ISTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

QUALITY CONTROL OF PILES CONSTRUCTED IN CLAYEY SOILS WITH SONIC INTEGRITY TESTING (SIT)

M.Sc. Thesis by
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Department : Civil Engineering

Programme: Soil Mechanics and
Geotechnical Engineering

JANUARY 2007

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PREFACE

I would like to thank to the respectful members of Soil Mechanics and Geotechnical Department of Istanbul Technical University for giving me the opportunity to study my MSc thesis. I specially would like to tell my regards to my supervisor Assist. Prof. Aykut ŞENOL for spending their valuable time on my thesis, giving their knowledge and interest and I would like to thank for their support and belief in that this thesis will be completed on time. I also would like to thank Mr. Orhan İnanır for his great support and Mr. Hakan Demir for sharing his knowledge and help with me, and to Mr. Semih Şentürk and Mr. Özgün Inan for their kindness help during the study of this thesis.

January 2007 B.Sc. Fuat ÖZBAŞ

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ACRONYMS

BS : British Standards
CFA : Continuous Flight Auger
SIT : Sonic Integrity Test
USACE : United States Army Corps of Engineers

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A : cross sectional area [m^2]
c : stress wave velocity [m/s]
F : the axial force in the bar (compression = positive)
f_0 : frequency at end of initial slope [s^{-1}]
k : dynamic pile stiffness [N/m]
l : pile length [m]
N : admittance [m/Ns]
t : time
x : the Lagrangian coordinate
Z : impedance [Ns/m]
Z_0 : impedance at end of initial slope [s/kg]
Δf : distance between two successive peaks [s^{-1}]
u : displacement
ρ : density [kg/m^3]

ÖZET

Bu çalışmada, İstanbul ili, Esenyurt semtindeki bir toplu konut projesinin temel inşaatında imal edilen 50 cm capında ve 12 ila 15 metre arasında değişen uznunluklardaki fore kazıkların “Sonik Süreklilik Testi” yöntemi ile kalite kontrolü yapılarak, sonuçlar değerlendirilmiştir. Tamamı kil tabakalardan oluşan zeminde inşa edilen 685 kazığın 166 tanesi (yaklaşık %24’ü) “Sonik Süreklilik Testi” ile test edilmiş ve bu kazıkların süreksizlikleri ve bu süreksizliklerin meydana geldiği derinlikler tespit edillerek, bu veriler tablolar ve grafikler halinde verilmiştir. Rastlanılan süreksizlik yüzdeleri ve bunların derinlikle olan ilişkisi incelenmiştir. İnceleme sonucunda genişleme tipi süreksizliklerin %54’ünün kazığın başlığının yaklaşık 1,50 m altı ila 4,50 m altı aralığında meydana geldiği tespit edilmişdir. Kazık çapında daralma tipi süreksizlik ise test edilen 166 kazığın sadece 2’sinde (%1,20) gözlemlenmiş ve bu kazıklar proje firmasınında onayı alınarak yeni kazıklar ile değiştirilmiştir.

ABSTRACT

In this study, cast-in-situ piles with 50 cm diameter and lengths ranging from 12 m to 15 m, which had been constructed under coverage of Housing Project located at Istanbul city, Esenyurt district, were tested for quality control with “Sonic Integrity Testing” and results were evaluated. 166 of 685 (approximately 24%) piles, constructed in soil composed of clay layers, were tested with “Sonic Integrity Testing”, discontinuities and their locations on piles were detected and data obtained from these tests were presented as tables and graphs. Percentages of discontinuities experienced and their relations with depth were investigated. As a result of this investigation, it is determined that 54% enlargement in cross-section type discontinuities were experienced in depth range 1,5 m to 4,5 m from pile top. Decrement type discontinuities were only experienced at 2 out of 166 piles (1,2*%) and these piles were replaced with new ones with approval of project designer.

1 INTRODUCTION

Pile foundations are the part of a structure used to carry and transfer the load of the structure to the bearing stratum located at some depth below ground surface. The main components of the pile foundations are the pile cap and the piles. Piles are long and slender members, which transfer the load to deeper soil or rock of high bearing capacity avoiding shallow soil of low bearing capacity, The main types of materials used for piles are timber, steel and concrete.

Piles transfer the load to ground in two ways, skin friction and base bearing. In general, project designers prefer one of these methods according to soil conditions and design the pile dimensions and reinforcement in accordance with preference. On the other hand, both skin friction and base bearing can be used in the same design.

Being unseen creates significant question marks about quality of piles that are constructed under foundation, particularly in cast-in-situ piles. Direct relation between load capacity and quality of piles makes the quality essential. Although, inspections are carried out during construction to ensure the highest quality, it is impossible to determine if the pile is constructed in intended quality or not without testing. This leaded to need of methods for testing the constructed piles.

Wide ranges of testing methods are now available for determining pile quality categorized under two main groups, which are destructive and non-destructive testing methods. Both categories have their own advantages and disadvantages, but owing to being economical, time consuming and easy to perform, non-destructive testing methods become more popular.

In this study, a non-destructive testing method “Sonic Integrity Testing” (SIT) was used in quality control of 166 out of 685 piles constructed under foundations of 16 residences in scope of Housing Project located in Istanbul city, Esenyurt district. The aim was to detect major discontinuities and determine their relations with depth and soil layers. Significant amount of data were obtained from SIT tests and evaluation of these data are presented as tables and graphs under coverage of this study.

2 PILES

A simple division of 'driven' or 'bored' piles is often a starting point, when defining types of piling works. This may be adequate in normal situations, but does not satisfactorily deal with the many hybrid types of pile that are in use. A more precise division into 'large displacement', 'small displacement' or 'replacement' piles overcomes this difficulty. (BS 8004, 1986)

With the large and small displacement piles, soil is moved radially, as the shaft enters the ground. There may also be a component of movement of the soil in the vertical direction. Granular soils are compacted by the displacement process. Large displacement piles include solid section or hallow section piles with a closed end. All types of driven and cast-in-place piles are in this category. Small displacement piles are also driven into the ground, but they have quite small cross sectional area. (www.pileinfo.com)

With the replacement piles, lateral stresses in the ground are reduced during excavation and only partly reinstated by concreting. Problems resulting from soil displacement are therefore eliminated, but the benefit of compaction in granular soils is lost. Replacement piles are formed by removing soil by boring and filling the borehole with concrete. Also, preformed elements (made up of timber, concrete or steel) may be placed into the borehole.(www.pileinfo.com)

These three main pile types can be classified even further. Types of piles according to BS 8004 are shown in Figure 2.1 as a diagram.

2.1 Driven Displacement Piles

2.1.1 Timber Piles

Timber is an ideal material for piling. It is easy to handle, it has a high strength to weight ratio, it is readily cut to length and trimmed after driving, and in favorable conditions of exposure durable type have an almost indefinite life. Timber piles, used in their most economical form, consist of round untrimmed logs which are driven butt uppermost. (Chellis, 1961 and Tomlinson, 1994)

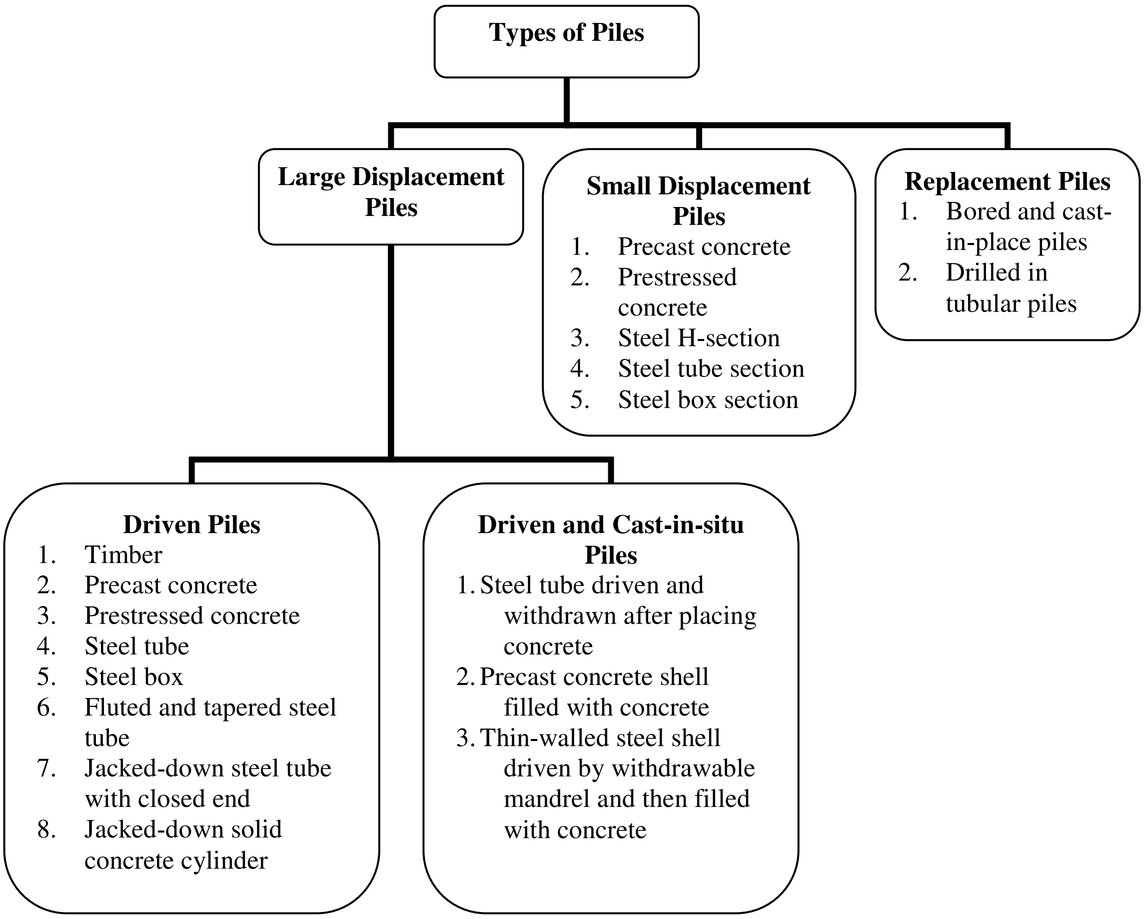


Figure 2.1: Types of Piles

Timber piles, when placed completely below ground-water level, are resistant to fungal decay and have an almost indefinite life. However, the portion above ground-water level, in a structure on land, is liable to decay. Although creosote or other preservatives extend the life of timber in damp or dry conditions, they will not lengthen its useful life. Therefore the usual practice is to cut off timber piles just below the lowest predicted ground-water level and to extend them above this level in concrete. If the ground-water level is shallow, the pile cap can be taken down below the water level. (Tomlinson, 1994)

In marine structures, timber piles are liable to be severely damaged by the mollusc-type borers, which infest the sea-water in many parts of the world. The severity of this form of attack can be reduced by using softwood, impregnated with creosote, or greatly minimized by the use of a hardwood, of a type known to be resistant to borer attack. (Tomlinson, 1994 and USACE, 2005)

When calculating the working stress on a pile, allowance must be made for bending stresses, due to eccentric and lateral loading and to eccentricity caused by deviations in the straightness and inclination of a pile. Allowance must also be made for reductions in the cross-sectional area due to drilling or notching, and to the taper on a round log. (Tomlinson, 1994)

During driving process, damage to a pile is most likely to occur at the head and toe. The problems of splitting of the heads and unseen brooming and splitting of the toes of timber piles occur, when it is necessary to penetrate layers of compact or cemented soils to reach the desired depth. This damage can also occur when attempting to drive deeply into dense sands and gravels or into soils containing boulders, in order to mobilize the required skin-frictional resistance for a given uplift or compressive load. Judgement is required to assess the soil conditions at a site so as to decide whether or not it is feasible to drive a timber pile to the desired depth without damage, or whether it is preferable to reduce the working load to a value which permits the usage of a shorter pile. As an alternative, jetting or pre-boring may be adopted to reduce the amount of driving required. The temptation to continue hard driving in an attempt to achieve an arbitrary set for compliance with some dynamic formula must be resisted. (Prakash and Sharma, 1990 and Tomlinson, 1994)

Minimizing the damage to a pile can be done by reducing the number of hammer blows necessary to achieve the desired penetration as far as possible and also by limiting the height of drop of the hammer. This necessitates the use of a heavy hammer, which should at least be equal in weight to the weight of the pile for hard driving conditions, and to one-half of the pile weight for easy driving. The lightness of a timber pile can be an embarrassment when driving groups of piles through soft clays or silts to a point bearing on rock. Frictional resistance in the soft materials can be very low for a few days after driving, and the effect of pore pressures caused by driving adjacent piles in the group may cause the piles, which are already driven, to rise out of the ground due to their own buoyancy relative to that of the soil. The only solution is to apply loads to the pile heads until all the piles in the area have been driven. (Prakash and Sharma, 1990 and Tomlinson, 1994)

During driving process, heads of timber piles should be protected against splitting by means of a mild steel hoop slipped over the pile head or screwed to it. A squared pile toe can be provided where piles are terminated in soft to moderately stiff clays.

