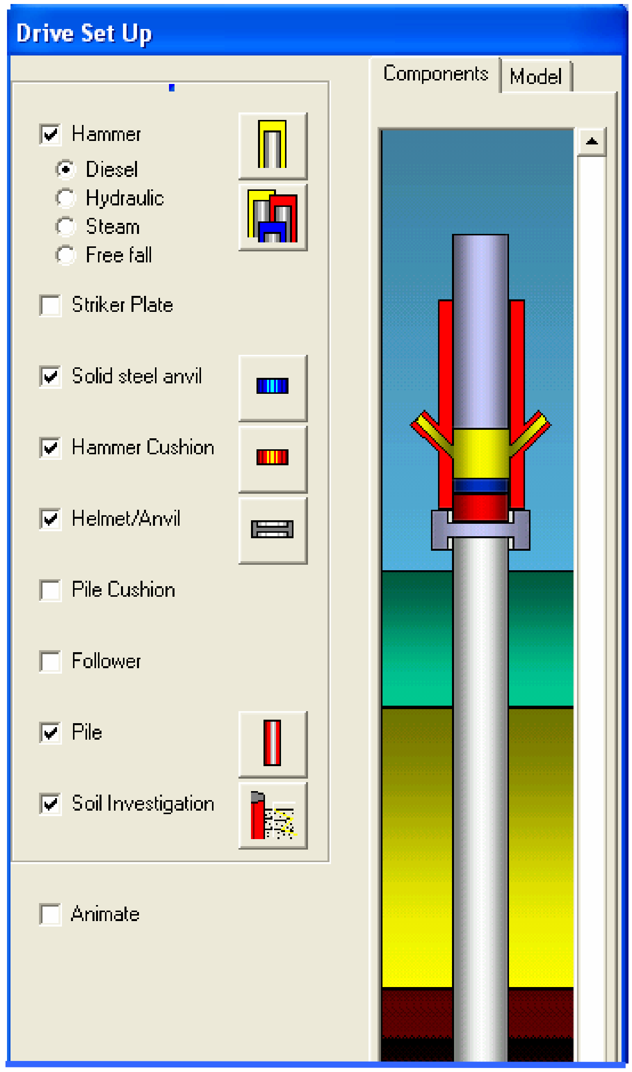


Figure 5.5 : Drive set-up window defining the driving equipment properties (PDPWAVE manual)

2. Pile data

In second step, the pile type, material and finally its dimensions are chosen. A pile can be modeled with a solid circular, solid square, open or closed ended open circular cross sectional area. Designed pile can either have same cross-sectional dimensions defined by one entire part or can have different cross-sectional properties having different parts. After selecting the pile material and cross section, the material properties are directly calculated by the program such as the elasticity modulus, density, total mass etc. (Fig. 5.6).

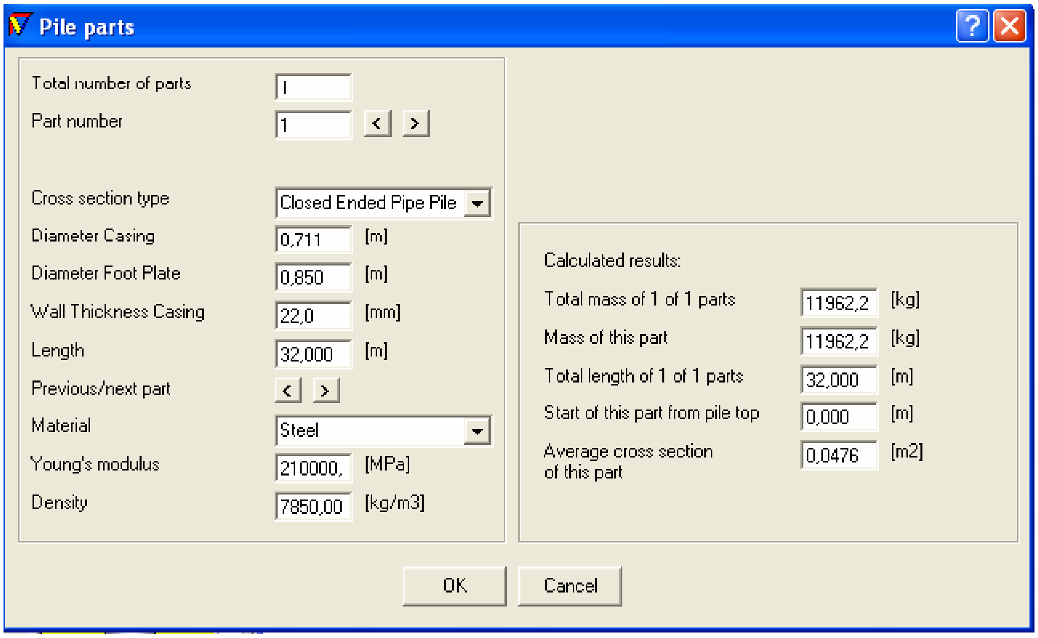


Figure 5.6 : Pile data input sheet screenshot (PDPWAVE manual)

3. Soil parameters

In final step, soil profile representing the actual field conditions must be modeled depending on the soil investigation results of; SPT, CPT, DMT, PMT, Cu and soil laboratory tests. Soil parameters are defined for each soil layer. Soil profile and soil type determination is the most important point affecting the accuracy of the analysis (Figure 5.7). Other than the soil investigation results defining the soil profile, dynamic soil parameters are also required in order to be able to model the dynamic behavior of the pile-soil system under loading conditions. Soil modeling parameters are:

- Yield Stress
- Quake value
- Yield factor
- Damping constant
- Soil fatigue factor

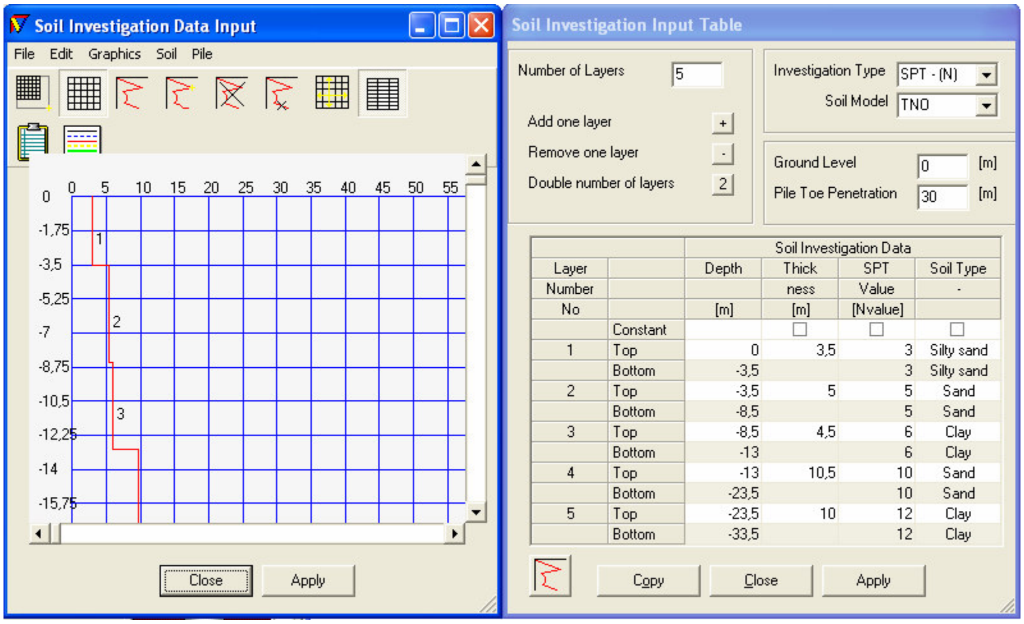


Figure 5.7 : Soil investigation data input (PDPWAVE manual)

Yield stress is the maximum plastic resistance force generated by an assumed spring model which is mentioned in previous chapters. The quake value determines the elastic range of the spring (displacement at which a spring behaves elastically). When the relative displacement between the soil and pile is larger than the quake value, the modeled spring behaves plastically. TNOWAVE software is based on the asymmetric elasto-plastic spring model. Thus both the loading quake value (Q_1) and the unloading quake value (Q_2) are required as an input (Fig.5.8).

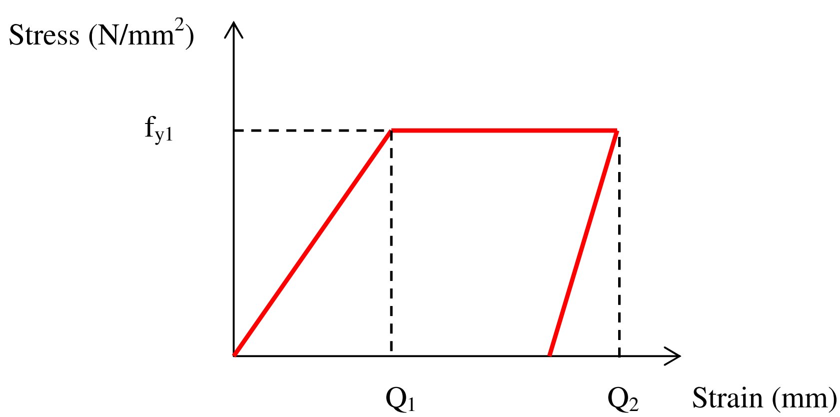


Figure 5.8 : PDPWAVE spring model

The yield stress for shaft friction is equal to 0.5% of the unit cone value or the unit local shaft friction. It must be noted that the unit shaft friction values can be much smaller than the measured unit shaft friction of a cone testing because of remolding effects during pile driving. When a redrive is conducted the values should agree. The yield stress values for toe resistance are much larger and equal the end bearing values from a cone testing. Yield stress values vary between 1-80 MPa for toe resistance and. Values to 0.3 MPa can be used for very dense sand layers. The yield stress value for soft clay is roughly 0.01MPa and 0.1MPa for stiff clay (PDPWAVE Helping manual).

Damping term represents the relation between damping force (W_v) of the soil and corresponding velocity of pile under dynamic loads. In the program the modeled TNO dash-pot system is assumed to behave in an exponential way (Fig 5.9).

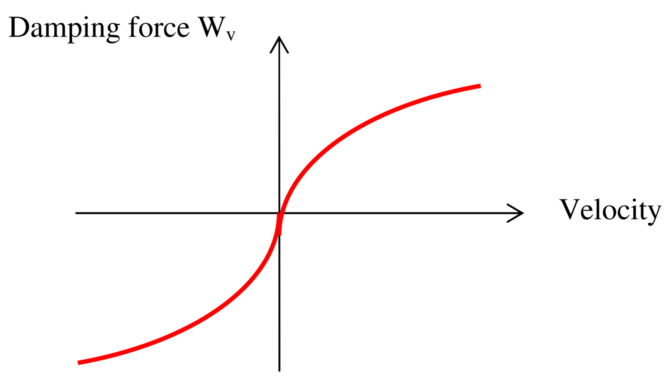


Figure 5.9 : PDPWAVE damper model

The damping force is defined as:

$$W_v = \begin{Bmatrix} C.|v|^N; v > 0 \\ -C.|v|^N; v < 0 \\ 0; v = 0 \end{Bmatrix} \tag{5.1}$$

Where;

W_v = the damping force [N/m^2]
C = damping constant [Ns/m^3]
v = velocity [m/s]
N = power alpha of the velocity

For N=1 the damping is linear

$$W_v = C.v \tag{5.2}$$

Since the late 1970s the exact form of damping of a soil for a wave equation analyses has been subject of much investigation and Heerema and Litkouhi and Poskitt have published findings indicating that for end-bearing in clays and sands and for kin friction in clays, damping is dependent on the fifth root of velocity thus the N value can be taken as 0.2in case no data is available (Tomlinson). N values are represented by several authors, varying between 0.17 and 0.32.

Program defines the damping constant, C as an input soil model parameter separately for shaft and toe thus it is expressed as a TNOWAVE parameter for shaft as;

$$C = \sqrt{G\rho} \tag{5.3}$$

where;

G = soil shear modulus (MPa)
ρ = soil density (kg/m^3)

and for toe as;

$$C = \frac{1.08\sqrt{G\rho}}{1-\mu} \tag{5.4}$$

Where;

μ= Poisson's ratio

In general, damping constants for sand are small when compared to clay and peat. Typical values vary between values from 1.06 to 10.6 Ns/m^3 for sands. Furthermore, damping is heavily influenced by the state of remolding of the surrounding soil. Damping at the toe is much larger ranging from 10.6 to 1060 Ns/m^3 though much less influenced by remolding effects. Damping factor for clay ranges from 10.6 to 106 Ns/m^3 (PDPWAVE Helping Manual). Soil property values typically used in wave equation analysis is given in the following brief table (Tomlinson).