During driving them into dense or hard materials, a cast steel point should be provided. As an alternative to a hoop, a cast steel helmet can be fitted to the pile head during driving. (Tomlinson, 1994)

By joining timber piles in the leaders of the piling frame, they can be driven in very long lengths in soft to firm clays. The abutting surfaces of the timber should be cut truly square at the join positions in order to distribute the stresses caused by driving and loading evenly over the full cross-section. (Tomlinson, 1994)

2.1.2 Precast Concrete Piles

Precast concrete piles are used in situations where the use of driven-and-cast-in-situ piles is impracticable or uneconomical like marine and river structures. For land structures unjointed precast concrete piles are frequently more costly than driven-and-cast-in-situ types for two main reasons. First one is, “Reinforcement must be provided in the precast concrete pile to resist the bending and tensile stresses which occur during handling and driving (Once the pile is in the ground, and if mainly compressive loads are carried, the majority of this steel is unnecessary)” and second one is, “The precast concrete pile is not readily cut down or extended to suit variations in the level of the bearing stratum to which the piles are driven”. However, there are many situations for land structures where the precast concrete pile can be the more economical. Where large numbers of piles are to be installed in easy driving conditions the savings in cost due to the rapidity of driving achieved may outweigh the cost of the heavier reinforcing steel necessary. Reinforcement may be needed in any case to resist bending stresses due to lateral loads or tensile stresses from uplift loads. Where high-capacity piles are to be driven to a hard stratum savings in the overall quantity of concrete compared with cast-in-situ piles can be achieved since higher working stresses can be used. Where piles are to be driven in sulphate-bearing ground or into aggressive industrial waste materials, the provision of sound high-quality dense concrete is ensured. The problem of varying the length of the pile can be overcome by adopting a jointed type. (Chellis, 1961 and Tomlinson, 1994)

There is still quite a wide range of employment for the precast concrete pile, especially for projects where the costs of establishing a precasting yard can be spread over a large number of piles. The piles can be designed and manufactured in ordinary reinforced concrete, or in the form of pre-tensioned or post-tensioned prestressed concrete members. The ordinary reinforced concrete pile is likely to be preferred for a project requiring a quite small number of piles, where the cost of establishing a production line for prestressing work on site is not reasonable and where the site is too far from an established factory to allow the economical transportation. In countries, where the precast concrete pile is used widely, the ordinary reinforced concrete pile is chosen to the prestressed design in almost all circumstances. (Chellis, 1961 and Tomlinson, 1994)

Precast concrete piles in ordinary reinforced concrete are usually square or hexagonal and of solid cross-section for units of short or moderate length. For saving weight, long piles are usually manufactured with a hollow interior in hexagonal, octagonal or circular sections. The interiors of the piles can be filled with concrete after driving. This is necessary to avoid bursting, where piles are exposed to severe frost action. Alternatively, drainage holes can be provided to prevent water accumulating in the hollow interior. (Chellis, 1961 and Tomlinson, 1994)

Economies in concrete and reductions in weight for handling can be achieved by providing the piles with an enlarged toe, if piles are designed to carry the applied loads mainly in end bearing. (Chellis, 1961 and Tomlinson, 1994)

The amount of main reinforcing steel in the form of longitudinal bars is determined by the bending moments induced when the pile is lifted from its casting bed to the stacking area. The magnitude of the bending moments depends on the number and positioning of the lifting points. In some cases the size of the externally applied lateral or uplift loads may necessitate more main steel than is required by lifting considerations. Lateral steel in the form of hoops and links is provided to prevent shattering or splitting of the pile during driving. In hard driving conditions, it is advantageous to place additional lateral steel in the form of a helix at the head of the pile. The helix should be about two pile widths in length with a pitch equal to the spacing of the link steel at the head. It can have zero cover where the pile head is to be cut down for bonding to the cap. (Chellis, 1961 and Tomlinson, 1994)

If it is compared with ordinary reinforced concrete piles, prestressed concrete piles have certain advantages. Their principal advantage is in their higher strength to weight ratio, enabling long slender units to be lifted and driven. However, slenderness is not always advantageous since a large cross-sectional area may be needed to mobilize sufficient resistance in skin friction and end bearing. The second main advantage is the effect of the prestressing in closing up cracks caused during handling and driving. This effect, combined with the high-quality concrete necessary for economic employment of prestressing, gives the prestressed pile increased durability which is advantageous in marine structures and corrosive soils. (Chellis, 1961 and Tomlinson, 1994)

For economy in materials, prestressed concrete piles should be made with designed concrete mixes with a minimum 28-day works cube strength of 40N/mm^2. As in the case of timber piles, this limitation is to prevent unseen damage to piles, which may be over-driven to achieve an arbitrary set given by a dynamic pile-driving formula.

Where precast concrete piles are driven through soft or loose soils into dense sands and gravels or firm to stiff clays, metal shoes are not required at the toes. A blunt pointed end appears to be just as effective in achieving the desired penetration in these soils as a more sharply pointed end and the blunt point is better for maintaining alignment during driving. A cast-iron or cast-steel shoe fitted to a pointed toe may be used for penetrating rocks or for splitting cemented soil layers. The shoe serves to protect the pointed end of the pile. (Tomlinson, 1994)

Piles may be cast on mass concrete beds, using removable side forms of timber or steel. The reinforcing cage is suspended from bearers with spacing forks to maintain alignment. Spacer blocks to maintain cover are undesirable. The stop ends must be set truly square with the pile axis to ensure an even distribution of the hammer blow during driving. Vibrators are used to obtain thorough compaction of the concrete. Furthermore the concrete between the steel and the forms should be worked with a slicing tool to eliminate honeycombed patches. The casting beds must be sited on firm ground, in order to prevent bending of the piles during and soon after casting. After removing the side forms, the piles already cast may be used as side forms for casting another set of piles in between them. If this is done, the side forms should be set to give a trapezoidal cross-section in order to ease release. Piles may also be cast in tiers on top of each other, but a space between them should be maintained to allow air to circulate. Casting in tiers involves a risk of distortion of the piles, due to settlement of the stacks. In addition, the piles, which are first to be cast are the last to be lifted, because the most-mature piles should be the first to be lifted and driven. (Chellis, 1961 and Tomlinson, 1994)

If piles are made in a factory, permanent casting beds can be formed in reinforced concrete with heating elements embedded in them to allow a 24-hour cycle of casting and lifting from the moulds. This type, which does not have removable side forms, necessitates the embedment of lifting plugs or loops into the tops of the piles. (,Tomlinson, 1994)

2.1.3 Jointed Precast Concrete Piles

The disadvantages of having to adjust the lengths of precast concrete piles, either by cutting off the excess or casting on additional lengths, to accommodate variations in the depth to a hard bearing stratum will be evident. These drawbacks can be overcome by employing jointed piles, in which the adjustments in length can be made by adding or taking away short lengths of pile which are jointed to each other by devices capable of developing the same bending and tensile resistance as the main body of the pile. (Tomlinson, 1994)

The 'Hercules' pile has a hexagonal cross-section, and is shown in Figure 2.2. The precast concrete units are locked together by a steel bayonet-type joint to obtain the required bending and tensile resistance. (Tomlinson, 1994)

The Hercules piles are factory-made and are manufactured in standard lengths (in the UK two square and two hexagonal sizes are manufactured in standard lengths of 6.1m, 9.2m and 12.2m). A length is chosen for the initial driving, which is judged to be suitable for the shallowest predicted penetration in a given area. Additional lengths are locked on if deeper penetrations are necessary, or if very deep penetrations requiring multiples of the standard lengths are necessary. It is claimed that penetrations of up to 90m are possible. (Chellis, 1961 and Tomlinson, 1994)

Precast concrete piles (consist of units joined together by simple steel end plates with welded butt joints) are not always suitable for hard driving conditions, or for driving on to a sloping hard rock surface. Welds made in exposed site conditions with the units held in the leaders of a piling frame may not always be sound. If the welds break due to tension waves set up during driving or to bending caused by any deviation from alignment, the pile may break up into separate units with a complete loss of bearing capacity. This type of damage can occur with keyed or locked joints when the piles are driven heavily, for example to break through thin layers of dense gravel. The design of the joint is, in fact, a critical factor in the successful employment of these piles. Tests to determine bending, tension, and compression properties of a jointed section are made after subjecting the section to test driving when jointed to a pile section already seated on rock. Where hard driving conditions are anticipated, the jointed pile should be provided with a central hole for all preliminary test piles and for a proportion of the working piles. (Chellis, 1961 and Tomlinson, 1994)

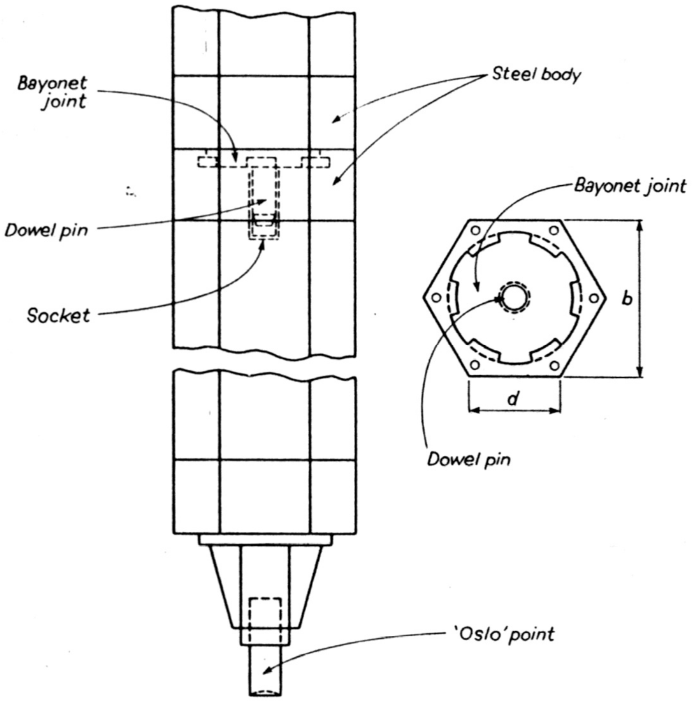


Figure 2.2: “Hercules” jointed precast concrete pile (Tomlinson, 1994)

2.1.4 Steel Piles

Being robust, light to handle, capable of carrying high compressive loads when driven on to a hard stratum, and capable of being driven hard to a deep penetration to reach a bearing stratum or to develop a high skin-frictional resistance, are the main advantages of steel piles. On the other hand, their cost per meter run is high compared with precast concrete piles. They can be designed as small displacement piles, which is advantageous in situations, where ground heave and lateral displacement must be avoided. They can be readily cut down and extended, where the level of the bearing stratum varies; also the head of a pile which buckles during driving can be cut down and re-trimmed for further driving. They have a good resilience and high resistance to buckling and bending forces. (Tomlinson, 1994)

Plain tubes, box-sections, H-sections, and tapered and fluted tubes (Mono-tubes) are types of steel piles. Hollow-section piles can be driven with open ends. If the base resistance must be eliminated when driving hollow-section piles to a deep penetration, the soil within the pile can be cleaned out by grabbing, by augers, by reverse water-circulation drilling, or by airlift. It is not always necessary to fill hollow-section piles with concrete. In undisturbed soil conditions, they should have an adequate resistance to corrosion during the working life of a structure, and the portion of the pile above the sea bed in marine structures. Concrete filling may be undesirable in marine structures, where resilience, rather than rigidity, is required to deal with bending and impact forces. (Prakash and Sharma, 1990 and Tomlinson, 1994, www.cflhd.gov)

If hollow-section piles are required to carry high compressive loads, they may be driven with a closed end to develop the necessary end-bearing resistance over the pile base area. If deep penetrations are required, they may be driven with open ends and with the interior of the pile closed by a stiffened steel plate bulkhead located at a predetermined height above the toe. An aperture should be provided in the bulkhead for the release of water, silt or soft clay trapped in the interior during driving. In some circumstances, the soil plug within the pile may develop the required base resistance. After driving is completed, concrete filling of light-gauge steel tubes is required, because the steel may be torn or buckled or may suffer corrosion losses. (Tomlinson, 1994 and www.cflhd.gov)

The facility of extending steel piles for driving to depths greater than predicted from soil investigation data, has already been mentioned. The practice of welding-on additional lengths of pile in the leaders of the piling frame is satisfactory for land structures, where the quality of welding may not be critical. A steel pile, supported by the soil, can continue to carry high compressive loads even though the weld is partly fractured by driving stresses. However, this practice is not desirable for marine structures, where the weld joining the extended pile may be above sea-bed level, in a zone subjected to high lateral forces and corrosive influences. Conditions are not conducive to first-class welding, when the extension pile is held in leaders or guides on a floating vessel, or on staging supported by piles swaying under the influence of waves and currents. It is preferable to do all welding on a prepared fabrication bed with the pile in a horizontal position, where it can be rotated in a covered welding station. The piles should be fabricated to cover the maximum predicted length and any surplus length cut off, rather than be initially of only medium length and then be extended. However, there are many situations, where in-situ welding of extensions cannot be avoided. The use of a stable jack-up platform, from which to install the piles is then advantageous. (Prakash and Sharma, 1990, Tomlinson, 1994 and www.cflhd.gov)

If steel tubular piles are required to be spliced below the ground surface and compressive loads only are carried, the 'Advance' purpose-made splicing devices can be used. The splicer consists of an external collar which is slipped on to the upper end of the pile section already driven and is held in position by an internal lug. The next length of pile is then entered into the collar and driven down. (Prakash and Sharma, 1990 and Tomlinson, 1994)

When soil has to be removed for subsequent placement of concrete, steel tubular piles are the preferred shape, since there are no corners from which the soil may be difficult to dislodge by the removing tools. They are also preferred for marine structures, where they can be fabricated and driven in large diameters to resist the lateral forces in deep-water structures. The circular shape is also advantageous in minimizing drag and oscillation from waves and currents. The hollow section of a tubular pile is also an advantage, when inspecting a closed-end pile for buckling. A light can be lowered down the pile and if it remains visible when lowered to the bottom, no deviation has occurred. If a large deviation is shown by complete or partial disappearance of the light, then measures can be taken to strengthen the buckled section by inserting a reinforcing cage and placing concrete. (Prakash and Sharma, 1990 and Tomlinson, 1994)

Concrete-filled steel tubular piles does not needed to be reinforced unless they are required to carry uplift or bending stresses, which would overstress a plain concrete section cast in the lighter gauges of steel. (Tomlinson, 1994)