Table 5.1 : Damping constants and quake values for different soil types

Soil type	**Damping constant**		**Quake side and end Q (mm)**
	Skin friction J' (Ns/m^3)	**End bearing J (Ns/m^3)**	
clay	0.65	0.01-1.0	2.5
Sand	0.15	0.33-0.65	2.5
silt	0.33-0.5	0.33-1.5	2.5

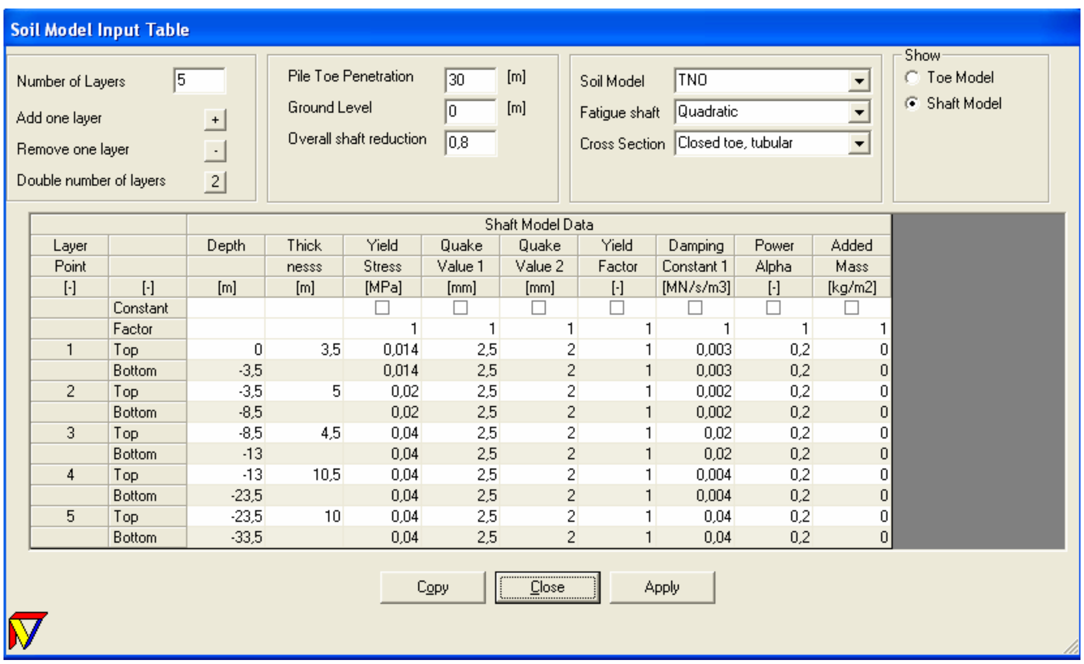
Finally the soil fatigue factor represents the soil strength reduction for most of soil types due to the cyclic loading during driving. Soil fatigue value for shaft and toe are modeled in different ways. The soil in the top layers will undergo more cycles than soil near the pile toe and suffer more soil fatigue. So the rate of soil strength reduction will be a function of the pile toe penetration depth. The soil fatigue factor implemented in TNOWAVE for the shaft assumes no fatigue in the first 5m from the pile toe and in strength to zero up to the ground level. The following reduction models are advised as an input depending on the pile length (PDPWAVE Helping Manual).

- For piles with a length less than 60m: quadratic reduction model
- For piles with longer than 60m: exponential reduction model
- In case of redriving case: linear reduction model

The soil at the toe will undergo some cycles before the pile penetrates and suffer soil fatigue. The following fatigue factors (reduction factors) are applied:

- For tubular open ended pile : 0.6
- For closed tubular pile : 0.8
- Precast pile: 0.8

Those mentioned basic soil modeling parameters are supplied to the program by the soil model input table (Fig.5.10)


Soil Model Input Table

Number of Layers 5
Add one layer +
Remove one layer -
Double number of layers 2

Pile Toe Penetration 30 [m]
Ground Level 0 [m]
Overall shaft reduction 0,8

Soil Model TNO
Fatigue shaft Quadratic
Cross Section Closed toe, tubular

Show
Toe Model
Shaft Model

		Shaft Model Data								
Layer Point [-]	[-]	Depth [m]	Thickness [m]	Yield Stress [MPa]	Quake Value 1 [mm]	Quake Value 2 [mm]	Yield Factor [-]	Damping Constant 1 [MN/s/m3]	Power Alpha [-]	Added Mass [kg/m2]
	Constant			☐	☐	☐	☐	☐	☐	☐
	Factor			1	1	1	1	1	1	1
1	Top	0	3,5	0,014	2,5	2	1	0,003	0,2	0
	Bottom	-3,5		0,014	2,5	2	1	0,003	0,2	0
2	Top	-3,5	5	0,02	2,5	2	1	0,002	0,2	0
	Bottom	-8,5		0,02	2,5	2	1	0,002	0,2	0
3	Top	-8,5	4,5	0,04	2,5	2	1	0,02	0,2	0
	Bottom	-13		0,04	2,5	2	1	0,02	0,2	0
4	Top	-13	10,5	0,04	2,5	2	1	0,004	0,2	0
	Bottom	-23,5		0,04	2,5	2	1	0,004	0,2	0
5	Top	-23,5	10	0,04	2,5	2	1	0,04	0,2	0
	Bottom	-33,5		0,04	2,5	2	1	0,04	0,2	0

Copy Close Apply

Figure 5.10 : Soil model input table datasheet (PDPWAVE manual)

Data can be supplied to soil model input table in two ways:

- **Indirectly:** by supplying soil investigation results first. Program will automatically translate the soil investigation results to basic soil model parameters and present the values in the soil model input table.
- **Indirectly:** by typing values for the basic soil parameter or changing the values supplied by the soil investigation results. Experienced users can use this option to adjust parameters according to their practical experiences.

In the mentioned indirect method the given soil investigation test results such as the SPT or CPT data is converted to TNOWAVE model parameters from the energy corrected, N-60 value and from the cone resistance, q_c value respectively. In the following tables soil type dependent parameters and correlations are given separately for shaft and toe as well as for SPT and CPT data used by TNOWAVE while running the application.

Table 5.2 : Soil model parameters and properties

			SPT correlations		**CPT correlations**	
Soil Type	**Density (kg/m^3)**	**Poisson ratio**	**Yield stress (MPa)**		**Yield stress (MPa)**	
			Shaft	**Toe**	**Shaft**	**Toe**
Peat	1100	0.30	$0.0150N_{60}$	$0.15\ N_{60}$	$0.100q_c$	q_c
Clay	1500	0.45	$0.0060\ N_{60}$	$0.20\ N_{60}$	$0.030q_c$	q_c
Silt	1800	0.30	$0.0063\ N_{60}$	$0.25\ N_{60}$	$0.025q_c$	q_c
Loam	1800	0.30	$0.0044\ N_{60}$	$0.20\ N_{60}$	$0.022q_c$	q_c
Silty sand	2000	0.30	$0.0042\ N_{60}$	$0.30\ N_{60}$	$0.014q_c$	q_c
Sand	2000	0.30	$0.0032\ N_{60}$	$0.40\ N_{60}$	$0.008q_c$	q_c
Gravel	2200	0.30	$0.0030\ N_{60}$	$0.60\ N_{60}$	$0.005q_c$	q_c
Rock	2500	0.30	$0.0006\ N_{60}$	$0.006\ N_{60}$	$0.001q_c$	q_c

5.3.2 Output Data

As a result of a TNOWAVE prediction, the performance of the hammer, efficiency and impact velocity, the performance of the pile, compression and tensile stresses, and the performance of the soil, static and driving resistances can be analyzed (Bielefeld, Middendorp, 1994). Resulting plots are obtained in three different categories as:

Function of time:

- Impact diagram
- Force and velocity time plots
- Downward and upward traveling waves
- Transferred energy

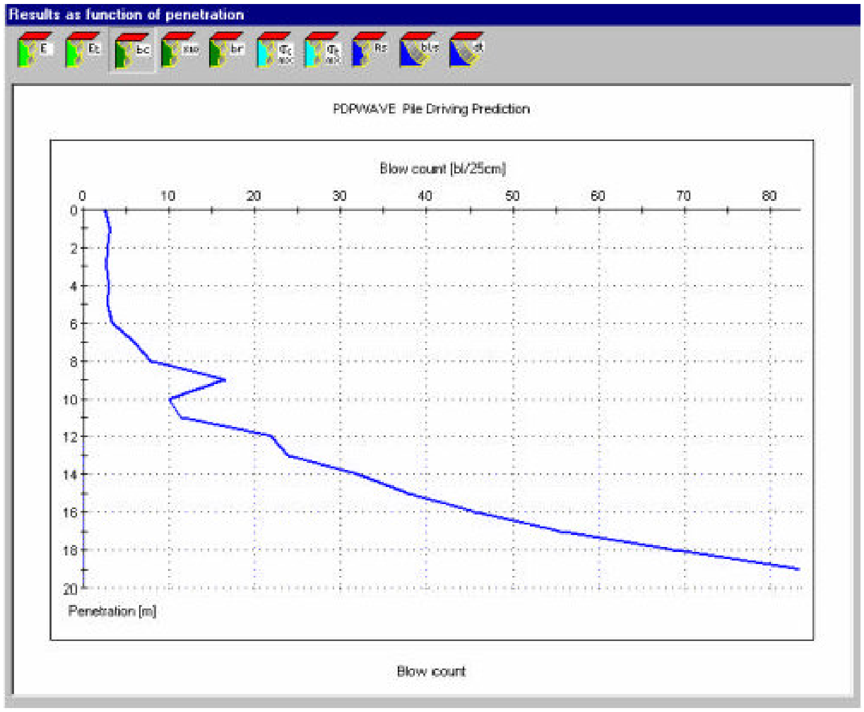
Function of penetration[†]:

- Impact energy of the hammer
- Energy in the pile
- Blow count, blow rate (Figure 5.11)
- Maximum stresses in the pile (compression and tension)
- Driving resistance
- Static resistance

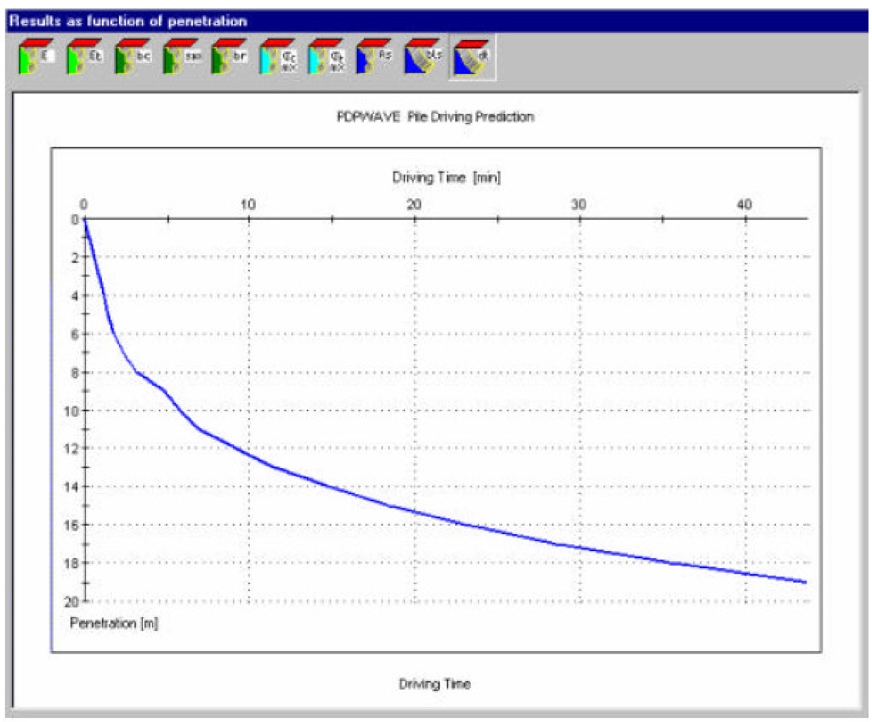
Function of pile axis:

- Forces in the pile
- Displacements
- Maximum stresses
- Shaft friction
- Velocities
- accelerations

[†] Plotted in the scope of the case study



(a)



(b)

Figure 5.11 : (a) Blow count graph and total driving time versus penetration (Function of penetration category)

6. USE OF PDPWAVE FOR DRIVEABILITY ANALYSIS – A CASE STUDY

Computer based numerical pile driving prediction studies based on the wave equation theory has been discussed in detail in previous sections. In order to improve the accuracy of the prediction results, the predicted results must be verified by field measurements. In this way the reliability of prediction and corresponding parameters used for constructing the simulation model can be tested.

6.1 Objectives

In the scope of this study, pile driving prediction analysis is performed by using PDPWAVE software based on the real field condition input data. Obtain prediction results are correlated with the actual results obtained in the field application. In this prediction analysis for a specific type of hammer, pile, soil configuration system the drivability of the given pile is analyzed. This analysis is performed depending on the variation of dynamic driving resistance which is the blow count per 0.25m against penetration. In this manner the prediction of blow count, evaluation of the maximum compression and tensile stresses as well as with the expected driving resistances makes it possible to decide the hammer performance related to its efficiency not to cause any damage to pile during driving.

In this section performed case study using PDPWAVE will be discussed in detail. Hammer specifications, pile properties and soil condition will be mentioned since they are the input data for running the program. Finally the obtained prediction results will be reported as well as with the actual field data and comparison will be carried out to verify the accuracy of both the field application and the program.

6.2 Soil Investigation

Test side is located in İzmit- Kocaeli Bay. Its plan view is shown in Figure 6.1. In scope of soil investigation 16 marine borings with changing depths varying between 30m to 45m and 4 land borings had been performed in the embankment site in order to obtain disturbed and undisturbed soil samples for laboratory tests.

Pile driving prediction studies are done based on the land boring results. Piles driven in this zone is analyzed. Land boring zone (zone A) is marked on the plan view in Figure 6.1. Zone A consists of the following boreholes with the corresponding depths:

Borehole Number	Depth (m)
S101	30.45
S102	30.45
S103	30.45
S104	33.45

- **Soil profile**

All these four boreholes provide a common soil profile characteristic of the studied zone. Depending on the borehole data fill material is observed in all boreholes down to a depth of 2.5-3.2m from the ground surface level. Material shows a heterogeneous structure due to varying particle size range (clay to gravel). Recently deposited marine sediment formation follows up the fill material with 9.5m to 18m changing thicknesses (Appendix A). Groundwater table level is located at 0,5m in depth that is so close to ground surface level. This shows that the soil is fully saturated.