Steel box piles are fabricated by welding together trough-section sheet piles, or specially-rolled trough plating. The types fabricated from sheet piles are useful for connection with sheet piling forming retaining walls, for example to form a wharf wall capable of carrying heavy compressive loads in addition to the normal earth pressure. However, if the piles rotate during driving there can be difficulty in making welded connections to the flats. Plain flat steel plates can also be welded together to form box piles of square or rectangular section. (Tomlinson, 1994)

MV piles are small square-section box piles ranging in size from 70mm square to 100mm square. They are driven with a shoe of larger overall dimensions, which forms an enlarged hole. This eliminates skin friction and enables the piles to be driven to the deep penetration required for their principal use as anchors to retaining walls. On reaching the design anchorage depth, a cement grout is injected to fill the annular space around the shaft. The grouted zone provides the necessary skin frictional resistance to enable them to perform as anchors. (Prakash and Sharma, 1990 and Tomlinson, 1994)

Due to having a small volume displacement, in situations where it is desired to avoid substantial ground heave or lateral displacement, H-section piles are suitable for driving in groups at close centers. They can resist hard driving and are useful for penetrating soils containing cemented layers and for punching into rock. Their small displacement makes them suitable for driving deeply into loose or medium dense sands without the 'tightening' of the ground that occurs with large displacement piles. (Prakash and Sharma, 1990 and Tomlinson, 1994)

H-piles cannot develop a high end-bearing resistance when terminated in soils or in weak or broken rocks, because they have relatively small cross-sectional area. In Germany and Russia it is frequently the practice to weld short H-scctions on to the flanges of the piles near their toes to form 'winged piles'. These provide an increased cross-sectional area in end bearing without appreciably reducting their penetrating ability. The bearing capacity of tubular piles can be increased by welding T-sections onto their outer periphery, when the increased capacity is provided by a combination of skin friction and end bearing on the T-sections. (Tomlinson, 1994)

A tendency to bend about its weak axis during driving is a disadvantage of the steel H-piles. The curvature may be sharp enough to cause failure of the pile in bending. A further complication arises, when H-piles are driven in groups to an end bearing on a dense cohesionless soil or weak rock. If the piles bend during driving so that they converge, there may be an excessive concentration of load at the toe and a failure in end bearing, when the group is loaded. (Prakash and Sharma, 1990 and Tomlinson, 1994)

By welding a steel angle or channel to the web of the pile, the curvature of H-piles can be measured. After driving, an inclinometer is lowered down the square-shaped duct to measure the deviation from the axis of the pile. (Tomlinson, 1994)

2.2 Driven-and-Cast-in-Place Displacement Piles

Driven-and-cast-in-place piles are installed by driving a heavy-section steel tube to the desired depth with its end closed. A reinforcing cage is next placed in a tube, which is filled with concrete. The tube is withdrawn while placing the concrete or after it has been placed. In other types of pile, thin steel shells or precast concrete shells are driven using an internal mandrel, and concrete, with or without reinforcement, is placed in the permanent shells after withdrawing the mandrel. Driven-and-cast-in-place piles have the principal advantage of being readily adjustable in length to suit the desired depth of penetration. Thus, in the withdrawable-tube types the tube is driven only to the depth required by the ground conditions. In the shell types, the length of the pile can be easily adjusted by adding or taking away the short units, which form the complete shell. The withdrawable-tube piles are the most economical type of pile for land structures. If the required penetration depth is within the capability of the piling rig to pull out the tube, and there are no restrictions on ground heave or vibrations, they can be installed more cheaply than any other type of driven or bored pile. They also have the advantage, which is not enjoyed by all types of shell pile, of allowing an enlarged base to be formed at the toe. (Tomlinson, 1994)

2.2.1 Withdrawable-Tube Types

The methods of installing these piles are basically the same. A piling rig consisting of a mast, leaders and winch on a track or roller-mounted frame is used to support and guide the withdrawable tube. The latter is of heavy wall section and its lower end is closed by an expendable steel plate. The tube is driven from the top by a simple drop hammer or by a diesel or vibrating hammer. On reaching the required toe level, the hammer is lifted off and a reinforcing cage is lowered down the full length of the tube. A highly workable self-compacting concrete is then placed in the tube through a hopper, followed by raising the tube by a hoist rope operated from the pile frame. The tube may be filled completely with concrete before it is lifted or it may be lifted in stages depending on the risks of the concrete jamming in the tube. The length of the pile is limited by the ability of the rig to pull out the drive tube. This restricts the length to about 20m to 30m. In the driven-and-cast-in-place pile, a full length reinforcing cage is always advisable. It acts as a useful tell-tale against possible breaks in the integrity of the pile shaft caused by arching and lifting of the concrete, as the tube is withdrawn. (Chellis, 1961 and Tomlinson, 1994)

The problem of inward squeezing of soft clays and peats or bulging of the shafts of piles from the pressure of fluid concrete is common to cast-in-place piles. A method of overcoming this problem is to use a permanent light gauge steel lining tube to the pile shaft. However, great care is needed in withdrawing the drive tube to prevent the permanent liner being lifted with the tube. Even a small amount of lifting can cause transverse cracks in the pile shaft of sufficient width to result in excessive settlement of the pile head under the working load. The problem is particularly difficult in long piles, when the flexible lining tube tends to snake and jam in the drive tube. Also, if piles are driven in large groups, ground heave can lift the lining tubes off their seating on the unlined portion of the shaft. Snaking and jamming of the permanent liner can be avoided by using spacers such as rings of sponge rubber. (Tomlinson, 1994)

In nearly all cases, the annulus left outside the permanent liner after pulling the drive tube will not close up. Thus, there will be no skin frictional resistance available on the lined portion. This can be advantageous because dragdown forces in the zone of highly compressible soils and fill materials will be greatly reduced. However, the ability of the pile shaft to carry the working load as a column without lateral support below the pile cap should be checked. (Chellis, 1961 and Tomlinson, 1994)

Allowable stresses on the shafts of these piles are influenced by the need to use easily workable self-compacting mixes with a slump in the range of 100mm to 175mm and to make allowances for possible defects in the concrete placed in unseen conditions. A cube compression strength in the range of 21N/mm^2 to 3()N/ mm^2 is usually adopted, BS 8004 limits the working stress of 25% of the 28-day cube strengths giving allowable stresses of $5N/mm^2$ to $7.5N/mm^2$. For these values, allowable loads for piles of various shaft diameters are given in Table2.1. (BS 8004, 1986)

Table 2.1: Nominal shaft diameter – Allowable working load (BS 8004, 1986 and Tomlinson, 1994)

Nominal shaft diameter (mm)	Allowable working load(kN)
300	350 to 500
350	450 to 700
400	600 to 900
450	800 to 1000
500	1 000 to 1400
600	1400 to 2 000

For the flow of concrete through bars in the reinforcing cage, the spacing of the bars should give sufficient space. Bars of 5mm diameter in the form of a spiral or flat steel hoops used for lateral reinforcement should not be spaced at centers closer than 100mm. (BS 8004, 1986 and Tomlinson, 1994)

The Vibrex pile employs a diesel or hydraulic hammer to drive the tube, which is closed at the end by a loose plate. A vibrating unit, which is clamped to the upper end of the tube is then employed to extract the tube after the concrete has been placed. A variation of the technique allows an enlarged base to be formed by using the hammer to drive out a charge of concrete at the lower end of the pile. The Vibrex pile is formed in shaft diameters of from 350 to 600mm and in lengths up to 38m.

The Fundex pile is a type of screwpile. A helically-screwed drill point is held by a bayonet joint to the lower end of the piling tube. The latter is then rotated by a hydraulic motor on the piling frame and at the same time forced down by hydraulic rams. On reaching founding level, a reinforcing cage and concrete are placed in the tube, which is then withdrawn. The Tubex pile also employs the screwed drill point, but the tubes are left in place for use in very soft clays when 'waisting' of the shaft must be avoided. The tube can be drilled down in short lengths, each length being welded to the one already in place. (Tomlinson, 1994)

The method used to compact the concrete in the shaft by alternate upward and downward blows of a hammer on the driving tube, is the specialty of the Vibro pile. The upward blow of the hammer operates on links attached to lugs on top of the tube.

This raises the tube and allows concrete to flow out. On the downward blow the concrete is compacted against the soil. The blows are made in rapid succession which keeps the concrete alive and prevents its jamming in the tube. (Chellis, 1961 and Tomlinson, 1994)

2.2.2 Shell Types

In preference to steel lining tubes, precast concrete sections can be lowered down the temporary drive tube and bedded onto a layer of cement mortar placed on the shoe. The space around the precast concrete sections can be filled with cement grout, which is injected as the drive tube is pulled out. The Positive pile is an example of the technique. (Tomlinson, 1994)

Types employing a metal shell, in general, consist of a permanent corrugated steel lining tube, which is locked onto a steel plate or precast concrete shoe. The lining tube and shoe can be driven down by a collapsible mandrel, which locks into the corrugations. Alternatively the shoe can be driven down by a temporary drive tube followed by placing the liner and locking it to the shoe, and then withdrawing the drive tube. The permanent liners are then filled with concrete and any necessary reinforcing steel. The feature of these piles is the provision for locking the lining tubes to the shoe, thus preventing uplift, when the drive tube or mandrel is pulled out. (BS 8004, 1986 and Tomlinson, 1994)

The West's Shell Pile incorporates precast concrete shell units, which are threaded onto an internal mandrel that carries a detachable precast concrete conical shoe at its lower end. The shells are joined by circumferential steel bands which are painted internally with bitumen to make a waterproof joint. On reaching founding level any surplus shells are removed, the mandrel is withdrawn and a reinforcing cage is lowered down. Then concrete is placed to the required level. The driving head is designed so that the main hammer blow is delivered to the mandrel, but a cushioned blow is also made on the shells to ensure that they move down with the mandrel. The precast concrete shells are reinforced with polypropylene fibres to increase their resistance to impact during driving. The West's pile has the advantage of being readily adjustable in length by adding or removing the 914mm long shell units as required. (Prakash, and Sharma, 1990 and Tomlinson, 1994)

In situation giving rise to ground, the shells of piles already driven are liable to be lifted while driving neighboring piles. This may result in excessive settlement when the piles are loaded. Pre-boring may be necessary to overcome this difficulty. Also lifting or damage to the shells may occur, if the mandrel deviates from a true line while driving past obstructions or onto a sloping bedrock surface. (Tomlinson, 1994)

2.3 Replacement Piles

During installation of replacement piles, at first the soil is removed by a drilling process, and then the pile is constructed by placing concrete or some other structural element in the drilled hole. The simplest form of construction consists of drilling an unlined hole and filling it with concrete. However, complications may arise, such as difficult ground conditions, the presence of ground water, or restricted access. Such complications have led to the development of specialist piling plant for drilling holes and handling lining tubes, but unlike the driven-and-cast-in-place piles, very few proprietary piling systems have been promoted. This is because the specialist drilling machines are available on sale or hire to any organization. The resulting pile, as formed in the ground, is more or less the same no matter which machine, or method of using the machine, is employed. There have been proprietary systems, such as the Prestcore pile, which incorporates precast units installed in the pile borehole, but these methods are largely out of date.(USACE, 2005, Tomlinson, 1994 and Coduto, 2005)

There are two principal types of replacement pile, which are bored-and-cast-in-place piles, and drilled-in tubular, including caisson, piles. (BS 8004, 1986 and Tomlinson, 1994)

2.3.1 Bored-and-Cast-in-Place Piles

An unlined hole can be drilled in stable ground by hand or mechanical auger. If reinforcement is required, a light cage is then placed in the hole, followed by the concrete. In loose or water-bearing soils and in broken rocks, casing is needed to support the sides of the borehole. In stiff to hard clays and in weak rocks an enlarged base can be formed to increase the end-bearing resistance of the piles. The enlargement is formed by a rotating expanding tool, or by hand excavation in piles having a large shaft diameter. A sufficient cover of stable cohesive soil must be left over the top of the enlargement in order to avoid a run of loose or weak soil into the unlined cavity. (USACE, 2005 and Tomlinson, 1994)

Bored piles, which are drilled by hand auger are limited in diameter to about 355mm and in depth to about 5m. They can be used for light buildings such as dwelling houses. However, even for these light structures hand methods are used only in situations, where mechanical augers are not available. (Tomlinson, 1994)

Bored piles, which are drilled by mechanical spiral-plate or bucket augers or by grabbing rigs, can drill piles with a shaft diameter up to 7.3m, but it is usual to limit the maximum size to 2.13m diameter to suit the auger plant generally available. Boreholes up to 120m deep are possible with the larger rotary auger machines. Under-reaming tools can form enlarged bases in stable soils up to 7.3m in diameter. The size of enlarged bases formed by hand excavation is limited only by the practical considerations of supporting the sloping sides of the base. It is also possible to drive headings by hand between adjacent piles and to fill them with concrete to form a system of deep beam foundations. Rotary drilling equipment consisting of drill heads with multiple rock roller bits have been manufactured for drilling shafts up to 8m in diameter. (BS 8004, 1986 and Tomlinson, 1994)

For the concern of economy and the need to develop skin friction on the shaft, it is the usual practice to withdraw the casing during or after placing the concrete. This procedure requires care and conscientious workmanship by the operatives in order to prevent the concrete being lifted by the casing, and thus resulting in voids in the shaft or inclusions of collapsed soil. (Tomlinson, 1994)

In bored-and-cast-in-place piles, reinforcement is not always needed unless uplift loads are to be carried. Reinforcement may also be needed in the upper part of the shaft to resist bending moments caused by any eccentricity in the application of the load, or by bending moments transmitted from the ground beams. However, it is often a wise precaution to use a full length reinforcing cage in piles, where temporary support by casing is required over the whole pile depth. The cage acts as a warning against the concrete lifting as the casing is extracted. (USACE, 2005 and Tomlinson, 1994)

Continuous flight auger or auger injected piles (CFA piles) are installed by drilling with a rotary continuous-flight auger to the required depth. In stable ground above the water table, the auger is then removed and a sand-cement grout is pumped through a flexible pressure hose. This type of pile is referred to as cast-in-place. In unstable or water-bearing soils, a flight auger is used with a hollow stem closed at the bottom by a plug. After reaching the final level, a fairly fluid cement-sand mortar, or a concrete made with coarse aggregate not larger than 20mm, is pumped down the stem and fills the void as the auger is slowly withdrawn with or without rotation. Thus the walls of the borehole are continually supported by the spiral flights and the soil within them. Reinforcing steel can be pushed into the fluid mortar to a depth of about 12m. (BS 8004, 1986, Tomlinson, 1994 and www.pileinfo.com)

In water-bearing and unstable soils, the CFA pile has considerable advantages over the conventional bored pile. Temporary casing is not needed, and the problems of concreting underwater are avoided. The drilling operations are very quiet and vibrations are very low making the method suitable for urban locations. However, in spite of these considerable advantages the CFA pile depends for its integrity and load bearing capacity, as much as any other in-situ type of pile, on strict control of workmanship. This is particularly required where a high proportion of the load is carried in end bearing. In conventional bored piles the quality of the soil or rock at the toe can be checked by examining drill cuttings or by probing, but In the case of the CFA pile a suitably resistant layer can only be assumed to be present by the increase in torque on the drill stem. The drill cuttings from toe level do not reach the surface until concrete or mortar injection has been completed. There are also doubts as to whether or not the injected material has flowed-out to a sufficient extent to cover the whole drilled area at the pile toe. For this reason it is advisable either to assume a base diameter smaller than that of the shaft or to adopt a conservative value for the allowable end bearing pressure. (BS 8004, 1986, Tomlinson, 1994 and www.pileinfo.com)

The CFA pile is best matched for ground conditions, where the majority of the working load is carried by skin friction. Care is also needed to adjust the rate of withdrawing the auger to the rate of injecting the mortar or concrete. If the auger is raised too quickly, the surrounding soil can collapse into cavities formed around the auger stem. The rate of raising the auger can be controlled by instruments monitoring the pressure and quantity of injected material. Because of the problems described above, it is desirable to specify that the CFA piling rigs should be provided with instruments to measure concrete flow rates and the torque on the drill stem, or the revolution/penetration rate of the auger. (Tomlinson, 1994 and www.pileinfo.com)

2.3.2 Drilled-in Tubular Piles

The use of a tube with a medium to thick wall is the essential feature of the drilled-in tubular pile. This tube is capable of being rotated into the ground to the desired level and is left permanently in the ground with or without an in-filling of concrete. Soil is removed from within the tube as it is rotated down, by various methods including grabbing, augering, and reverse circulation. The tube can be continuously rotated by a hydraulically-powered rotary table or be given a semi-rotary motion by means of a casing oscillator. (BS 8004, 1986 and Tomlinson, 1994)

For penetrating ground containing boulders or other massive obstructions, the drilled-in tubular pile is a useful method. It is also used for founding in hard formations, where a rock socket capable of resisting uplift and lateral forces can be obtained by drilling the tubes into the rock. In this respect the drilled-in tubular pile is a good type for forming berthing structures for large ships. These structures have to resist high lateral and uplift loads for which a thick-walled tube is advantageous. In rock formations the resistance to these loads is provided by injecting a cement grout to fill the annulus between the outside of the tube and the rock forming the socket. (BS 8004, 1986 and Tomlinson, 1994)

3 PROBLEMS IN PILE CONSTRUCTION

To be concerned is necessary at all stages of design and construction with any civil engineering work. Especially, workmanship and supervision is of major importance. Construction or piled foundations is a specialist activity, requiring substantial knowledge as well as dependable workmanship, the more so as the completed element can rarely be inspected for defects. If curative work to piling is required at a later date, it can be extremely time-consuming and expensive, moreover sometimes impossible. (Flaming, 1992)

There are three main pie types, which are in widespread use: driven displacement piles, driven and cast-in-place (cast-in-situ) piles and bored (replacement) piles. Each system has particular advantages and disadvantages and selection of the correct pile type requires considerable experience. (BS 8004, 1986 and Tomlinson, 1994)

3.1 Driven Displacement Piles

The piles, which take the form of prefabricated e1ements driven into the ground, are preformed driven displacement piles. Steel, concrete and timber are commonly used materials for the manufacture of these piles. This type of pile has the advantage that the pile element can be thoroughly inspected before driving. (Flaming, 1992 and Prakash and Sharma, 1990)

3.1.1 Design and Manufacture

The piles should be designed to resist handling and driving stresses. Stresses arising from these impermanent conditions are generally more severe than the stresses in service. (Flaming, 1992)

Throughout the manufacture of concrete piles, sufficient curing of the piles is essential. The formation of shrinkage cracks can lead to corrosion of the reinforcement. Similarly, premature lifting of the pile can cause cracking. Such defects are usually easy to detect. On the other hand, it is rather subjective to decide when a crack is too wide. Piles with major cracks owing to extreme bending moments should be condemned. Experience and personal decision are necessary when assessing the severity of cracking; the aggressiveness of the environment should be considered as well. It is not necessary to throw out all cracked piles, and a series of transverse hair cracks, that has a width of 0.15 to 0.3 mm may be considered acceptable. (BS 8110-1, 1997 and Flaming, 1992)

The design and manufacture of steel H-piles is quite simple. The pile may be fitted with a shoe or a rock point, if heavy driving situation are expected. Preparation of tubular or hexagonal piles may involve a significant amount of welding. As far as possible, the weld should be handled in the fabrication shop under the control of suitably skilled personnel. Pile driving will subject the welds to high repetitive stresses and any defects can straightforwardly lead to the propagation of brittle fractures. It is important that the pile sections are correctly aligned, as lack of straightness will lead problem on driving and result in high bending stresses being locked into the pile. (Flaming, 1992 and Coduto, 2005)

3.1.2 Installation of Driven Displacement Piles

While driving concrete piles, the most important damage is generally caused by deformation of the pile head. Such damage is frequently the result of persistence on driving to a predetermined length. The driving of piles is usually controlled by a specified final set, and driving to a length may involve the acceptance of method such as jetting or preboring. (Flaming, 1992 and USACE, 2005)

To be able to penetrate to thin dense layers within the soil profile, heavy driving of the pile may be necessary. The piles selected should be capable of accepting this heavy driving, indeed it may also be necessary to consider test piles to check the performance of the pile in this respect. Steel piles will accept heavy driving as they infrequently break. However, severe deformation of the pile head may occur. Damage to the pile toe may also occur, but this is not obvious unless piles are extracted. (Flaming, 1992 and USACE, 2005)

Convinient piling plant selection can reduce the probable damages caused by pile driving. Selection of the correct hammer weight is particularly important. In general, a heavy hammer with a low drop results in lower impact stresses than a light hammer with a higher fall, giving the same rated energy per blow. The purpose of the dolly is to reduce the peak impact stresses arising from the hammer blow, and selection of the correct packing materials requires considerable experience. It should be noted that the elasticity of the packing changes with constant use, and most commonly used materials harden noticeably. Selection of packing for driving steel piles is not so important. However, the head covering should be designed to suit the particular section driven, to reduce deformation of the pile head. Driving precast shell piles requires special equipment as the energy supplied by the hammer has to be distributed between the mandrel and the shells. Normally, such piles would be installed by a specialist subcontractor experienced in the use of this type of piling. (Flaming, 1992 and Togrol, 1970)

It is not usually practicable to examine the toe of a driven pile, except in the case of tubular piles. In the case that wondering whether or not the pile toe penetrate the various strata, a rock point should be fitted to help penetration. Problems can be expected, while piles are being driven through the soil profile containing boulders or other obstructions. In such soil conditions, concrete piles may be broken and steel H-section piles deformed or split into apart. (Flaming, 1992)

It is important to preserve the designed geometry of the pile group. Positive control is possible only at the ground surface, and piling frames should be set on stable foundations. If guide trestles are used, they should be of considerable construction, and positive guides for the piles should be fitted. It is not possible to correct variation once pile driving has started. Piles, particularly flexible piles, may drift off line considerably below ground level. An extreme case of an H-pile turning through 180^0 has been reported. However, a greater danger is that piles will crash in a closely spaced group. (Chellis, 1961 and Flaming, 1992)

3.2 Replacement (Bored) Piles

3.2.1 Excavation of the Pile Bore

In convenient soil conditions like stiff clays, for a short guide length bored piles can be constructed without casing. Under less suitable conditions temporary or permanent casings or bentonite drilling fluid may be used to support the pile. Excavation of the soil is performed by augers, buckets, or a variety of grabs. Continuous flight auger equipment has recently become popular, which can place piles in many different soil conditions without the use of any casing. (Chellis, 1961)

3.2.2.1 Overbreak.

In cohesionless soils, the formation of cavities or overbreak outside the nominal diameter of the pile is a significant problem especially below the water table (Figure 3.1). Under such conditions, it is necessary to use a temporary steel casing and the cutting edge of the casing is driven below the base of the advancing bore. If the casing is not at the base of the hole, slumping or washing of soil into the bore can occur, with following problems on concreting the pile. The cavities may or may not be filled with water. Even with well-controlled drilling, the flow of water inside the base of the advancing bore is possible. This seriously loosens the surrounding soil and causes loss of skin friction. (Flaming 1992 and Prakash and Sharma, 1990)

In very clean sands or rounded gravels, a soil arch can form around the borehole, even in dry conditions. Under these circumstances collapse of soil into the bore will form large irregular cavities. However, the problem is easily overcome by using temporary casing. (Flaming, 1992)

The problem occur during drilling under water can be solved by maintaining an excess water head in the casing or by using bentonite drilling mud. Alternatively the boreholes may be permanently cased. (Tomlinson, 1994))

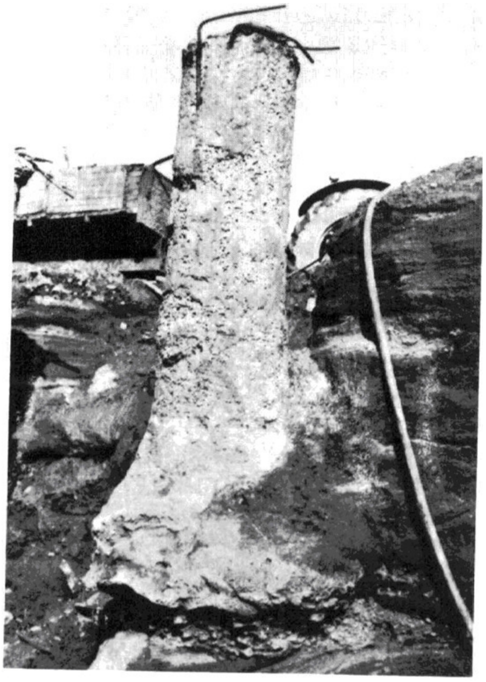


Figure 3.1: Bulbous projection on pile shaft caused by overbreak. (Flaning, 1992)

In boulder clay, severe boring problems can occur as a result of overbreak in pile shafts. In such heterogeneous materials, it is very difficult to estimate the maximum size of any rock fragments from site investigation data. It is frequently advantageous to select large-diameter piles, which are drilled with heavy equipment to allow the largest boulder to be removed from the bore. The alternatives of chiselling to break up boulders or to redrill the pile at another position can be expensive and may result in serious delays in the piling program. (Flaming, 1992)

3.2.2.2 Base of Boreholes

To be able to improve the end-bearing capacity, it is necessary to make sure that the base of the bore is clean and undisturbed. In clay soils, the boreholes can be cleaned with special tools, but in stiff fissured clays with sand partings and perhaps with water seepage, blocks of material may fall into the borehole. Underreamed piles are particularly liable to this problem. (Flaming, 1992 and Tomlinson, 1994)

The base of the boreholes should be cleaned to remove any debris and also should be examined before concreting. High-intensity spot lamps are now available, which may be used for this purpose. The importance of cleaning the base of boreholes cannot be overstressed: cases of pile failure have been assigned to neglecting this step. (Flaming, 1992)

In granular soils, loosening of the base is almost impossible to avoid, even when drilling under water or bentonite. For piles founded in such soil deposits, the allowable capacity of the pile is calculated by reducing the effect of end bearing. Special techniques may be used to consolidate the loosened soil. These usually consist of placing a stone fill pack at the base of the borehole, followed by high pressure grouting after the pile is concreted. The main purpose of the grouted packing is to preload the pile and reverse the direction of the shaft friction. The overall load/settlement performance of the pile is improved, but the technique has little effect on the ultimate bearing capacity of the pile. Sand or silt may accumulate at the base of a pile before concreting, leading to the effect shown in Figure 3.2. The fines can take place because of local seepages into the borehole, or because of sediment falling from the drilling water or mud. Cleaning the base in water filled hole is very important to obtain the adequate behaviour of the pile, which gets the bearing capacity from a rock socket. In such circumstances final cleaning may be made by the use of an air-lift or by using the reversed-circulation drilling technique. (Flaming, 1992 and USACE, 2005)

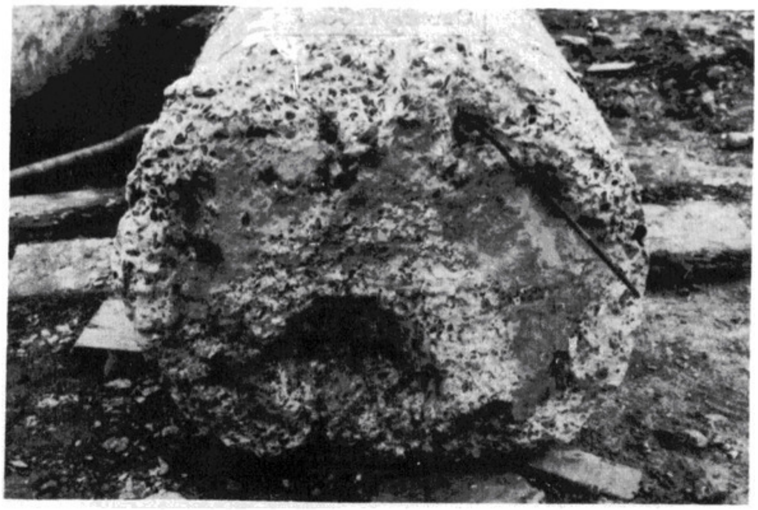


Figure 3.2: Toe of small-diameter pile filled with silt prior to concreting. (Flaning. 1992)

3.2.2.3 Effects of Water in Boreholes

The entrance of small quantities of water into the pile bores is not a serious problem. However, an increase of water at the base of the hole can cause problems on concreting. Small amount of water can result in segregation of the concrete at the toe of a pile. Figure 3.3 shows the defects found in practice arising from this cause. (Flaming, 1992)

Depth of the probable accumulated water at the base of the borehole should be measured. For accumulations more than a few centimeters deep, the water should be removed before concreting, or alternatively a tremie pipe may be used to place the concrete. Some other methods can also be applied, such as placing a dry batch of concrete, which are not usually successful. Water will cause softening of the surface in clay soils, and for this reason piles should be concreted as soon as possible after boring. If concreting is delayed, slumping of softened soil from the shaft may occur and low values of adhesion will take place. Unfortunately, in practice, due to volume difference between single pile shaft and concrete mixer, concreting of pile should wait until enough piles are bored to complete a necessary volume of concrete to fill a concrete mixer.