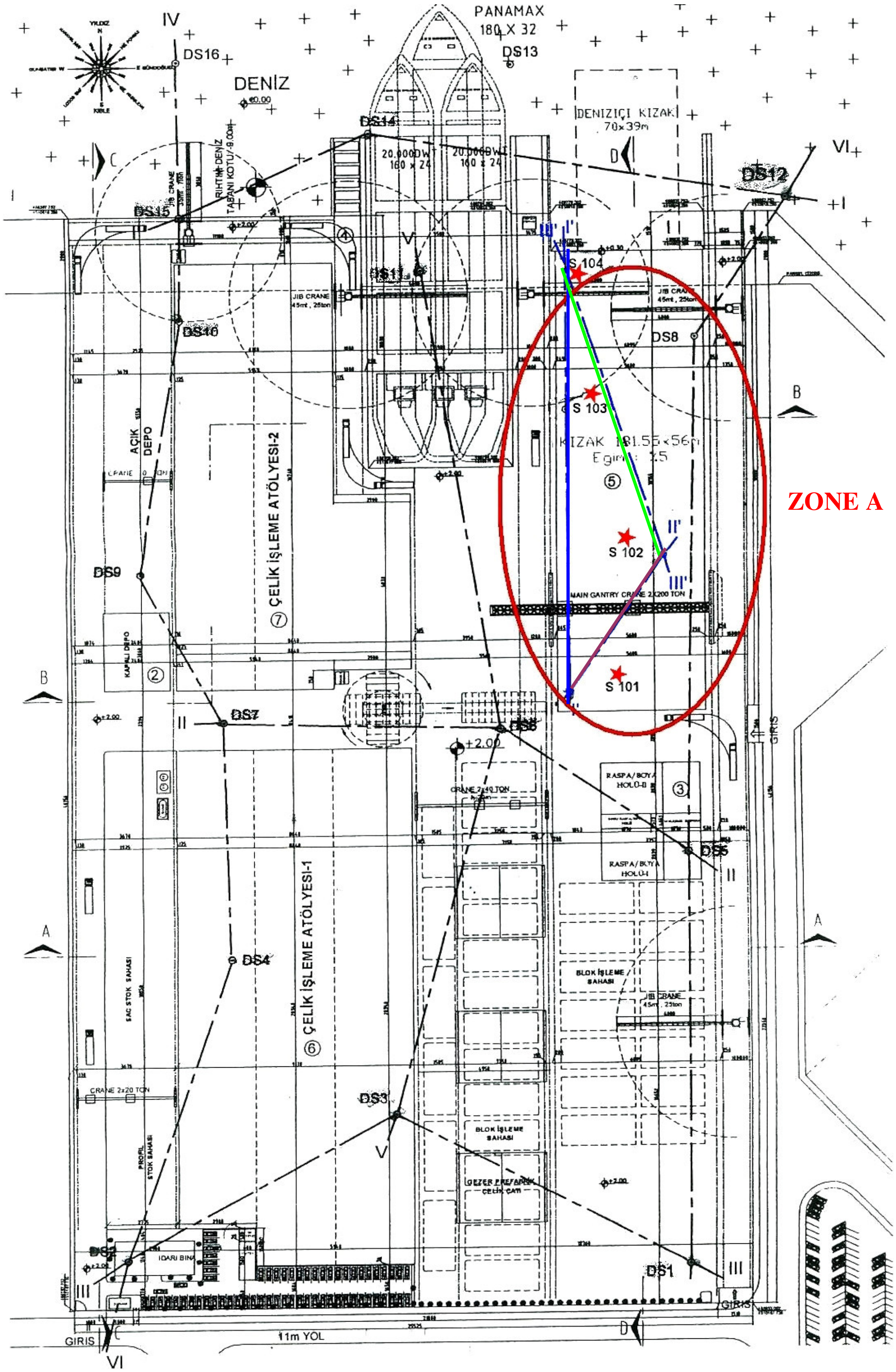


Figure 6.1 : Plan view of test site with soil test boring locations

Those marine sediments can be given consecutively with the corresponding approximate standard penetration test (SPT) results:

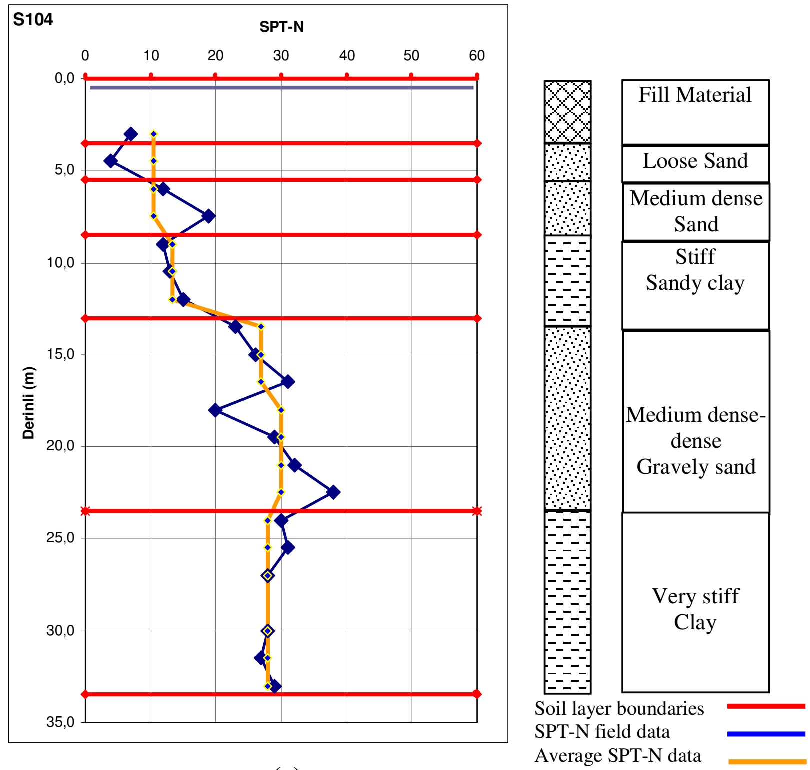
- Loose state, gravely sand — N_{30} = 3-9
- Medium dense, silty sand/gravely sand — N_{30}= 12-30
- Soft, clay / silt — N_{30}= 4-5
- Firm, clay — N_{30}= 4-9
- Stiff, sandy clay — N_{30}= 12-15

More stiff and consolidated sediments follow the recently deposited marine sediments. They can be given as follows again with the corresponding approximate standard penetration test (SPT) results:

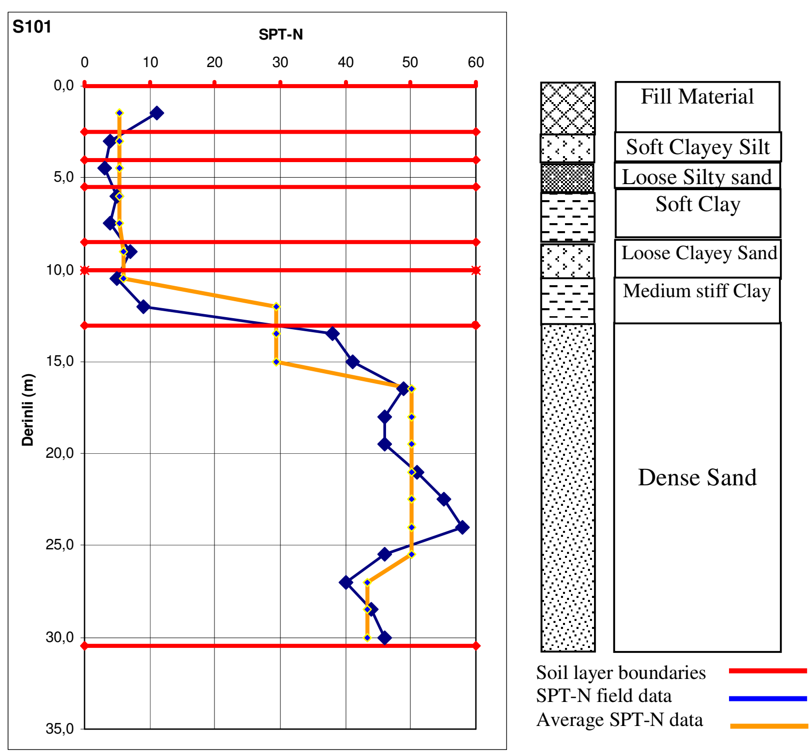
- Medium dense / dense gravely sand — N_{30}= 20-38
- Dense sand (with low gravel percent) — N_{30}=42-over
- Very hard clay — N_{30}=27-30

Borings had been ended within these layers. Borehole loggings and geological cross-sections along the profiles in Zone A are given in Appendix A and Appendix B respectively.

SPT results of the S101 and S104 boreholes mainly show two different characteristic of the zone separately. While in S101 pile is being driven to sand layer, in S104 it is being driven into clay layer. SPT results versus depth graph for S101 and S104 boreholes are given in Figure 6.2. Consideration and interpretation of the borehole data are discussed in order to obtain an idealized soil profile which accurately defines the studied area. Idealized soil profile is obtained during driving prediction analysis by try and error technique not getting far from the real soil condition.



(a)



(b)

Figure 6.2 : Soil profile SPT-N versus depth (a) Borehole S104 (b) Borehole S101

- **Corrected soil profile**

Standard penetration test raw results are seriously affected by several factors during driving. Those factors contribute to the variation of the standard penetration number N at a given depth for similar soil profiles. Among these factors are the SPT hammer efficiency, borehole diameter sampling method, and rod length factor. On the basis of field observations, it appears reasonable to standardize the field penetration number as a function of the input driving energy. Measured penetration numbers in the field are corrected for hammer efficiency, borehole diameter, sampler and rod length as follow:

$$N_{60} = \frac{N\eta_H\eta_B\eta_S\eta_R}{60} \qquad \textbf{(6.1)}$$

Where;

N_{60} = standard penetration number, corrected for field conditions

N = measured penetration number in the field

η_H = hammer efficiency (%)

η_B = correction for borehole diameter

η_R = correction for rod length

η_S = sampler correction

Variations of η_H; ηB; $\eta_{S;}$ η_R based on recommendations given by Seed et al. and Skempton in the years 1985 and 1986 respectively (Das; 2004). Actual raw field borehole data given in Figure 6.2 as a soil profile is considered to be erroneous since the SPT N values are comparatively high for saturated recently deposited marine sediments. When compared to the borehole S101 data, S104 soil profile presents more reasonable SPT values. Thus; idealized soil profile is constructed based on S104 profile.

Given borehole data are corrected depending on these correction factors. Borehole diameter, rod length and sampling method correction factors are taken as equal to one and hammer efficiency correction factor is taken as 0,75 depending on the literature knowledge (Das, 2004).

Another additional correction factor is valid for saturated sandy soils where SPT-N value is bigger than 15. This is known as the groundwater table correction. The relation is given by the formula in Eq.6.2. All mentioned corrections are summarized in Table 6.1 for S104 borehole data.

$$N = \frac{N-15}{2} + 15 \quad \textbf{(6.2)}$$

Table 6.1 : SPT-N correction table for S104 borehole data

Depth(m)	SPT-N Field	N'	ηH	N_{60}	AverageN_{60}	Soil Type
3	7	7	0,75	5	5	Fill Material
4,5	4	4	0,75	3		Loose Sand
6	12	12	0,75	9		
7,5	19	12	0,75	9	7	
9	12	12	0,75	9		Medium Stiff Clay
10,5	13	13	0,75	10		
12	15	15	0,75	11	10	
13,5	23	19	0,75	14		Medium dense Sand
15	26	20	0,75	15		
16,5	31	23	0,75	17		
18	20	17	0,75	13		
19,5	29	22	0,75	17		
21	32	23	0,75	17		
22,5	38	26	0,75	19	16	
24	30	30	0,75	22		Stiff Clay
25,5	31	31	0,75	23		
27	28	28	0,75	21		
30	28	28	0,75	21		
31,5	27	27	0,75	21		
33	29	29	0,75	21	20	

Thus the soil profile turns out be as follows in Figure 6.3. In the performed driving analysis the given SPT N_{60} values are used. Details concerning the idealized soil profile are discussed in the field observation and calculation section.

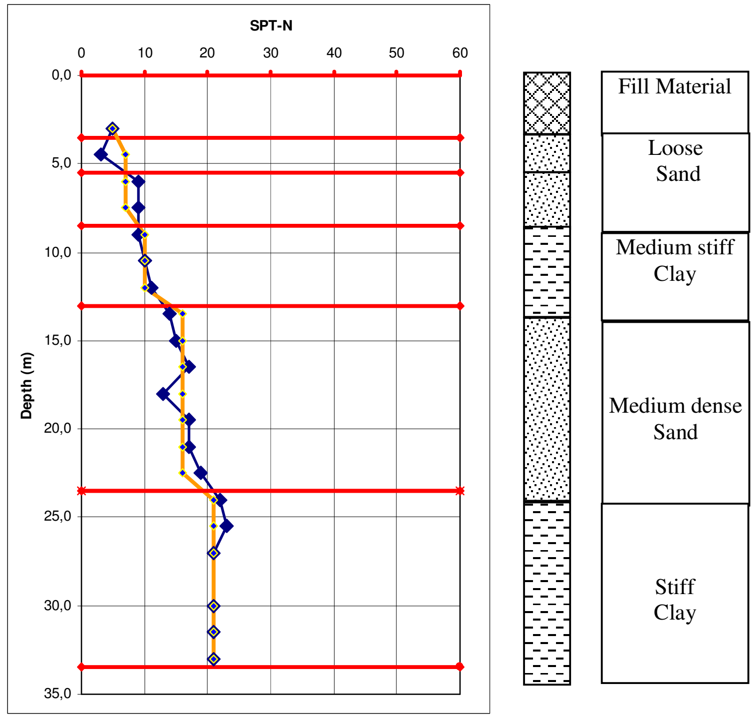


Figure 6.3 : Corrected SPT N_{60} soil profile for drivability analysis

6.3 Pile Data and Installation

In the actual project constructed pile type is vibrex pile. As it has been mentioned before vibrex pile is a type of driven and cast-in-place pile. It uses a steel casing driven into ground to form a precast concrete unit. After the installation of concrete, steel casing is withdrawn. In this aspect, for the drivability studies steel casing will be used. Pile is set on to a steel plug in order not to damage pile tip during driving thus forming a close ended pipe pile. Pile installation stages are observed in the field and presented in Figure 6.4a through Figure 6.4d. Piles' cross sectional dimensions and properties are given as follows

Table 6.2 : Pile material and dimensional properties

Pile Specifications	
Cross section	Close ended pipe pile
Material	Steel
Casing outer diameter(mm)	711
Steel plug diameter (mm)	850
Thickness (mm)	22
Total length (m)	32

The same steel casing is used in all pile driving process. Thus the pile material properties do not change during the pile driving prediction analysis. There are 60 pile driving records obtained in the filed observations. Thirty of which are driven with the diesel hammer and the rest are driven with a hydraulic hammer. These field records can be analyzed in two ways correlated within each others. They are both used in order to be able to construct the idealized soil profile and to compare and verify the accuracy of the analysis results obtained by using PDPWAVE.