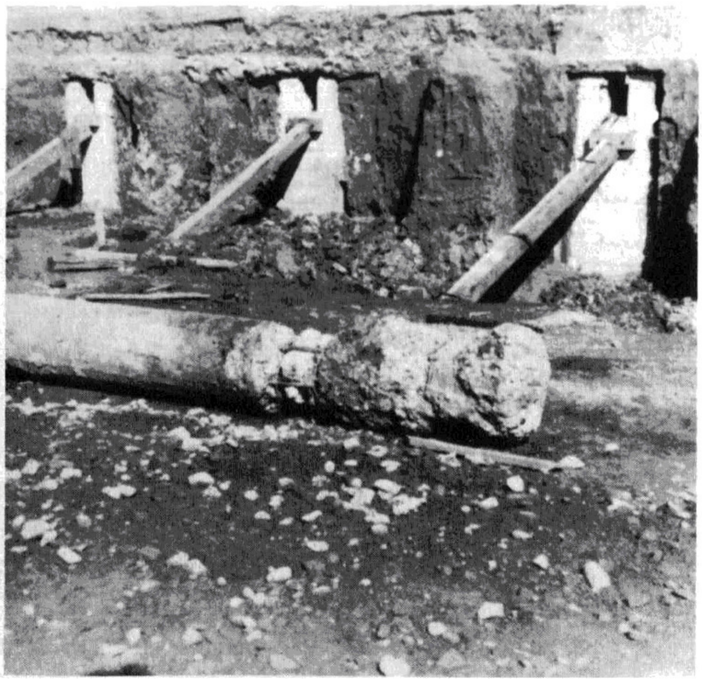


Figure 3.3: Toe of pile after concreting with water in bore. (Flaning, 1992)

3.2.2 Concreting the Pile

Concreting cast-in-place piles is a special operation requiring considerable skill and the correct design of the concrete mix. More problems are caused by using a mix, which is too stiff than by using one with too high a slump. (Flaming, 1992)

3.2.2.1 Quality of the Concrete

Method of placing concrete in bored piles has severe limits depending on the type of concrete that will be used. In general, it is preferable to ensure the pile shaft is free from defects by using a highly workable mix, than to specify one with a low water/cement ratio. Using concrete with too low workability is a major cause of defects in bored pile construction. (Flaming, 1992 and Togrol, 1970)

It is not practicable to vibrate concrete in a pile shaft; because the energy of the falling stream of concrete will be enough to place itself. The free-fall method of placing concrete will result in the segregation of the concrete if the mix is unsuitable.

In general terms concrete, which is suitable for constructing cast-in-place piles should be free flowing; the mixes are designed to be self-compacting. The slump test is quite adequate to evaluate the former property. However, cohesiveness or resistance to segregation is difficult to define. If available, rounded aggregates should be chosen and high sand content is accepted. Plasticizers may be used to improve the workability of the mix with advantage. Experiments with super-plasticizers have shown that these additives are not useful owing to their high cost and the limited time for which the effect occurs. Similarly, air-entraining agents are not effective, except for concrete to be placed by tremie pipe, as the entrained air is lost on impact of the falling concrete. (USACE, 2005)

The strength of the concrete is difficult to settle with the high workability required. Very few cases of worsening of cast-in-place piles have been reported in the literature, and no cases of failure are known. The site investigation should reveal the presence of aggressive ground conditions. In aggressive conditions, it is recommended that the cover to the steel is increased, and the outer 50 mm or so of the shaft may be considered as a protective coating (BS 8004, 1986). Sulphate attack, the most common problem, may be overcome by using sulphate-resisting cements. Extremely aggressive conditions may be encountered in sabka deposits of the Middle East, with highly saline groundwaters. In these situations, it is useful to case the pile with a permanent plastic sleeve, mainly to prevent chloride attack and steel corrosion in that part of a pile, which is near to or above the ground water level. The combined presence of water, chlorides and oxygen can lead to rapid steel corrosion. (Prakash and Sharma, 1990)

3.2.2.2 Placing Concrete

Concrete is generally placed in the bore by letting it to fall from the ground surface. It is essential to make preparations to avoid the concrete impinging on either the reinforcement or on the sides of the hole. The concreting process should be continuous and be completed uninterrupted. (Chellis,, 1961)

If extended delays between batches occur, water and laitance bleeding from the previously placed concrete may cause a weak point in the pile shaft. This requires attention to the need for good mix design. For boreholes filled with water or drilling mud, the tremie method of concreting is a necessity. Underwater, bottom-discharge concreting skips are not normally accepted because of the difficulty of ensuring complete discharge at each withdrawal of the skip. (Flaming, 1992)

3.2.2.3 Extracting Temporary Casing

Later than concreting, if delays occur, removal of the momentary casing can make problems particularly and partial separation of the pile shaft may result, as shown in Figure 3.4. Friction between the concrete and the casing is obviously aggravated by the use of dirty or dented casings. With unsuitable mix designs, it is known for the casing to be withdrawn still filled with concrete. Difficulties arise in pulling the casing if a low-slump concrete is used, or if the aggregate is particularly angular. Insistence on vibrating the concrete can lead to arching within the casing. (Flaming, 1992)

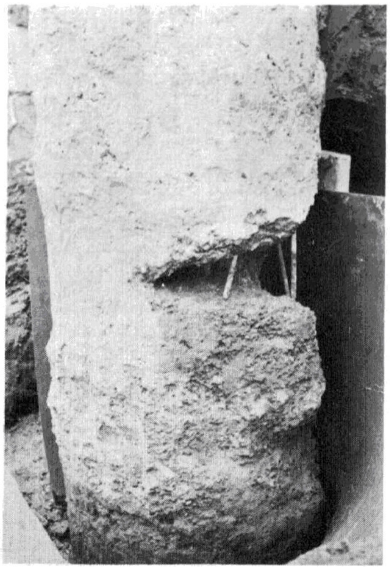


Figure 3.4: Separation of pile shaft caused by removal of casing. (Flaning, 1992)

Bleeding of mixing water from the joints of screwed casings may cause local consolidation of the concrete. This causes disturbance of the reinforcement cage and difficulty in extracting the casing. (Tomlinson, 1994)

Cavities outside the casing can lead to serious defects in the pile, particularly if they are waterfilled. Figure 3.5 shows the mechanism of the formation of the defect. (Flaming, 1992) If airfilled voids exist outside the casing, defects are not structurally serious. Normally the concrete in the shaft would slump to fill the cavity resulting in bulbous projections. (Prakash and Sharma, 1990)

There is a danger that debris can be dragged into the pile shaft, particularly if large cavities exist. Dry voids may form, if thin gravel layers exist or if piles are formed in fill. The formation of such voids can be reduced by the use of a vibrator to insert the pile casing. Obviously, if there is any doubt that satisfactory pile shafts will be constructed, thin permanent casings should be used to support the soil. (Chellis, 1961)

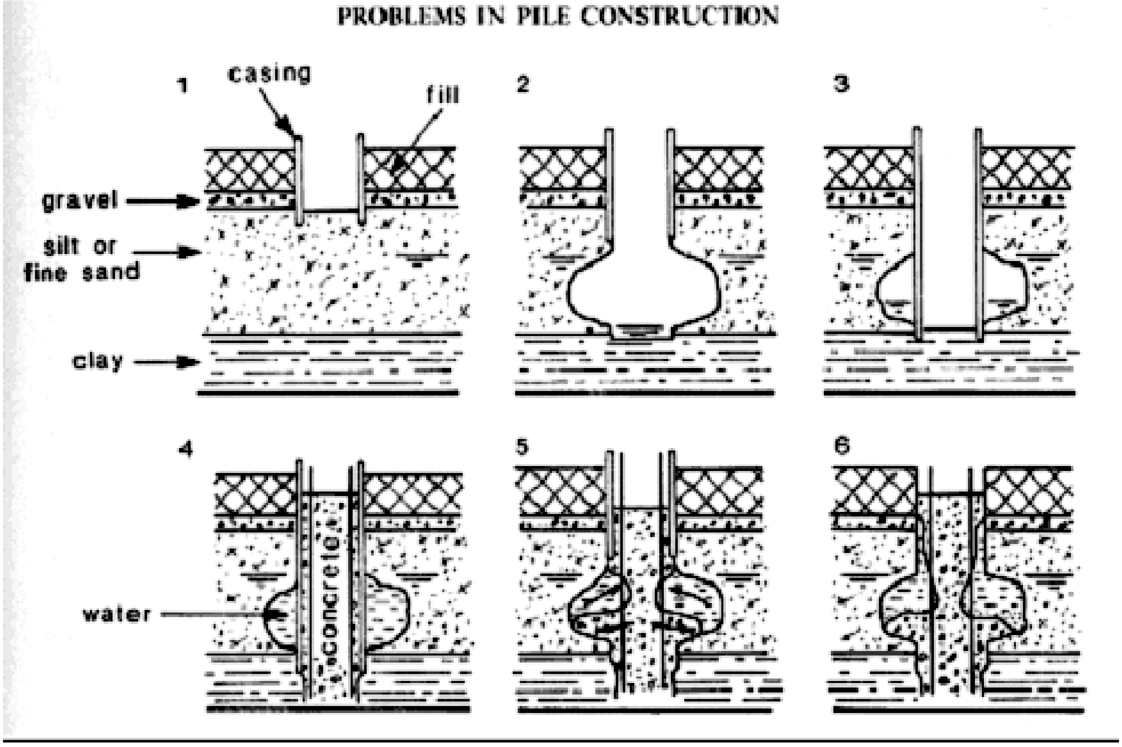


Figure 3.5: Schematic presentation of formation of water-filled cavities. (Flaning, 1992)

Wherever slumping of concrete into voids has occurred, topping up the pile shaft with fresh material frequently leads to problems. Debris may be dislodged by removal of the casing or by the placement of additional concrete. (Flaming, 1992)

3.2.2.4 Problems in Soft Ground

Different kinds of defects may arise, when forming cast-in-place piles in very soft alluvium exhibiting undrained shear strengths less than 15 or 20 kNjm^2. These defects relate to the lateral pressure exerted by the fresh concrete. The lateral pressure of the concrete can easily exceed the passive resistance of soft soils, and bulges on the pile shaft will occur. Such defects may be detected by a close check on the volume of concrete used or by sonic integrity testing. (Flaming, 1992)

The lateral pressure of the concrete may be low near the head of the pile, and additional reductions in pressure can be caused by friction as the casing is removed. In such situations, it is possible for soft soil to squeeze the pile section, leading to local waisting of the concrete. Again, the likelihood of this type of defect may be overcome by casting concrete to ground level or by the use of permanent light steel casings. (Chellis, 1961)

3.2.2.5 Effects of Groundwater

The major effect of groundwater seepage, mainly in soils containing thin bands of silt, is to cause local slumping of the bore. The resulting debris may not be completely removed before concreting, or the material may become included in the body of the concrete. (Flaming, 1992)

Leaching of cement from fresh concrete is most unusual as high water velocities are required. Occasional cases of damage have been reported in made ground. A more common effect is shown in Figure 3.6, where the pile cut-off level is below the groundwater level. For this situation to develop, the water pressure would have to exceed twice the head of the concrete at the base of the casing. The problem may easily be overcome by overfilling the pile or by filling the casing above the pile with a little sand and topping it up with water. If groundwater causes instability of the pile bore, drilling using a permanent casing or bentonite slurry should be resorted to.