(a)
Pile is placed on the steel plug

(b)
Installation of steel casing

(c)

Installation of the reinforcing cage

(d)

Casing is withdrawn

Figure 6.4 : Installation steps of a vibrex pile – field observation

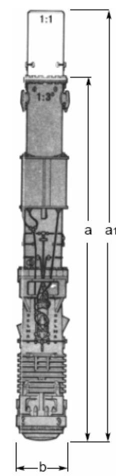
6.4 Hammer Data

Piles are equipped with two different types of hammers. These are an open-end single acting Delmag D46-32 diesel hammer and Junttan HHK 9A hydraulic hammer. Analysis is performed for both types of hammers for both different type of soil profile characterizing the field condition. Hammer specifications and configuration details are given separately for each hammer type.

- **DELMAG D46-32 Diesel Hammer**

Table 6.3 : Delmag D46-32 Diesel Hammer Specifications

Total ram mass (kg)	4600
Blow rate (bl/min)	37/53
Hammer mass (kg)	9000
Weight of ram (kg)	4600
Outer diameter of impact block (mm)	660
	5285
Length over cylinder extension –a_1(mm)	6285



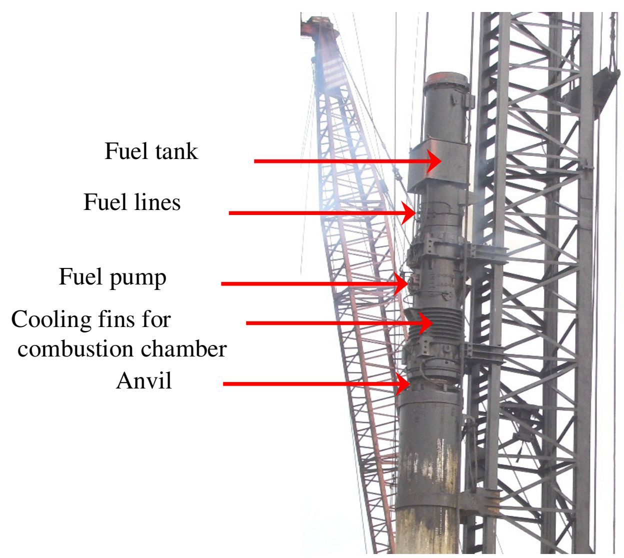
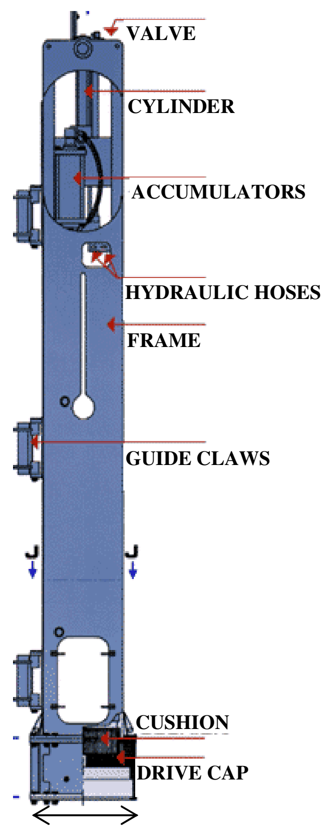


Figure 6.5 : Delmag D46-32 Diesel Hammer configuration

- **JUNTTAN HHK 9A Hydraulic Hammer**

Junttan HHK 9A hydraulic hammer is also used on site beside the diesel hammer. Junttan hammers are suitable for driving steel tube pipes, sheet piles, precast concrete and timber piles. The stroke and blow rates are adjustable to optimize the pile driving performance in all conditions. The ram weight can also be changed according to pile type requirements. All this means that one hammer serves efficiently a wide range of different pile driving operations. The hammers can be mounted on all kinds of leaders, or can be freely suspended. Junttan hydraulic hammers are friendlier to the environment and more practical than conventional diesel and air hammers, as they generate less noise, vibration and emissions. Hammer configuration and specifications are given in Figure 6.6 and in Table 6.4 respectively.



b

Figure 6.6 : Junttan HHK 9A Hydraulic Hammer Configuration (www.junttan.fi)

Table 6.4 : Junttan HHK 9A Hydraulic Hammer Specification

Maximum Energy (kgm)	106
Blow rate (bl/min)	40/100
Hammer mass (kg)	13400
Weight of ram (kg)	9000
Drop height (mm)	50-1200
Outer diameter of impact block –b (mm)	940
Overall length (mm)	7010
Drive Cap Types	A-type for metal tubes B-type for concrete piles

6.5 Field Data and Calculations

Field data obtained during pile driving consists of the records of number of blow counts corresponding to each 0.25m of penetration of the pile toe. Vibrex piles had been marked for each 0.25m to be able to log the blow counts. Also the blow rate (bl/min) and the total driving time were observed in order to analyze the efficiency of preferred type of hammer and driving performance.

Before analyzing the pile drivability with a computer based study, the field requirements must be determined such as bearing capacity and refusal criteria. Pile load tests show that the designed piles work for an allowable bearing capacity, Q_{all} of 200tons.When the negative shaft friction is activated the added 100 tons of load is applied to the pile. Then finally the Q_{all} becomes equal to 300tons. Corresponding set value (net penetration of pile toe for an individual hammer blow) and the refusal criterion value is obtained using the Dutch Formula given in Eq3.16. From the dynamic formula set value becomes equal to:

$$s = \frac{WE}{Fs(Q_{all}(W_p + W_r)} \qquad \textbf{(6.3)}$$

Where the calculation is performed by the field engineers based on the following values:

W_r = 4600kg; weight of ram

g = 399 kg/m; weight of pile per unit length

W_p = 10773kg; weight of pile (casing length is taken as L =27 m)

Q_{all} = 300tons; allowable bearing capacity

Fs =2.5; safety factor

E = 14600kgm; maximum hammer energy

Then set per blow is found out to be equal to:

s = 0.0058m/blow = 5.8mm/blow

From the calculated set value refusal criteria is obtained for Delmag D46 hammer as:

Refusal = 250mm / 5.8mm = 43 blows / 25cm

Refusal criterion for the Junttan HHK 9A Hydraulic hammer can be calculated in the same way considering its ram weight and maximum hammer energy. Thus it is obtained as 30-35mm/25cm.

Piles reaching up to this refusal value are accepted as satisfying the requirements. However, in case when the refusal is not occurred until the depth of penetration of pile toe to 32m then this depth is accepted.

PDPWAVE pile driving prediction program algorithm is quite complex. As mentioned before the program converts the soil investigation test results directly into the soil dynamic parameters. In theoretical aspect for this conversion the software utilizes the correlation factors (Eq. 6.4) that had been given in Table 5.2 but for the studied site it is experienced that these factors do not give appropriate results. Thus; the algorithm must be calibrated with respect to the specific soil properties and driving conditions.

$$\text{SPT } N_{60} \ * K_{toe} = \text{Yield stress}_{toe}$$

$$\text{SPT } N_{60} \ * K_{shaft} = \text{Yield stress}_{shaft} \qquad \textbf{(6.4)}$$

The correlations between the soil investigation test and the yield stress are represented in Eq. 6.4. In PDPWAVE software these correlations are not given to the user so different solving techniques must be applied in order to be able to build up the accurate relation for the investigated soil profile. Thus; in the scope of this study several approaches are applied to calibrate the software outputs reference to the field records.

The major concepts for the pile driving prediction analysis have been highlighted as follow:

1. The analysis have been performed by using the corrected soil profile not changing any of the variables affecting the result of calculation such as the dynamic soil parameters and remolding affect of the soil.

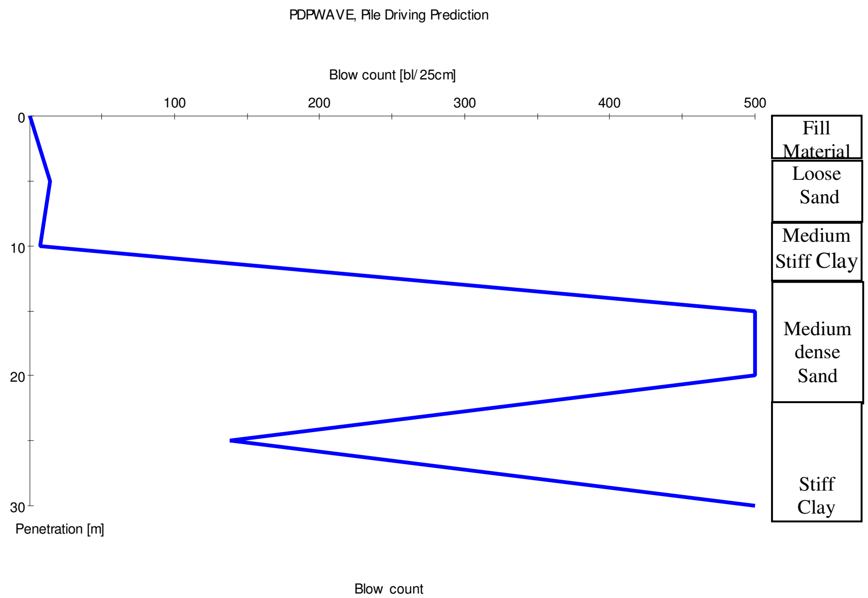
2. The analysis have been performed by using the corrected soil profile but this time the dynamic soil parameters have been also modified in order to approach the results of the analysis to the given field results.

3. In the third and the final step a kind of inverse analysis technique is performed to construct the idealized soil profile which matches with the field driving records.

All three steps are explained in the corresponding graphs of blow count per penetration and total blow count individually for the defined Delmag D-46 diesel hammer. Analysis results are compared with the actual field records to discuss both the accuracy of the PDPWAVE predictions and to form the accurate soil profile. Finally the analysis is repeated for Junttan hydraulic hammer to verify the prediction and soil model results with different driving equipment.

6.5.1 Solution depending on the SPT N_{60} values

Standard penetration test (SPT) results as N – values are corrected to obtain N_{60} values as mentioned in section 6.2. The developed soil profile is given in Figure 6.3. These values are used as an input in the PDPWAVE and the corresponding dynamic soil modeling parameters which are generated by the software are not changed. The obtained prediction results for Delmag D-46 are given as follow in Figure 6.7.



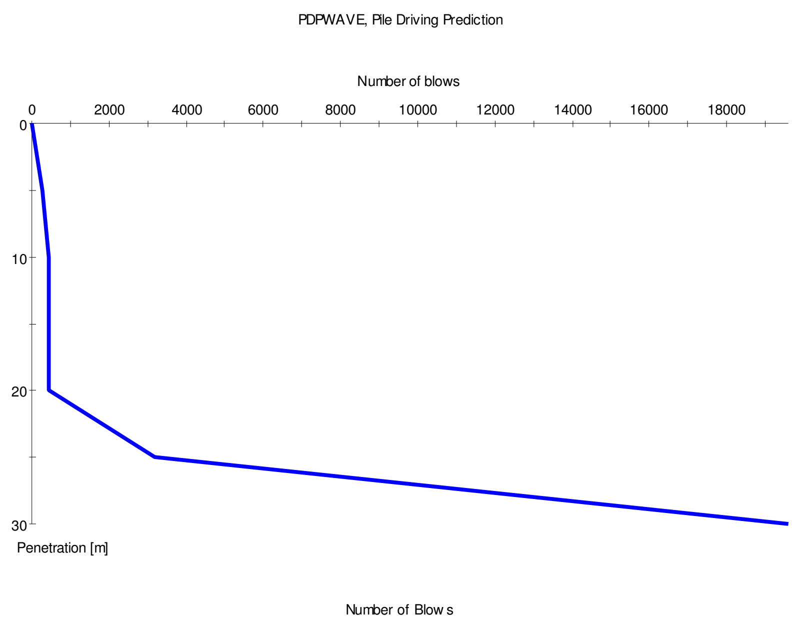


Figure 6.7 : Blowcount/25cm and total blow count graphs depending on N_{60} values

As it is observed in Fig. 6.7 in medium dense sand layer the blow count per 25cm reaches up to 500 which is impossible in practice. This result also does not suit with the actual field results. This shows that the assumed correlation factors (Table 5.2) must be adopted to the given soil profiles for the accurate solution.

6.5.2 Solution with the Adjusted Soil Modeling Parameters

In order to calibrate the system first the SPT N_{60} values are kept constant and the corresponding yield stress values are changed for each layer step by step by a definite fraction factor. Finally the driving graphs in Figure 6.7 turn into the form as in Figure 6.8.

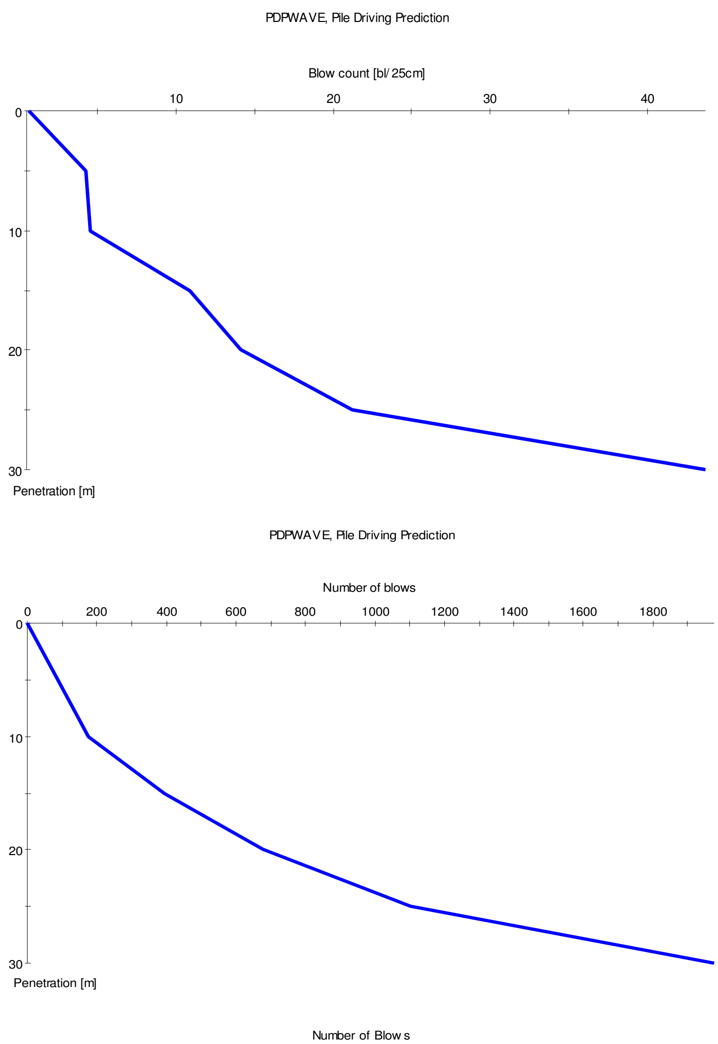


Figure 6.8 : Blowcount/25cm and total blow count graphs depending on soil modeling parameters

The selected yield stress values, damping constants and quake values are given in Table 6.5 and 6.6 for pile shaft and pile tip separately. Overall reduction factor considering the remolding affect of the soil during driving is taken as 0.8 which is adequate for close ended pipe piles.

Table 6.5 : Shaft friction model parameters

Shaft friction model parameters									
Layer Point		**Depth**	**Thickness**	**Yield Stress**	**Quake Value 1**	**Quake Value 2**	**Yield Factor**	**Damping Constant 1**	**Power Alpha**
[-]	[-]	[m]	[m]	[MPa]	[mm]	[mm]	[-]	[MN/s/m3]	[-]
1	Top	0	3,5	0,011	2,5	2	1	0,003	0,2
	Bottom	-3,5		0,011	2,5	2	1	0,003	0,2
2	Top	-3,5	5	0,016	2,5	2	1	0,003	0,2
	Bottom	-8,5		0,016	2,5	2	1	0,002	0,2
3	Top	-8,5	4,5	0,032	2,5	2	1	0,002	0,2
	Bottom	-13		0,032	2,5	2	1	0,002	0,2
4	Top	-13	10,5	0,032	2,5	2	1	0,002	0,2
	Bottom	-23,5		0,032	2,5	2	1	0,02	0,2
5	Top	-23,5	10	0,032	2,5	2	1	0,02	0,2
	Bottom	-33,5		0,032	2,5	2	1	0,02	0,2

Table 6.6 : Toe model parameters

Toe model parameters									
Layer Point		**Depth**	**Thickness**	**Yield Stress**	**Quake Value 1**	**Quake Value 2**	**Yield Factor**	**Damping Constant 1**	**Power Alpha**
[-]	[-]	[m]	[m]	[MPa]	[mm]	[mm]	[-]	[MN/s/m3]	[-]
1	Top	0	3,5	1	2,5	2	0,1	0,25	0,2
	Bottom	-3,5		1	2,5	2	0,1	0,25	0,2
2	Top	-3,5	5	2	2,5	2	0,1	0,25	0,2
	Bottom	-8,5		2	2,5	2	0,1	0,31	0,2
3	Top	-8,5	4,5	1,5	2,5	2	0,1	0,31	0,2
	Bottom	-13		1,5	2,5	2	0,1	0,31	0,2
4	Top	-13	10,5	2	2,5	2	0,1	0,31	0,2
	Bottom	-23,5		2	2,5	2	0,1	0,275	0,2
5	Top	-23,5	10	2	2,5	2	0,1	0,275	0,2
	Bottom	-33,5		2	2,5	2	0,1	0,275	0,2

The accuracy of the analysis result is verified by comparing them with the field records (Fig 6.9). Piles having toe penetration depth of 30m verify the predicted analysis result. Piles numbered as 129 and 124 corresponds the required refusal criteria. Analysis results also match with the situation.

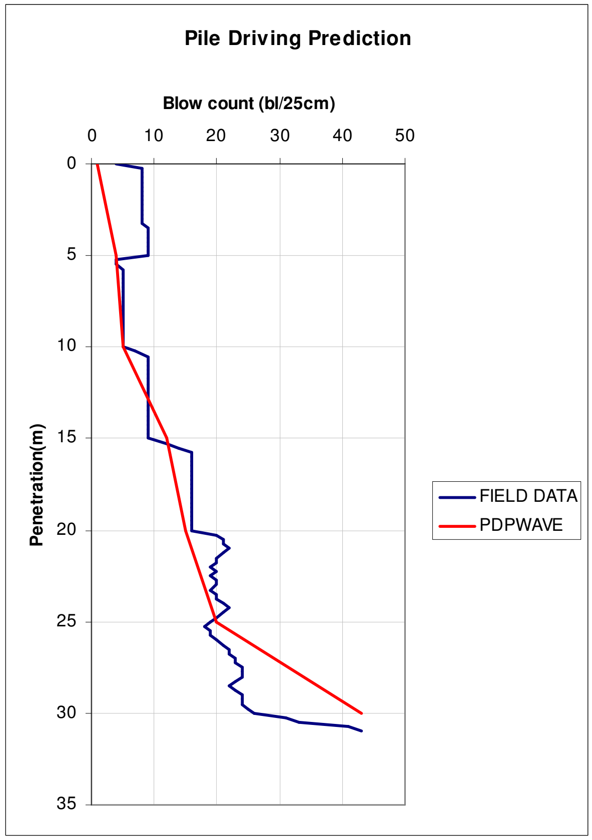


Figure 6.9 : Comparison of pile driving prediction results obtained by PDPWAVE with the field records from pile #129 for Delmag D46

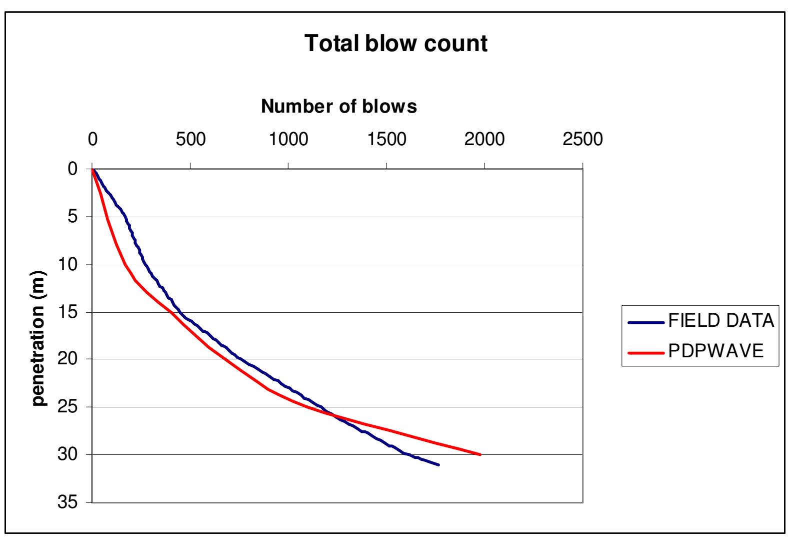


Figure 6.10 : Comparison of total blow count obtained by PDPWAVE with the field records from pile #129 for Delmag D46

6.5.3 Inverse Analysis and Idealized Soil Profile

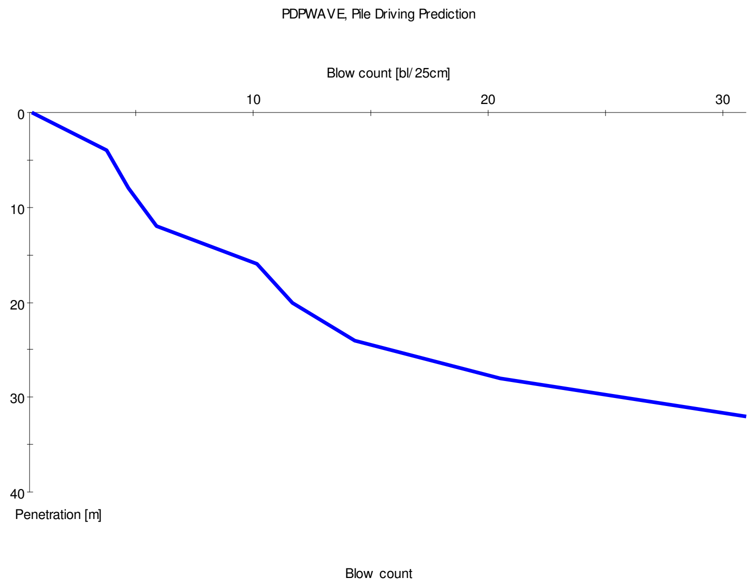
In the third and the final step of the solution methods an inverse analysis is performed to obtain the idealized soil profile comfort with the field driving records. This time the yield stress values for each soil layer obtained in section 6.5.2 in Table 6.5 and 6.6 is kept constant thus the correlation in Eq. 6.2 is solved for the SPT N_{60} values. When we obtain the N_{60} values, the idealized soil profile is build up which generally represents the field soil condition. Soil characteristics such as the consistency or the relative density do not change remarkably but on the other hand the SPT N values are distinctively lower than the actual field testing results.

Table 6.7 : Idealized soil profile

Layer Number	**Layer Boundaries**	**Depth**	**Thickness**	**Yield Stress (Shaft)**	**Yield Stress (Toe)**	**Correlated SPT N_{60} values**	**Correlated SPT N values**	**Soil Type**
[-]	[-]	[m]	[m]	[MPa]	[MPa]	[-]	[-]	[-]
1	Top	0	3,5	0,011	1	3	4	Fill Material
	Bottom	-3,5		0,011	1	3	4	
2	Top	-3,5	5	0,016	2	5	7	Loose Sand
	Bottom	-8,5		0,016	2	5	7	
3	Top	-8,5	4,5	0,032	1,5	8	10	Medium Clay
	Bottom	-13		0,032	1,5	8	10	
4	Top	-13	10,5	0,032	2	5-7	7-9	Loose Sand
	Bottom	-23,5		0,032	2	5-7	7-9	
5	Top	-23,5	10	0,032	2	9-12	12-16	Medium Clay
	Bottom	-33,5		0,032	2	9-12	12-16	

Though the marine sediments are fully saturated and consists of normally consolidated young clay the SPT N_{60} values vary between 0-9 (Bowles, 1996), in the same manner the coarse grained sand material has N_{60} values range between 3-9. The idealized soil profile totally differs from the actual site investigation test results. The constructed profile matches with the field driving records and so gain acceptance for representing the area.

This time the PDPWAVE pile driving prediction analysis is performed with the N_{60} values obtained in Table 6.7. Majority of the field records of thirty piles match the analysis results. One of them is presented in Fig 6.11 the others are presented in Appendix C for comparison and to support the obtained result.



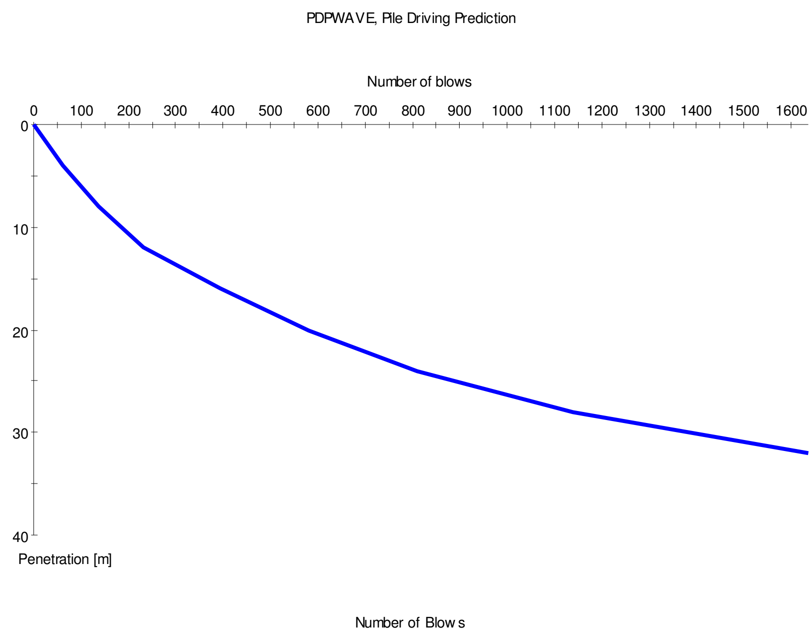


Figure 6.11 : Blow count/25cm and total blow count graphs depending on the idealized soil profile

In the following figure (Fig 6.12) the computer based results are compared with the actual records. The refusal criterion is not reached but the penetration depth of the pile meets the service requirements.

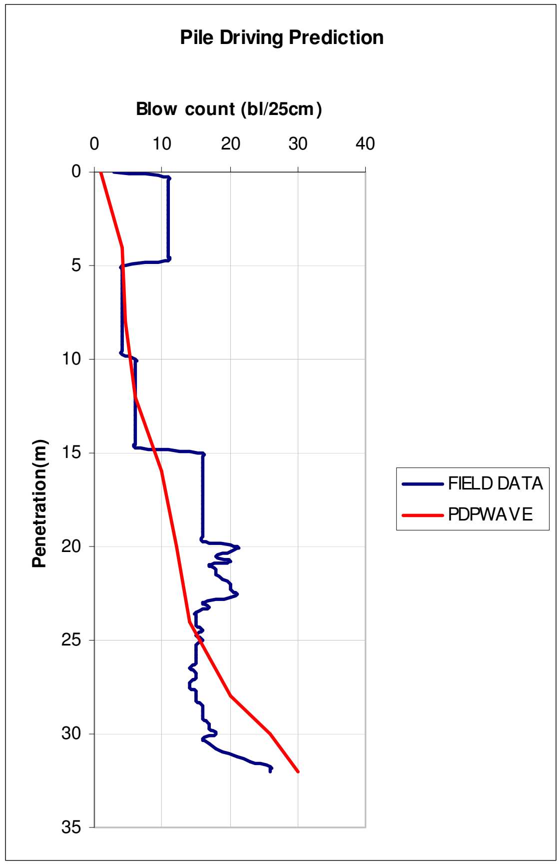


Figure 6.12 : Comparison of pile driving prediction results obtained by PDPWAVE with the field records from pile #117 for Delmag D46

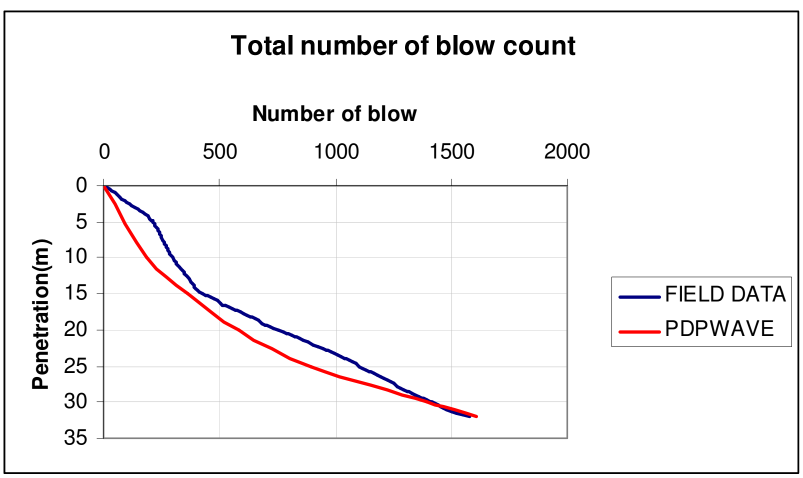


Figure 6.13 : Comparison of total blow count obtained by PDPWAVE with the field records from pile #117 for Delmag D46

Junttan HHK 9A hydraulic hammer is the other type of hammer used in application on site and the pile driving prediction analysis is performed for the corrected and idealized soil profile separately for hydraulic hammer. Comparative graphs are presented in Figure 6.14.