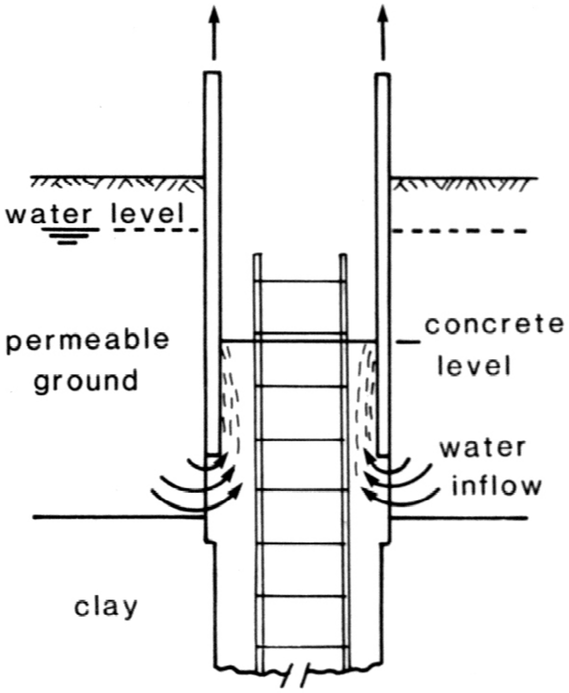


Figure 3.6: Effect of water inflow at head of pile (Flaning, 1992)

3.2.3 Pile Reinforcement

Unsuitable detailing of the reinforcing cage is a common problem with the construction of cast-in-place piles. In some instances the spacing of the bars may be so close as to completely retain the concrete. Even if this is not so, free flow of the concrete may be sufficiently impeded to prevent complete placement of the steel. This problem is related to the workability of the concrete and is more likely to occur with low-slump concrete. A few large-diameter bars are to be preferred, with a minimum spacing of at least 100 mm. (Coduto, 2005)

The lateral outward pressure on the main steel caused by the flowing concrete may be considerable, and hoop steel should be sufficient to restrain the main steel. Displacement of the reinforcement due to this cause has been observed; the defect usually occurs by the downward movement of the projecting bars at the pile head. Attempts to support a slumped cage by the insertion of individual free bars, which are pushed into the wet concrete can result in the defect. (Chellis, 1961)

If the pile shaft contains a large amount of steel, widely-spaced welded steel bands may be used instead of the normal helical binding. Such welded bands will also prevent twisting of a heavy cage, which can occur when setting the steel or concreting of the shaft. (Flaming. 1992)

Experience has shown that reinforcement cages should extend at least 1 m below the level of any temporary casing. This will prevent movement of the cage caused by drag as the casing is extracted.

3.2.4 Piles Constructed with the Aid of Drilling Mud

Excavation of bored piles with the aid of a bentonite-based drilling mud may result in particular defects associated with the method of construction. Slurries with a concentration of bentonite clay in the range 4 to 6% are normally used to support the borehole. The fresh slurry can become contaminated with soil arising from the excavation. This results in an increase in density and viscosity of the drilling fluid. The borehole is basically stabilized by the pressure exerted by the drilling mud. If this pressure is less than the active soil pressure, it will result the failure of the sides of the bore. Such failures are most likely to occur near the head of the pile, especially if the water table is high. Short guide casings should be used and the level of the drilling mud should be maintained at least 1 m, and normally 1.5 m above the groundwater level to avoid problems of instability. Excessive viscosity of the drilling fluid may occur with heavy contamination of soil. In this situation rapid movement of the drilling bucket may cause a 'piston effect'. On withdrawal of the bucket, significant reductions in pressure can occur and local collapse of the borehole may result. Clearly control of the viscosity of the drilling fluid should be enforced to avoid this problem and digging buckets should be provided with a fluid by-pass. (Flaming, 1992 and Tomlinson, 1994)

Cleaning the pile base should be carried out with extra care, as it is not possible to inspect the base. Cleaning may be effected by means of a special bucket or with an air lift, and loosening of the base should be avoided. Fine sand and silt suspended in the bentonite slurry will gradually settle to the base of the pile, for this reason final cleaning of the hole should be carried out shortly before concreting. For placing concrete under a bentonite drilling mud, the tremie method is used and it is important that a suitably designed mix is adopted. In other respects, concreting bentonite-filled holes gives rise to problems similar to those of concreting open bores. (Flaming, 1992)

It has been found that the process of forming a pile under bentonite suspension does not materially reduce the shaft friction. Normally a very thin and weak filtercake layer will form on the wall of a bore in any reasonably fine permeable soil, and the rising column of concrete will largely remove this from the hole. However, if the bore is left filled with slurry for long periods, or if the bentonite suspension is heavily contaminated with soil particles, this may accumulates an appreciable thickness of filtercake. If the filtercake is strong enough to resist abrasion during the placing of concrete, it will cause a significant reduction in shaft friction on the completed pile. In clay soils, the concrete becomes contaminated with debris and intermixed with slurry. It is therefore necessary to overfill the piles sufficiently to ensure sound concrete is present at pile cut-off level. In practice, 1 m of overfilling has been found to be satisfactory. (Chellis, 1961)

3.3 Driven Cast-in-Place Piles

There are many systems of constructing driven cast-in-place piles. In those systems, which incorporate a permanent lining of steel or concrete, defects in placing concrete are likely to be few, because the lining system is watertight. Such piles are very similar in behaviour to driven precast piles. Piling systems which use a driven temporary casing, basically to form a hole, may also suffer from some of the defects associated with bored cast-in-place piles. One of the major advantages of driven cast-in-place piles is that the driving records provide a valuable check on the stiffness of the bearing stratum. (Flaming, 1992)

In some instances, casings are driven by an internal drop hammer acting on a gravel or dry concrete plug at the base of the casing. If the dry mix has the wrong consistency, driving may not cause the plug to lock in the casing. In this situation, any end plate will be broken, and ingress of soil and water into the casing cannot be prevented and the tube may have to be withdrawn and re-driven using a correct mix. Heavy driving with an internal hammer may result in bulging of the casing, or in extreme cases, splitting of the steel if it is of inadequate thickness. (Prakash and Sharma 1990)

With some piling systems, the internal hammer is used to compact low-slump concrete as the casing is withdrawn. With this system the hammer may catch on the reinforcement and damage it, or over-vigorous compaction may cause the cage to spread. It is highly desirable to use a robust steel cage and to check the level of the reinforcement of such piles before and after concreting.

Concrete placed in driven cast-in-place piles is liable to suffer many of the problems associated with bored piles. With this type of pile it is recommended that concrete should be cast to ground level in order to maintain a full pressure head and prevent contamination of the pile. (Flaming, 1992)

4 NONDESTRUCTIVE PILE TESTING METHODS

Nondestructive test methods can be classified into two main groups;

- Borehole Nondestructive Test (NDT) Methods
- Surface nondestructive test methods

4.1 Borehole Nondestructive Test (NDT) Methods

These NDT methods require access from a borehole drilled close to the foundations.

4.1.1 Parallel Seismic (PS)

The Parallel Seismic (PS) method is a borehole test method for determining depths of foundations. The method can also detect major anomalies within a foundation as well as provide the surrounding soil velocity profile. The method requires the installation of cased borehole close to the foundation being tested. The method can be used when the foundation tops are not accessible or when the piles are too long and slender (such as H piles or driven piles) to be testable by sonic echo techniques.

Basic Concept: The Parallel Seismic (PS) method involves hammer impacts at any part of the exposed structure that is connected to the foundation (or impacting the foundation itself, if accessible). A hydrophone or a three-component geophone located in a nearby borehole records the compressional and/or shear waves traveling down the foundation. Therefore, the PS test requires drilling a 5- to 10-cm-diameter hole as close as possible to the foundation being tested (preferably within 1.5 m). The borehole should extend at least 3 to 5 m below the expected bottom of the foundation. If hydrophones are used, the hole must be cased, capped at the bottom, and the casing and hole filled with water. For geophone use, the hole must usually be cased and grouted to prevent the soil from caving in during testing.

PS tests can be performed on concrete, wood, masonry, and steel foundations. Some portion of the structure that is connected to the foundation must be exposed for the hammer impacts. (www.cebtp-solen.com)

Data Acquisition: The field setup for Parallel Seismic tests is shown in figure 4.1. In a PS test, a hammer strikes the structure, and the response of the foundation is monitored by a hydrophone or a geophone receiver placed in the borehole. A signal analyzer records the hammer input and the receiver output. The receiver is first lowered to the bottom of the hole, and a measurement is taken. Then, the receiver is moved up 30 or 60 cm, and the second measurement is made. This process is continued until the receiver has reached the top of the boring.

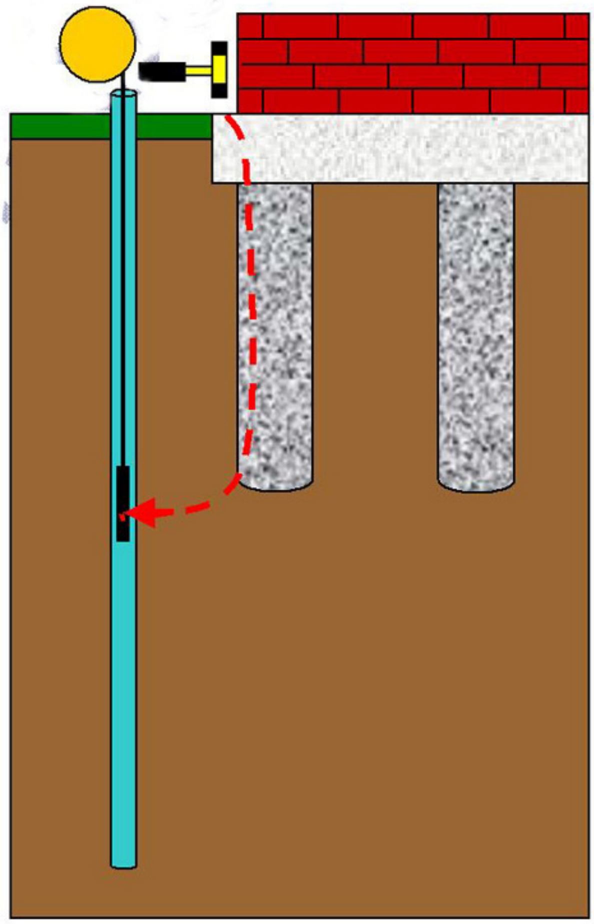


Figure 4.1: Parallel Seismic survey setup (www.testconsult.co.uk)

Advantages: The Parallel Seismic method is more accurate and more versatile than other nondestructive surface techniques for determining unknown foundation depths. The accuracy of the method depends on the variability of the velocity of the surrounding soil and the spacing between the borehole and the foundation element. Depths are normally determined with 95% accuracy or better.

Limitations: A borehole is needed for Parallel Seismic tests, which adds to the cost of the investigation (unless borings are also required for other geotechnical purposes). The borehole should be within 1.5 m of the foundation, which sometimes cannot be achieved. Note that for very uniform soils (such as saturated sands), a successful test can be performed with up to 4.5 to 6 m spacing between the source and the borehole. As the borehole moves away from the foundation, interpretation of the PS data becomes more difficult, and the uncertainty in the tip depth determination becomes greater. (www.cflhd.gov)

4.1.2 Induction Field (IF)

Induction Field (IF) method is used for the determination of the unknown depth of steel or continuously reinforced concrete piles.

Basic Concept: This is an electrical method that relies on detecting the magnetic field in in response to an oscillating current impressed into a steel pile. In order for this method to work, the pile must, therefore, contain electrically conductive materials. For reinforced concrete piles, this usually implies that reinforcing rebar extends along its full length.

A sensor is placed down a drillhole located close to the pile and detects the changing magnetic field strength. This sensor could be a magnetic field sensor or a coil. Along the length of the pile, the magnetic field strength will be relatively strong. However, the magnetic field strength will be significantly diminished at levels in the drillhole beneath the bottom of the pile to a residual conductivity value of the soil or bedrock. This change in the magnetic field strength is used to determine the depth of the pile.

Data Acquisition: An electrical contact must be made to the pile in question when conducting an Induction Field survey. Another electrode must be placed some distance from the pile. This can be another pile or simply an electrode placed in the ground. Oscillating current is then made to flow between these two electrodes. Figure 4.2 shows the layout of the pile, borehole and electrodes.

Advantages: The Induction Field method is a proven technology for the determination of unknown depth of piles containing electrically conductive material, such as rebar. If the reinforcement is continuous in a concrete foundation, it can be used to detect the presence of piles underneath a footing.

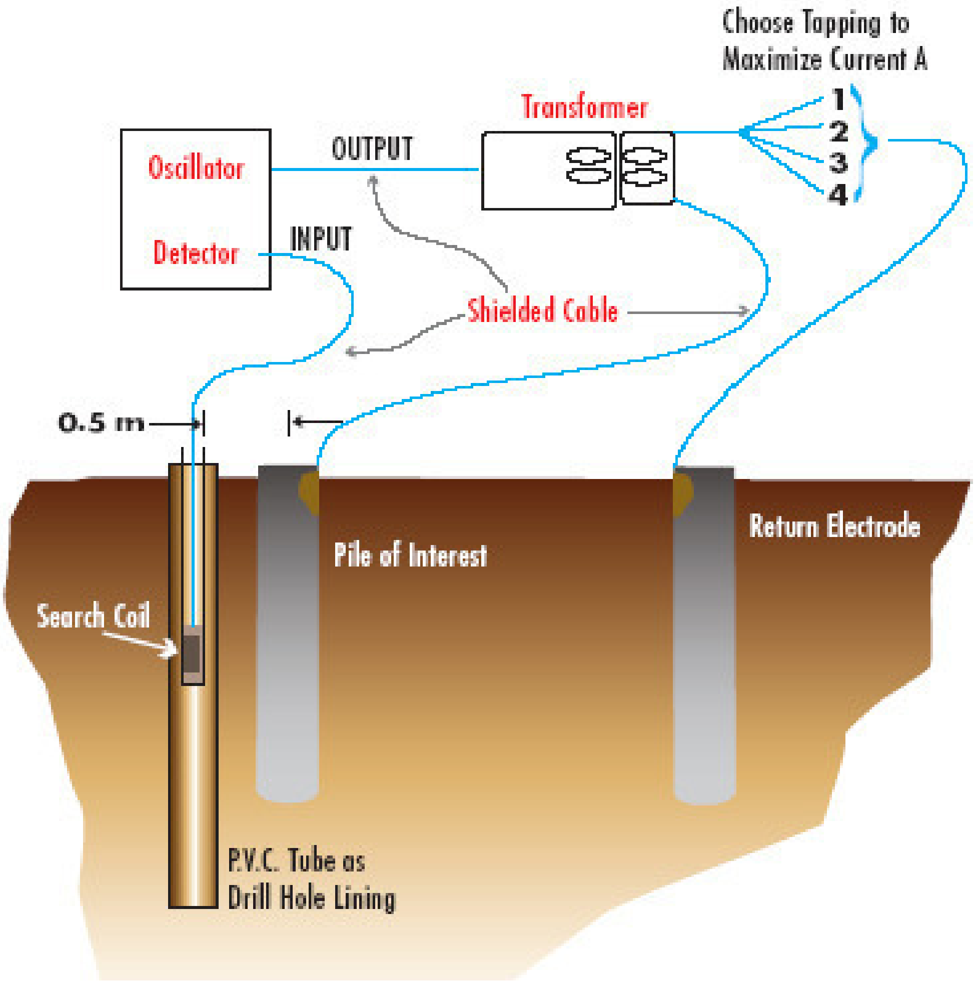


Figure 4.2: Induction Field method setup (www.olsonengineering.com)

Limitations: The interpretation of data is complicated by the existence of conductive materials in the bridge structure and the surrounding ground (including the water-table). Probably the most restrictive requirement is that there must be metal (usually reinforcing rebar) inside the pile, and it must extend continuously along the length of the pile. In addition, an electrical contact with the metal must be possible near the top of the pile. A PVC cased borehole is required. No signal would be received through a steel-cased borehole.