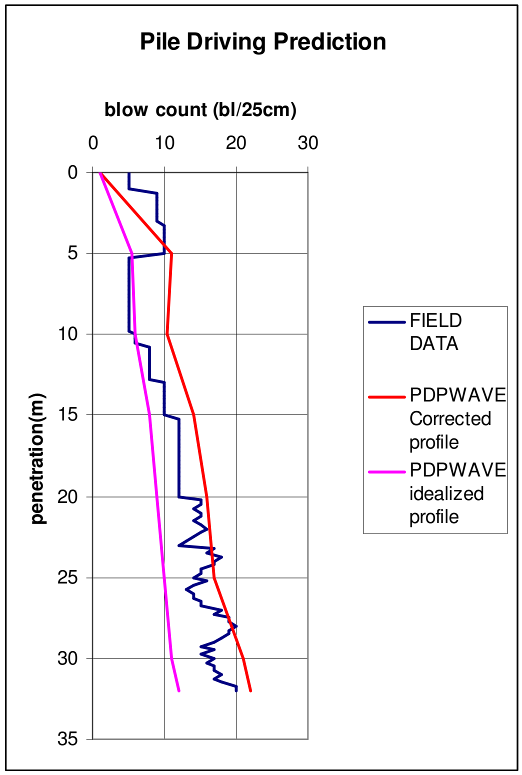


Figure 6.14 : Comparison of pile driving prediction results obtained by PDPWAVE with the field records from pile #167 for Junttan HHK 9A

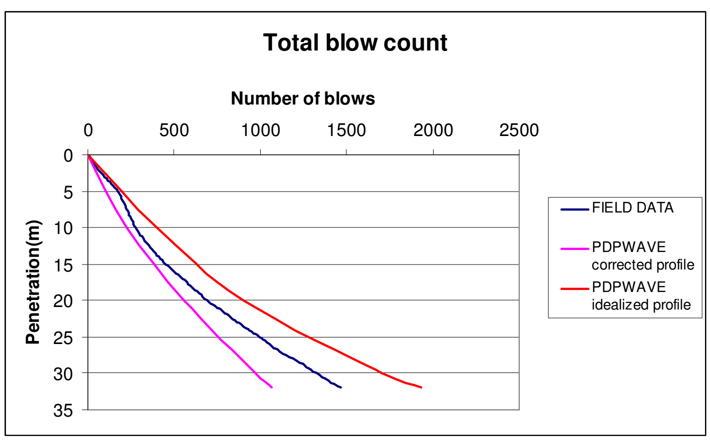


Figure 6.15 : Comparison of total blow count obtained by PDPWAVE with the field records from pile #167 for Junttan HHK 9A

As shown in Figure 6.14 the corrected profile analysis and the idealized profile analysis show the same pattern with the field data. Junttan hydraulic hammer is considered to be more powerful than the diesel hammer depending on the transferred energy to the pile. Thus, the corresponding obtained blow counts are lower than the ones obtained for the diesel hammer.

Finally, the PDPWAVE uses the method of characterstics as default solving technique but depending on the users' choice the program can run the driving analysis by using the Smith's numerical method. In scope of the study, the Smith's model is also used in order to compare the obtained results for both field data and the method of characteristics. Smith's model analysis results are given in Appendix D for study. It has been applied for both the corrected and idealized soil profile data. Both results match with the actual pattern. Thus, analysis results have been supported with an another solving technique.

7. CONCLUSION

In this study, pile driveability and behavior of pile under dynamic loading is discussed. In theoretical engineering literature, static and dynamic formulas have broad application areas. However, the developing calculation techniques over past years have made it possible to realize the inaccuracies of the conventional calculation methods. Application of wave equation analysis to piles is more accurate and reliable way for modeling the pile behavior. Analysis are based on the propagation of stress wave through pile during driving. In this way pile is assumed to be act as a propagating compression wave. Several different solution approaches have been derived in order to model the pile. Among these, method of characteristics and Smith's lumped model has gained the most currency. Based on these solution methods various user-friendly programs have been developed. PDPWAVE is one of those wave equation analysis based programs performing pile driveability studies in order to predict early or late refusal, pile damages related with optimized selection of hammer type. Theoretical knowledge forming the PDPWAVE program algorithm and the application steps is given briefly. The study is concluded with a case study in order to compare the application of program with the real field data.

The application part of this study is performed for the site located in Izmit. The data has been taken actually from the driven piles at this site. Dynamic resistances (blow count / 25cm) against pile penetration have been measured in the field. The actual driving equipment and the entire driving configuration including the pile and finally the soil model are used as an input to the PDPWAVE program. Calibration of the analysis have been made and several runs have been performed. Based on the case study performed with the Pile Driving Prediction Wave Equation Programe (PDPWAVE), the following points are concluded:

- A driveability study provides the prediction of blow count, maximum compression stresses and tensile stresses, expected driving resistance, expected hammer performance, hammer efficiency and damage relating with the pile material strength.

- The obtained analysis results match with the field data by applying the appropriate calibration. In this way the driving resistance (bl /25cm) and the total blow count plots fit with the measured data. As a result it shows that the selected type of diesel and hydraulic hammer types are appropriate for driving since they do not damage the driven pile.

- On the other hand, comformity of the results must be analyzed in detail because there are so many different parameters that affect the drivability analysis. At this point the field observation gain great importance.

- Due to the existence of several affecting factor the calculated results on computer must be verified carefully on site with reliable investigation test results. The accurate analysis results can only be achieved by making the loop of prediction, verification and post analysis, in a sufficient number. Experience can only be build up in this way and an increase in accuracy will be obtained.

- The experience of an engineer plays the most important role while selecting the appropriate dynamic model parameters and judgment.

- The resistance of pile and soil has major influence on the performance of a diesel hammer and has to be taken into consideration in the prediction.

- Finally the site investigation test results must be reliable. Tests must be performed with a great care.

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www.pilebuck.com

www.geoforum.com

APPENDIX A

YER : SERBEST BÖLGE	OFFSET :	KUYU NO :	S101
EKİPMAN : GMS-300		ZEMİN KOTU :	0.00 m.
SONDAJ YÖNTEMİ : ROTARY ; 0.00-30.45m arası.	KOORD. : N :	E :	
KUYU ÇAPI : 0.00-30.00m arası - 89mm.	BAŞLANGIÇ : 06.08.2005	BİTİŞ : 07.08.2005	

NUMUNE VE YERİNDE DENEY. Drn. (M)	TİP	S.P.T. darbe sayısı 15	7.5 / 7.5	7.5 / 7.5	N	Muh. Drn. (m) Tarih	Y.A.S. Drn. (m)	TCR %	RQD %	SCR %
1.50 – 1.95	D1	5	4	7	11	06.08.20	YAS 0.50			
3.00 – 3.45	D2	1	2	2	4					
4.50 – 4.95	D3	2	2	1	3					
6.00 – 6.45	D4	1	2	3	5					
7.50 – 7.95	D5	1	2	2	4					
9.00 – 9.45	D6	2	3	4	7					
10.50 – 10.95	D7	1	2	3	5					
12.00 – 12.45	D8	2	4	5	9					
13.50 – 13.95	D9	11	17	21	38					
15.00 – 15.45	D10	13	19	22	41					
16.50 – 16.95	D11	18	23	26	49					
18.00 – 18.45	D12	17	21	25	46					
19.50 – 19.95	D13	20	23	23	46					

Drn. (m)	ZEMİN CİNSİ	KOT (m)	LEJAND
2.50	Dolgu zemin (Killi kumlu kırmataş)	-2.50	
4.00	Yumuşak, koyu gri, killi SILT	-4.00	
5.50	Gevşek, gri renkli, siltli killi KUM	-5.50	
8.50	Yumuşak, grimsi kahve renkli, KIL	-8.50	
10.00	Gevşek, grimsi kahve renkli, killi KUM	-10.00	
13.00	Orta katı, grimsi kahve renkli KIL	-13.00	
	Sıkı, gri renkli, az çakıllı KUM		

YER : SERBEST BÖLGE	OFFSET :	KUYU NO :	S101
EKİPMAN : GMS-300	,	ZEMİN KOTU :	0.00 m.
SONDAJ YÖNTEMİ : ROTARY - 0.00-30.45m arası.	KOORD. : N :	E :	
KUYU ÇAPI : 0.00-30.00m arası - 89mm. 30.00-30.45m arası - 76mm.	BAŞLANGIÇ : 06.08.2005	BİTİŞ : 07.08.2005	

NUMUNE VE YERİNDE DENEY Drn. (M)	TİP	S.P.T. darbe sayısı 15	7.5	7.5	7.5	7.5	Muh. Drn. (m) Tarih	Y.A.S. Drn. (m)	TCR %	RQD %	SCR %	Drn. (m)	ZEMİN CİNSİ	KOT (m)	LEJAND
							06.08.20								
21.00 - 21.45	D14	20		25	N = 51	26									
22.50 - 22.95	D15	24		24	N = 55	31									
24.00 - 24.45	D16	25		28	N = 58	30									
25.50 - 25.95	D17	20		22	N = 46	24							Sıkı, gri renkli, az çakıllı KUM		
27.00 - 27.45	D18	16		20	N = 40	20									
28.50 - 28.95	D19	17		21	N = 44	23									
30.00 - 30.45	D20	23		23	N = 46	23	30.00					30.45	SONDAJ BİTİMİ	-30.45	

YER : SERBEST BÖLGE	OFFSET :
KUYU NO : S104	
EKİPMAN : GMS-300	ZEMİN KOTU : 0.00 m.
SONDAJ YÖNTEMİ : ROTARY : 0.00-33.45m arası.	KOORD. : N : E :
KUYU ÇAPI : 0.00-30.00m arası - 89mm.	BAŞLANGIÇ : 06.08.2005 BİTİŞ : 07.08.2005

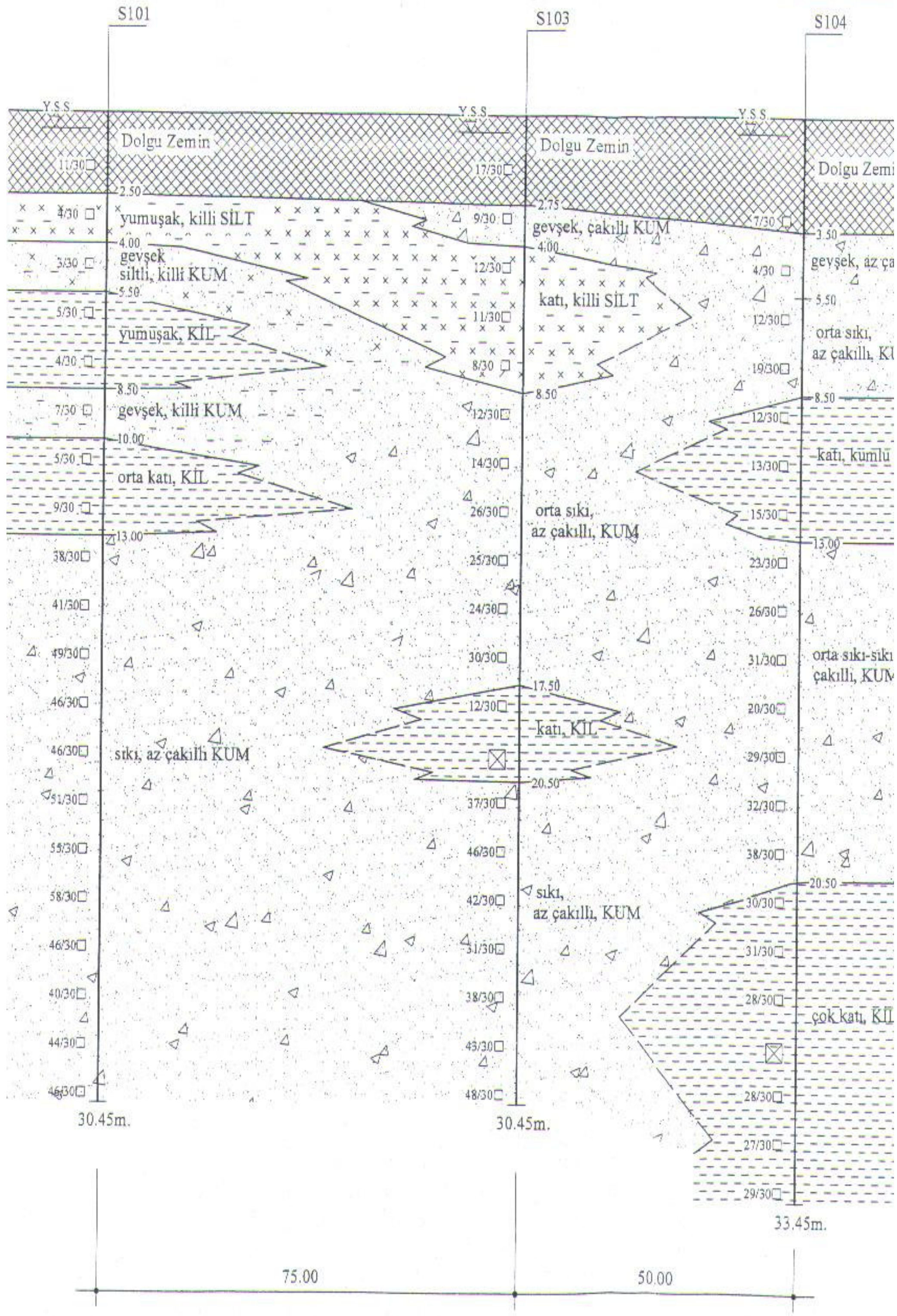
NUMUNE VE YERİNDE DENEY. Drn. (M)	TİP	S.P.T. darbe sayısı 15	7.5	7.5	7.5	7.5	N	Muh. Drn. (m) Tarih	Y.A.S. Drn. (m)	TCR %	RQD %	SCR %
								06.08.20	Y A S 0.50			
1.50 / 1.95		50										
3.00 / 3.45	D1	7		5		2	N = 7					
4.50 / 4.95	D2	2		2		2	N = 4					
6.00 / 6.45	D3	5		5		7	N = 12					
7.50 / 7.95	D4	5		9		10	N = 19					
9.00 / 9.45	D5	4		5		7	N = 12					
10.50 / 10.95	D6	6		6		7	N = 13					
12.00 / 12.45	D7	6		7		8	N = 15					
13.50 / 13.95	D8	7		10		13	N = 23					
15.00 / 15.45	D9	7		11		15	N = 26					
16.50 / 16.95	D10	10		14		17	N = 31					
18.00 / 18.45	D11	10		10		10	N = 20					
19.50 / 19.95	D12	10		14		15	N = 29					