4.1.3 Borehole Logging Methods

Two borehole geophysical logging methods, magnetic logging and electromagnetic induction logging methods, can be used for the determination of unknown depth of steel of reinforced concrete piles.

No information was found on the use of these methods for measuring pile depths. However, it seems clear that these methods should work if the conditions are appropriate. In view of their simplicity and ease of use, it is included herein as methods that could be tested.

A borehole magnetometer could be used to find the depth of piles if they contain reinforcing rebar along the length of the pile. The reinforcing rebar will have induced magnetization due to the influence of the Earth's magnetic field and will radiate a secondary magnetic field. A magnetometer will respond to both the Earth's magnetic field and the secondary field. Beneath the toe of the pile, the secondary field will rapidly diminish resulting in a decrease in the magnetic field strength. A magnetometer in a nearby drill hole will detect this magnetic field change from which the toe of the pile can be induced.

Alternatively, borehole electrical methods, such as induction logging, can be used. In the induction logging method, an AC current is transmitted into the ground by the source coil and another coil is used for receiving the returning signal. The transmitted AC current generates a time-varying primary magnetic field, which induces eddy currents in the conductive ground or steel reinforcing rebar. These eddy currents set up secondary magnetic fields, which induces a voltage in the receiving coil. The magnitude of the received current can be used to determine the pile toe.

These two geophysical logging methods were not found in any of the references and are under investigation by Blackhawk GeoServices.

4.1.4 Dynamic Foundation Response

The Dynamic Foundation Response uses the resonant frequencies of structures to differentiate foundation types. The vibration response of a bridge substructure will exhibit lower resonant frequency responses when excited for a shallow foundation versus the comparatively higher resonant frequency response of a deep foundation system.

<u>Basic Concept:</u> The method is unproven for this use in bridges, but is based on the dynamic analysis theory for vibration design of foundations (soil dynamics) and geotechnical analyses of foundations subjected to earthquake loading.

<u>Data Acquisition:</u> A hammer with a built-in dynamic force transducer is used as the vibration source. A triaxial block of seismic accelerometers records the resulting

signals. Typically, a bridge is excited at five to six locations, and the triaxial response is measured at five to six locations giving rise to 25 to 36 source-receiver combinations. The bridges are impacted in the vertical and horizontal directions to excite these modes as well as rocking modes along the frame of the substructures. This type of testing is known as modal testing; when the impulse force is measured and the resultant vibration response is measured, the transfer function can be calculated as in the Impulse Response test.

Limitations: In tests, the Dynamic Foundation Response method showed some sensitivity in the response of the foundation as a function of depth and existence of piles, particularly for vertical vibrations. However, in practice, the bridges generally could not be excited with the 5.5 kg hammer at frequencies comparable to the natural frequencies identified in the theoretical modeling. More research is needed to explore different sources to generate the very low frequencies required for the experimental modal tests and then to perform curve fitting techniques to experimental data to be able to extract the mode shapes from the experimental data to compare with the theoretical mode shapes.

Practical tests showed that a 5.5 kg hammer was not sufficient to generate the required waves in bridges at the required frequencies. However, the method showed some response of the foundation as a function of depth and existence of piles. This was particularly true for vertical vibrations. It is possible that this method may be appropriate for monitoring over time, where changes in the response could be detected. However, no references were found indicating that this is currently done.

4.1.5 Borehole Radar

Borehole Radar uses a borehole ground penetrating radar (GPR) antenna to obtain reflection echoes from a foundation for the determination of unknown depth and geometry of foundation.

Data Acquisition: In borehole radar, an antenna transmits radar energy into the surrounding rock and soil, and a receiver then records reflections that occur as the radar signals encounter and reflect from interfaces with different dielectric properties. The method is very similar to the borehole sonic method, where seismic waves are used rather than electromagnetic waves.

Borehole radar can be used in reflection mode or in cross-hole tomography mode. The radar measurements are either directional or omnidirectional, depending on the type of equipment and antennas. Only the reflection mode will be discussed in this web manual.

Radar uses radio waves with frequencies varying generally between 10 and 2,000 MHz. These waves are influenced primarily by the dielectric properties of the medium through which they are traveling and the electrical conductivity of the medium. Highly conductive materials attenuate the radar signals and limit its depth of penetration. Although the lower frequencies penetrate more than higher frequencies, they have less resolution.

For radar frequencies of 100 MHz, penetration varies from 10 to 40 m in resistive rocks. In conductive, clay-rich rocks, penetration will be less than 5 m. Figure 4.3 shows the borehole radar system.

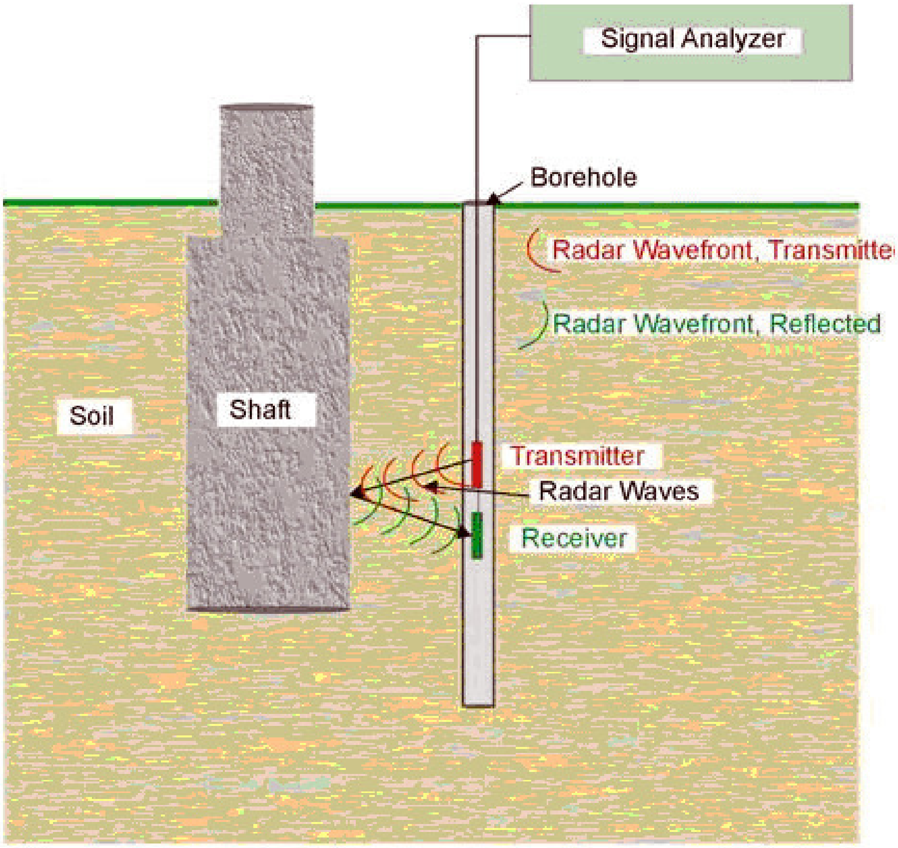


Figure 4.3: Borehole Radar system (www.ndt.net)

<u>Limitations:</u> Borehole radar requires a PVC-cased borehole; the method will not work if the hole is steel-cased. The depth of penetration is significantly influenced

by the electrical conductivity of the rocks and soil surrounding the borehole, which may not be known before the radar survey is completed. Penetration up to about 10 m may be achieved in resistive conditions. In conductive materials, since the penetration of the GPR signal will be limited, getting the borehole as close to the pile as possible will be advantageous.

4.1.6 Borehole Sonic

This method is similar to Sonar and is based on using borehole seismic sources and geophones to obtain reflection echoes from a foundation for the determination of unknown depth and geometry of foundations.

Basic Concept: This method is a borehole equivalent of the surface reflection seismic method. It requires a borehole close to the pile into which a sonde is lowered containing a seismic source and a seismic wave detector. The requirement for the method to work is that there is a seismic impedance contrast between the pile (for example, concrete) and the soil into which the borehole is drilled. It is likely that this will frequently be the case since the velocity of seismic waves in soil is typically much lower than those in concrete.

Data Acquisition: The borehole sonic surveys are conducted using a single borehole utilizing borehole logging sondes with low frequency sources or alternatively using separate boreholes for the source and receiver that are held at the same measurement depths. The survey is conducted by taking measurements at various depths within the borehole. In many logging methods, readings are taken when the sonde is ascending.

Limitations: Seismic waves in soil are attenuated significantly with distance and may also be dispersive, thus limiting the higher frequency content of the signal. In addition, the waves are reflected from a curved surface (the pile), which may provide less returned energy than a plane surface as is common in surface seismic reflection methods. To this date, this method is not proven for the determination of the unknown foundation depths.

4.1.7 Cross-borehole Seismic Tomography

Two- and three-dimensional tomography is used for the high resolution imaging of the subsurface between boreholes.

Basic Concept: Tomography is an inversion procedure that provides for two- or three-dimensional (2-D and 3-D) velocity (and/or attenuation) images between boreholes from the observation of transmitted first arrival energy.

Data Acquisition: Tomography data collection involves scanning the region of interest with many combinations of source and receiver depth locations, similar to medical CAT-SCAN. Typical field operation consists of holding a string of receivers (geophones or hydrophones) at the bottom of one borehole and moving the source systematically in the opposite borehole from bottom to top. The receiver string is then moved to the next depth location and the test procedure is repeat until all possible source-receiver combinations are incorporated.

Advantages: Tomography provides high-resolution two-dimensional area or three-dimensional volumetric imaging of target zones for immediate engineering remediation. Tomography can then be used in before and after surveys for monitoring effectiveness of remediation. Tomography can also be used in before and after surveys for monitoring fluid injections between test holes or for assessing the effectiveness of soil improvement techniques.

Attenuation tomography can be used for the delineating fracture zones. Wave equation processing can be used for a high-resolution imaging of the reflection events in the data including those outside and below the area between the boreholes.

Limitations: Tomography is data-intensive and specialized 3-D analyses software is required for true three-dimensional imaging. Artifacts can be present due to limited ray coverage near the image boundaries.

4.2 Surface Nondestructive Test (NDT) Methods

As mentioned above, surface methods are applied from the accessible surface of the foundation element. Therefore, the foundation structure is used directly as the medium for the transmission of the acoustic energy.

4.2.1 Sonic Integrity Testing

Sonic integrity test (SIT) is performed to evaluate the integrity and determine the length of deep foundations. This method can be used to detect defects, soil inclusions and pile necking, diameter increases (bulbing) as well as approximate pile lengths.

SIT test is performed on drilled shafts and driven piles (concrete or timber) or auger-cast piles. This test can also be performed on shallow wall structures such as an abutment or a wall pier of a bridge, provided the top of the wall is accessible.

Basic Concept: The Sonic integrity test requires a measurement of the travel time of seismic waves (time domain), and the Impulse Response method uses spectral analysis (frequency domain) for interpretation.

In SIT, the reflection of compressional waves from the bottom of the tested structural element or from a discontinuity such as a crack or a soil intrusion is measured. The generated wave from an impulse hammer travels down a shaft or a pile until a change in acoustic impedance (depends on velocity, density, and changes in diameter) is encountered, where the wave reflects back and is recorded by a receiver placed next to the impact point.

Data Acquisition: For drilled shafts and piles, the best results from SIT test is obtained if the top of the drilled shaft or the pile is exposed for receiver attachment and hammer hitting, If the top is not exposed, then the SIT test is performed on the side. This requires at least the upper 30 to 60 cm of the shaft to be exposed (see figure 4.4b). For wall-like shallow structures, the top of an abutment or a pier should be exposed for sonic integrity testing (see figure 4.4c). In these cases where the superstructure is in place, the SIT data becomes more difficult to interpret because of the many reflecting boundaries, and two or more receivers should be used to track reflections (please refer to the Ultraseismic Method).

In a SIT test, a hammer strikes the foundation top, and a receiver monitors the response of the foundation. A digital signal analyzer records the hammer input and the receiver output.

Advantages: This is a quick and economical test method used mostly in columnar shaped foundations without access tubes. Any defects can be found early with minimal delays to construction.

Limitations: The SIT method works best for free-standing columnar-shaped foundations, such as piles and drilled shafts, without any structure on top. Typically, Sonic Integrity Test are performed on shafts or piles of length-to-diameter ratios of up to 20:1. Higher ratios (30:1) are possible in softer soils. The method can only detect large defects with cross-sectional area changes greater than 5%.

A toe reflection is not possible if the pile is socketed in bedrock of similar stiffness (or acoustic impedance) as concrete. If the pile is embedded in very stiff soils, penetration may be limited up to 8 m. For the softer soils, echoes can be observed from piles of up to 80 m in length. This method cannot be used for steel H-piles. (See Section 5 for detailed information about SIT)

4.2.2 Bending Waves

This method uses flexural (bending) waves, rather than the compressional waves used in the SIT method to determine integrity and unknown depth of deep foundations. It is limited to applications on rod-like deep foundations such as timber piles, concrete piles, and drilled shafts that extend above the ground or water surface.