Drn. (m)	ZEMİN CİNSİ	KOT (m)	LEJAND
3.50	Dolgu zemin	-3.50	
5.50	Gevşek, gri renkli, az çakıllı KUM	-5.50	
8.50	Orta sıkı, gri renkli, az çakıllı KUM	-8.50	
13.00	Katı, grimsi kahve renkli, kumlu KİL	-13.00	
	Orta sıkı-sıkı, gri renkli, çakıllı KUM		

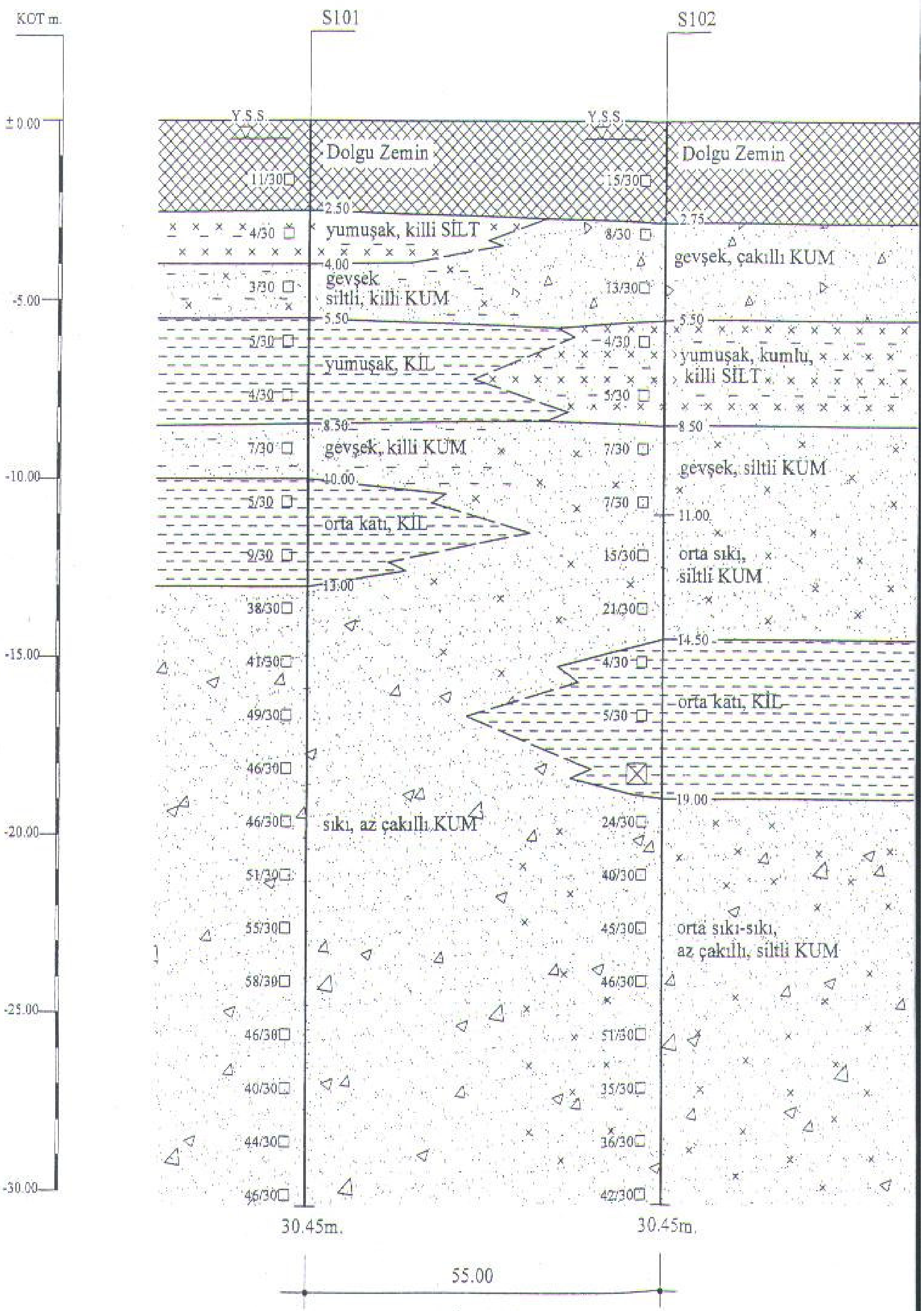
YER : SERBEST BÖLGE	OFFSET :	KUYU NO : S104
EKİPMAN : GMS-300		ZEMİN KOTU : 0.00 m.
SONDAJ YÖNTEMİ : ROTARY : 0.00-33.45m arası	KOORD. : N :	E :
KUYU ÇAPI : 0.00-30.00m arası - 89mm. 30.00-33.45m arası - 76mm.	BAŞLANGIÇ : 06.06.2005	BİTİŞ : 07.08.2005

NUMUNE VE YERİNDE DENEY. Drn. (M)	TİP	S.P.T. darbe sayısı 15	7.5	7.5	7.5	7.5	N	Muh. Drn. (m) Tarih	Y.A.S. Drn. (m)	TCR %	RQD %	SCR %	Drn. (m)	ZEMİN CİNSİ	KOT (m)	LEJAND
								06.06.20								
21.00 21.45	D13	12		15		17	N = 32									
														Orta sıkı-sıkı, gri renkli, çakıllı KUM		
22.50 22.95	D14	10		18		20	N = 38									
													23.50		-23.50	
24.00 24.45	D15	13		15		15	N = 30									
25.50 25.95	D16	15		15		16	N = 31									
27.00 27.45	D17	9		13		15	N = 28									
28.50 29.00	U1													Çok katı, kahve renkli, KİL		
30.00 30.45	D18	10		12		16	N = 28	30.00								
31.50 31.95	D19	10		12		15	N = 27									
33.00 33.45	D20	10		14		15	N = 29						33.45	SONDAJ BİTİMİ	-33.45	

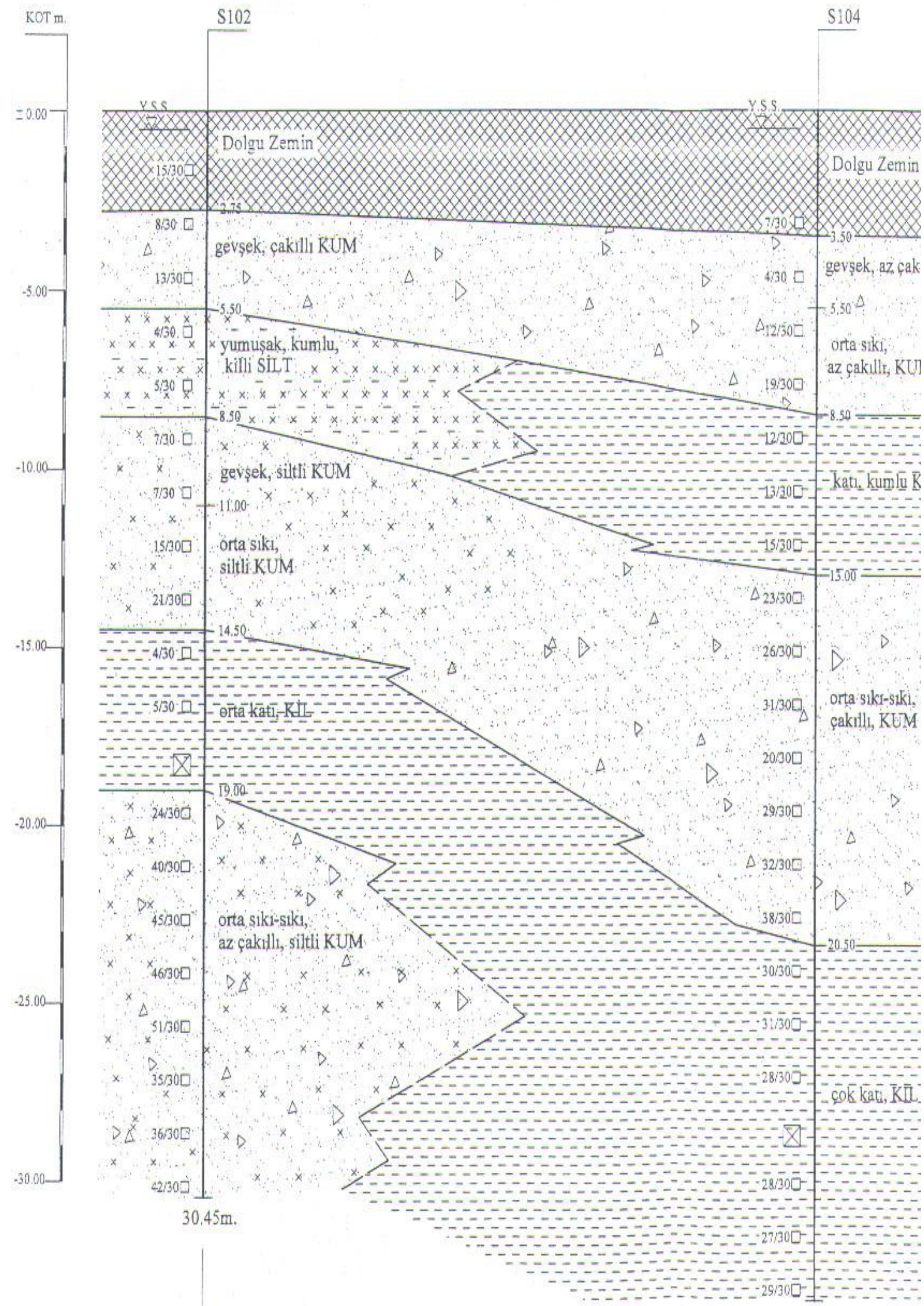
APPENDIX B


S101
S103
S104
Y.S.S.
Dolgu Zemin
yumuşak, killi SİLT
gevşek siltli, killi KUM
yumuşak, KİL
gevşek, killi KUM
orta katı, KİL
sıkı, az çakıllı KUM
gevşek, çakıllı KUM
katı, killi SİLT
orta sıkı, az çakıllı, KUM
katı, KİL
sıkı, az çakıllı, KUM
orta sıkı, az çakıllı, KU
katı, kumlu
orta sıkı-sıkı çakıllı, KUM
çok katı, KİL
30.45m.
30.45m.
33.45m.
75.00
50.00

I' – I'

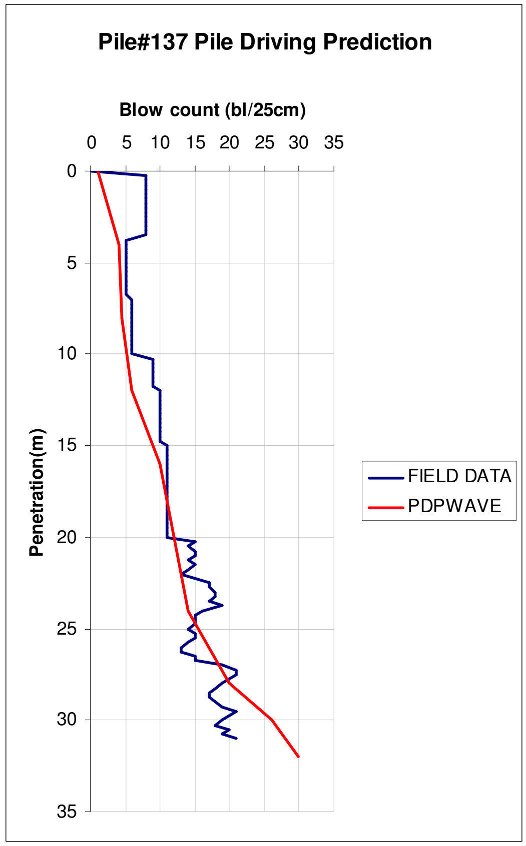

KOT m.
S101
S102
±0.00
-5.00
-10.00
-15.00
-20.00
-25.00
-30.00
Y.S.S.
Dolgu Zemin
11/30
15/30
2.50
2.75
yumuşak, killi SİLT
4/30
8/30
gevşek, çakıllı KUM
4.00
gevşek
siltli, killi KUM
3/30
13/30
5.50
5/30
4/30
yumuşak, KİL
yumuşak, kumlu, killi SİLT
5/30
4/30
8.50
7/30
gevşek, killi KUM
7/30
gevşek, siltli KUM
10.00
5/30
7/30
11.00
orta katı, KİL
9/30
15/30
orta sıkı, siltli KUM
13.00
38/30
21/30
14.50
41/30
4/30
49/30
5/30
orta katı, KİL
46/30
19.00
46/30
sıkı, az çakıllı KUM
24/30
51/30
40/30
55/30
45/30
orta sıkı-sıkı, az çakıllı, siltli KUM
58/30
46/30
46/30
51/30
40/30
35/30
44/30
36/30
46/30
42/30
30.45m.
30.45m.
55.00

II' – II'

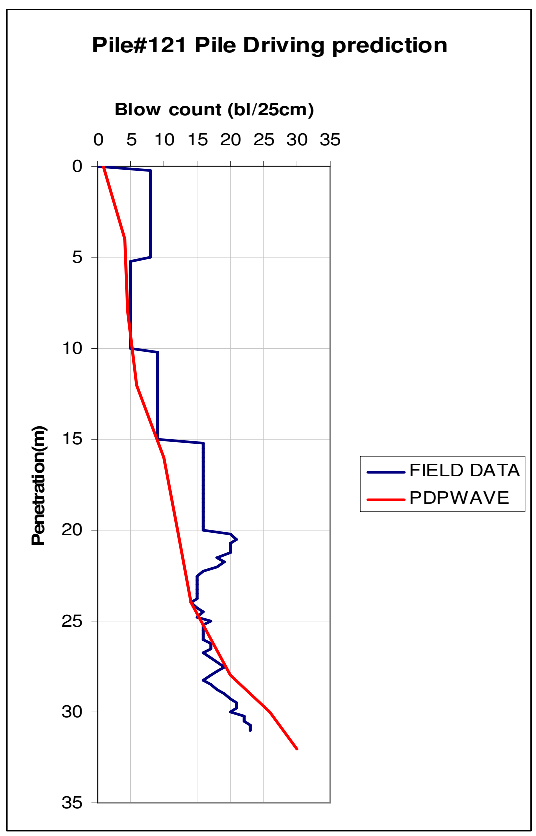

KOT m.
S102
S104
± 0.00
-5.00
-10.00
-15.00
-20.00
-25.00
-30.00
Y.S.S.
Dolgu Zemin
15/30
2.75
8/30
gevşek, çakıllı KUM
13/30
5.50
4/30
yumuşak, kumlu, killi SİLT
5/30
8.50
7/30
gevşek, siltli KUM
7/30
11.00
orta sıkı, siltli KUM
15/30
21/30
14.50
4/30
5/30
orta katı, KİL
19.00
24/30
40/30
45/30
orta sıkı-sıkı, az çakıllı, siltli KUM
46/30
51/30
35/30
36/30
42/30
30.45m.
Y.S.S.
Dolgu Zemin
7/30
3.50
gevşek, az çak
4/30
5.50
12/50
orta sıkı, az çakıllı, KU
19/30
8.50
12/30
katı, kumlu K
13/30
15/30
15.00
23/30
26/30
31/30
orta sıkı-sıkı, çakıllı, KUM
20/30
29/30
32/30
38/30
20.50
30/30
31/30
28/30
çok katı, KİL
28/30
27/30
29/30
33.45m.
105.00

III' – III'

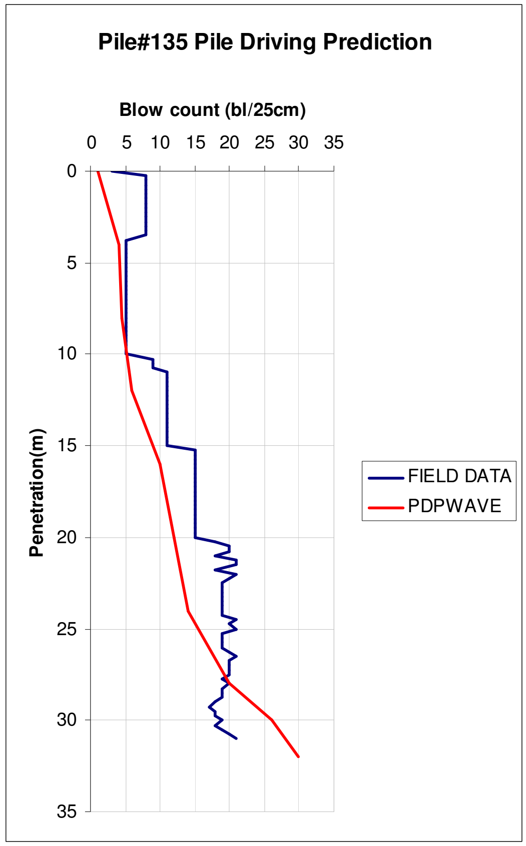
APPENDIX C

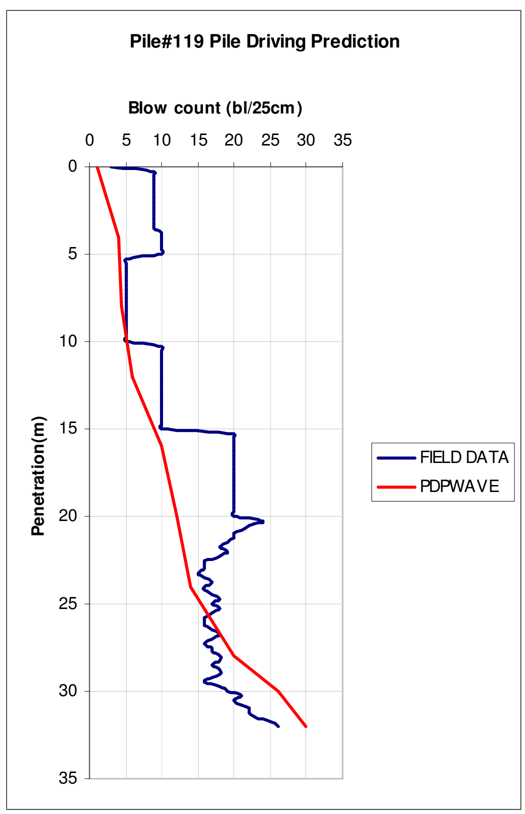


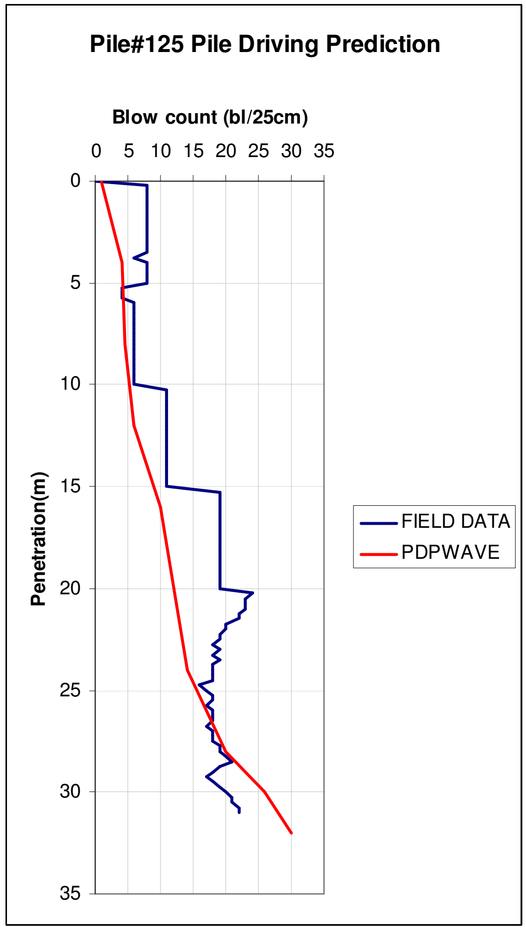
Pile#137 Pile Driving Prediction
Blow count (bl/25cm)
0 5 10 15 20 25 30 35
Penetration(m)
0 5 10 15 20 25 30 35
FIELD DATA
PDPWAVE

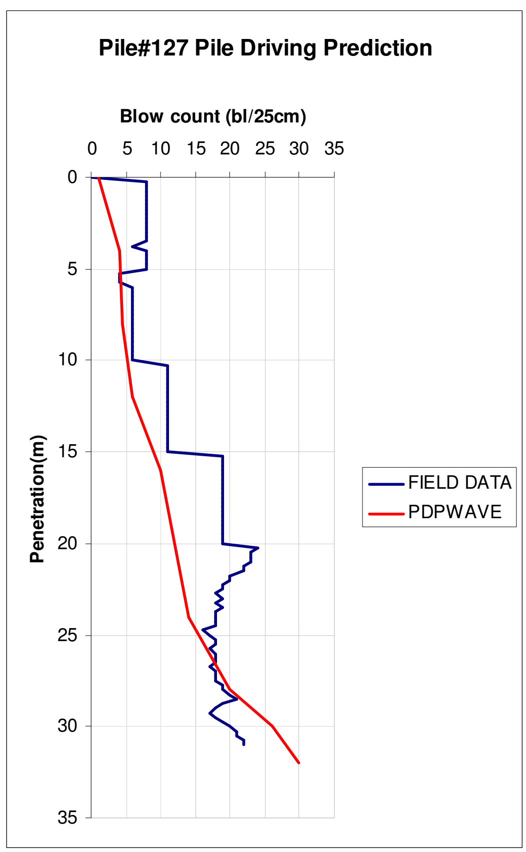


Pile#121 Pile Driving prediction
Blow count (bl/25cm)
0 5 10 15 20 25 30 35
Penetration(m)
0 5 10 15 20 25 30 35
FIELD DATA
PDPWAVE

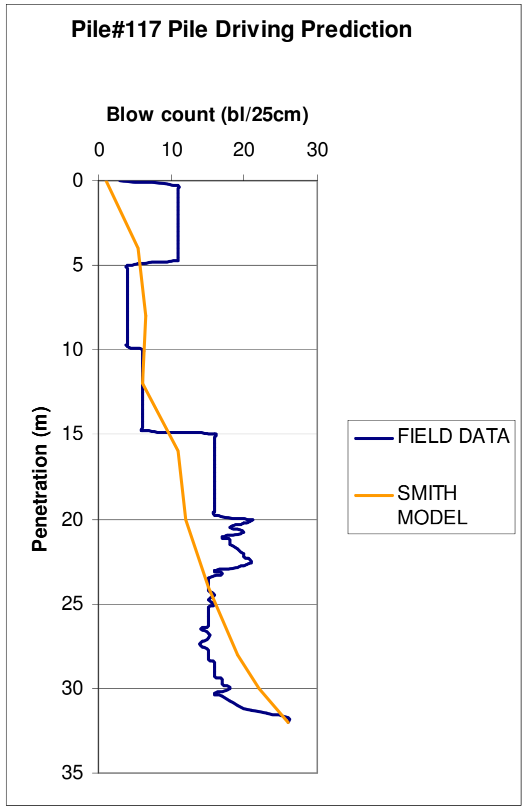

Pile#135 Pile Driving Prediction
Blow count (bl/25cm)
0 5 10 15 20 25 30 35
Penetration(m)
0 5 10 15 20 25 30 35
FIELD DATA
PDPWAVE

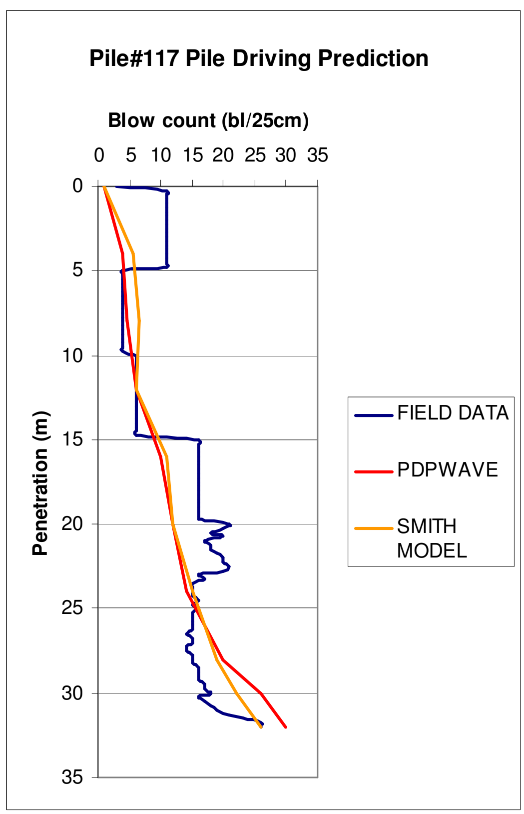

Pile#119 Pile Driving Prediction
Blow count (bl/25cm)
0 5 10 15 20 25 30 35
Penetration(m)
0 5 10 15 20 25 30 35
FIELD DATA
PDPWAVE


Pile#125 Pile Driving Prediction
Blow count (bl/25cm)
0 5 10 15 20 25 30 35
0 5 10 15 20 25 30 35
Penetration(m)
FIELD DATA
PDPWAVE


Pile#127 Pile Driving Prediction
Blow count (bl/25cm)
0 5 10 15 20 25 30 35
0 5 10 15 20 25 30 35
Penetration(m)
FIELD DATA
PDPWAVE

APPENDIX D


Pile#117 Pile Driving Prediction
Blow count (bl/25cm)
0
10
20
30
0
5
10
15
20
25
30
35
Penetration (m)
FIELD DATA
SMITH MODEL


Pile#117 Pile Driving Prediction
Blow count (bl/25cm)
0
5
10
15
20
25
30
35
0
5
10
15
20
25
30
35
Penetration (m)
FIELD DATA
PDPWAVE
SMITH MODEL

BIOGRAPGHY

She was born in Istanbul in 1981. She attended the Department of Geophysics Engineering of Mine faculty of Istanbul Technical University (ITU). She graduated as a Geophysics Engineer in 2003 with a degree of high honour. She then attended M.Sc. program of Soil Mechanics and Geotechnical Engineering Department of Civil Engineering Department of Istanbul Technical University. While studying on M Sc she started to work as a research assistant at the Geotechnical Engineering Department. Her research fields are the signal analysis, surface waves, nondestructive investigation of soil and structural elements and pile driving prediction analysis.