The Bending wave method has been used experimentally on timber piles, but will also work on other, more slender members. The method has been used on timber piles up to 18.3 m in length. The key feature of this method is that only a horizontal blow is required, which is easy to apply to the side of a substructure.

Basic Concept: This method uses the propagation of flexural or bending waves in piles that are highly dispersive in nature. The bending wave velocity decreases with increasing wavelength, with most of the velocity decrease occurring at wavelengths that are longer than the pile diameter. These longer waves propagate as flexural or bending wave energy.

Correspondingly, as wavelengths become shorter than the diameter of a pile, the bending wave velocity limit is approximately that of the surface (Rayleigh) wave velocity, and this wave energy propagates as surface waves. Compressive waves are also dispersive in piles, but in a different way that, in practice, results in a velocity decrease only when a deep foundation has a low length-to-diameter ratio of about 2:1 or less, which is uncommon for deep foundations.

The classical wave equation, often called the traditional wave solution, does not account for dispersion. It is an ideal mathematical description of the behavior of the pulse, with its solution implying that a wave will travel through a rod without affecting the shape of the rod. This is the difference with dispersive signal analysis; dispersive analysis of the wave data extracts a selected group of frequencies. These frequencies are then analyzed for the individual time required to travel to the tip of the pile and back or from their tops to the location of an area of damage such as a void, break, soil intrusion, or material deterioration.

Data Acquisition: The Bending Wave method involves horizontally impacting the pile to generate flexural or bending waves that travel up and down the pile as illustrated in figure 4.4. The receivers are in a plane with the hammer blow on the same side of the pile. The equipment consists of a recording oscilloscope or dynamic signal analyzer, small to large hammers or other impact sources ranging from rubber to hard plastic to steel, and cushioning materials to protect and dampen the blow to the timber pile for metal-tipped hammers. Two accelerometers (G1 and G2) are used to measure the initial bending wave arrivals and subsequent reflections. The bending wave propagation is monitored by two horizontal accelerometer receivers mounted on the same side of the pile as the impact (see figure 4.4).

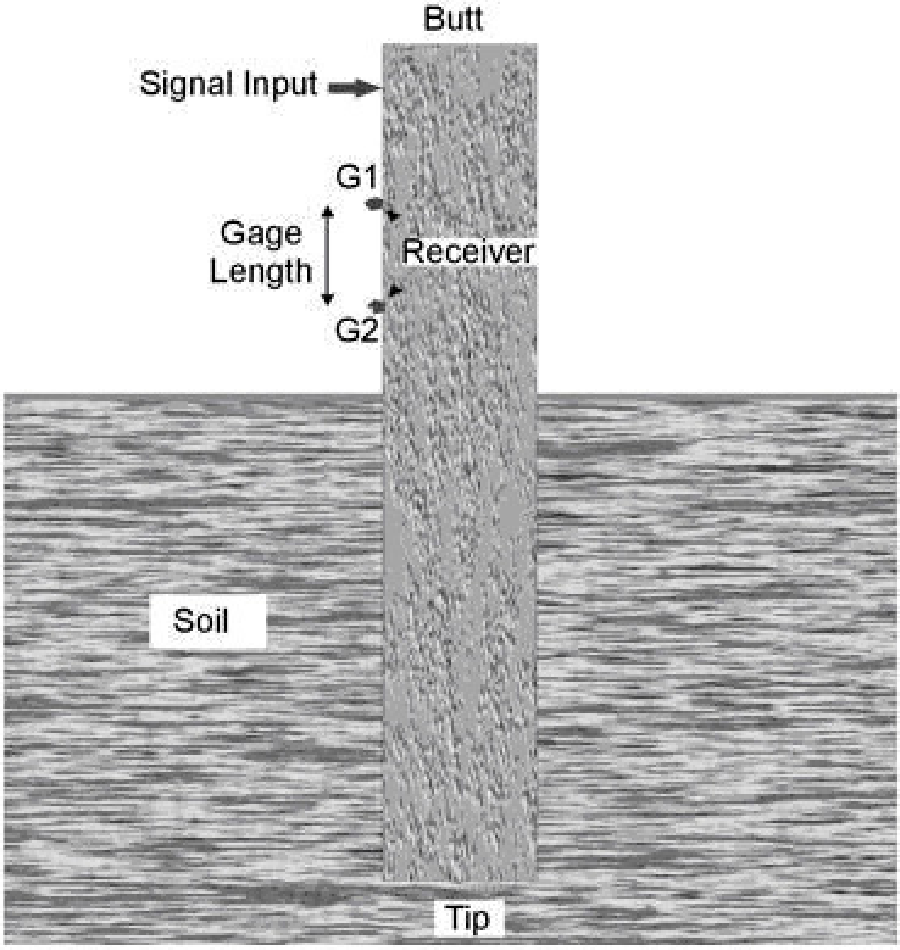


Figure 4.4: Field setup for the Bending Wave method for piles (www.cflhd.gov)

Advantages: This method provides a quick and economical test that does not require access tubes inside the foundation or boreholes outside.

Limitations: Theoretical studies have shown that for a 12-m-long, 1-m-diameter concrete shaft, depth predictions may not be able to be made for depths greater than 5 m due to the high attenuation associated with flexural waves as compared to compressional waves traveling down a rod. Stiff soil layers can result in apparent short pile lengths being predicted. Reflection events from top of the grade may be present in the data records resulting in false interpretation of the data. This method cannot be used for steel H-piles.

4.2.3 Ultraseismic

The Ultraseismic (US) method is performed to evaluate the integrity and determine the length of shallow and deep foundations. US tests can be performed on drilled shafts and driven or auger-cast piles. The tests can also be performed on shallow wall-shaped substructures such as an abutment or a wall pier of a bridge provided at least 1.5 to 1.8 m of the side of the structural element is exposed for testing. The method is particularly useful in testing abutments and wall piers of bridges because of the relatively large exposed areas available for testing.

The Ultraseismic tests can be performed on concrete, masonry, stone, and wood foundations. Steel pile foundations can also be tested, but damping of the energy in this case is much greater than that of concrete and wood due to the large surface areas and small cross-sectional areas of steel piles.

Ultraseismic method was developed by Mr. Frank Jalinoos as part of NCHRP 21-5 research program in response to the difficulty encountered in interpreting the SIT and Bending wave methods from complex structures (such as a bridge) where many reflecting boundaries are present. This method is an adaptation of multi-channel seismic reflection method to bounded engineered structures.

Basic Concept: The Ultraseismic method uses multi-channel, three-component (vertical and two perpendicular horizontal receivers, i.e., triaxial receiver) recording acoustic data followed by computer processing techniques adapted from seismic exploration methods. Seismograph records are typically collected by using impulse hammers as the source, and accelerometers as receivers that are mounted on the surface or side of the accessible bridge substructure at intervals of 30 cm or less. The bridge substructure element is used as the medium for transmission of the seismic energy. Four wave modes of longitudinal (compressional) and torsional (shear) body waves as well as flexural (bending) and Rayleigh surface waves can be recorded by this method. Seismic processing can greatly enhance data quality by identifying and clarifying reflection events that are from the foundation bottom and minimizing the effects of undesired wave reflections from the foundation top and attached beams. For concrete bridge elements, useful wave frequencies up to 4 to 5 kHz are commonly recorded.

This method can be used in two modes: Ultraseismic Vertical Profiling (VP), presented below, and Horizontal Profiling (HP).

<u>Data Acquisition</u>: The Vertical Profiling test geometry is presented in figure 4.5 and shows the accelerometers and impact point. The impact point can be located either at the top or the bottom of the receiver line. Vertical impacts to the substructure are comparatively rich in compressional wave energy, although more flexural/Rayleigh (surface) wave energy is generated. Horizontal impacts are rich in flexural wave energy when the impacts generate wavelengths that are longer than the thickness of the substructure element. Impacts that generate wavelengths shorter than the thickness will be rich in Rayleigh wave energy. The VP accelerometer array is useful for differentiating downgoing events from the upgoing events based on their acteristic time moveout, and accurately measure their velocity. A VP accelerometer array is also used to tie reflection events from the bottom to a corresponding horizon in an HP section. For a medium with a bounded geometry, such as a bridge column, four types of stress waves are generated that include longitudinal (compressional), torsional (shear), surface (Rayleigh), and flexural (bending) waves. In longitudinal vibration, each element of the column extends and contracts along the direction of wave motion that is along the column axis. In torsional vibration, each transverse section of the column remains in its own plane and rotates about its center. Finally, in flexural vibration, the axis of the column moves laterally in a direction perpendicular to the axis of the column. Each wave type can independently provide information about the depth of the foundation or the presence of significant flaws within the bridge substructure. However, practically, longitudinal (P-wave, compressinal)

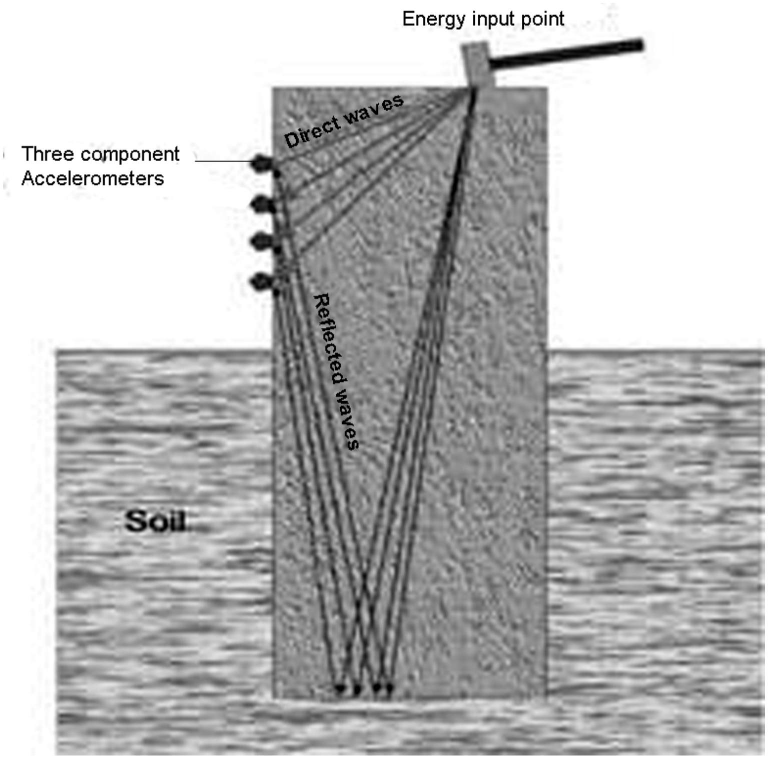


Figure 4.5: Ultraseismic test method and vertical profiling test geometry (www.cflhd.gov)

and flexural (bending) waves are much easier to generate on bridge substructures than torsional waves. Consequently, compressional and flexural wave energy were generated by orienting impacts to substructures vertically and horizontally, respectively.

The Ultraseismic Horizontal Profiling Method (HP), presented in figure 4.6, was developed for potential use on massive abutment and wall substructure elements and dam like structures, which typically have greater widths of top or side surfaces. This permits access to a line of receivers to be placed at a common elevation. The HP method uses the same basic equipment as the VP test, but since the receivers are at the same elevation, reflection events from footing bottoms should have the same arrival time in the seismic records.

<u>Advantages:</u> This method can be used to obtain two-dimensional reflection images from complex structures, such as bridges, buildings, and dams. It uses well-proven processing techniques developed in the seismic exploration method. Multiple-channel recording allows for differentiation of bottom echoes from other complex wave modes far more reliably than single-channel Sonic/Pulse Echo or Bending

wave methods. Ultraseismic tests can determine the depth of foundations with an accuracy of about 95%.

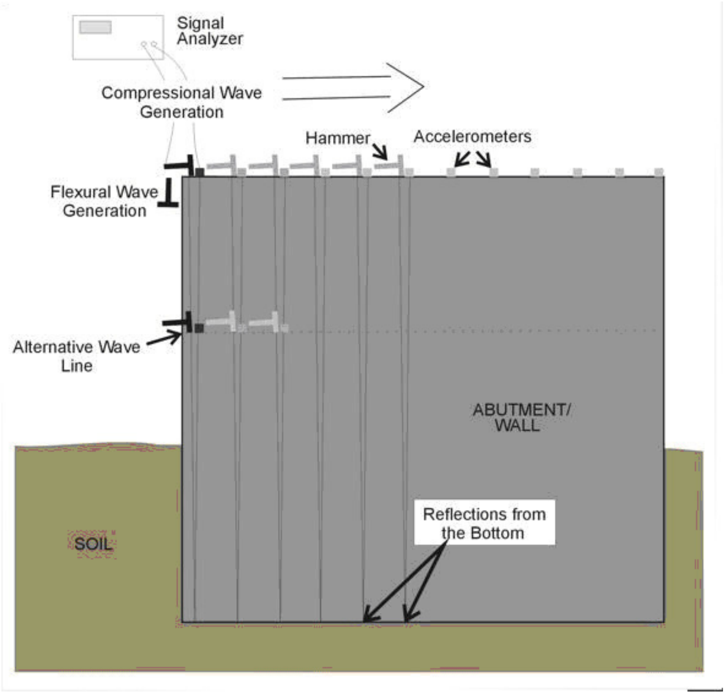


Figure 4.6: Ultraseismic test method and horizontal profiling test geometry (www.cflhd.gov)

<u>Limitations:</u> The Ultraseismic method requires at least 1.5 to 1.8 m of the structural member to be exposed, which is not always possible. For very deep foundations, echoes from the bottom may not be obtained because of the attenuation of energy in the surrounding soil. The Ultraseismic method is not capable of determining the depths of buried piles, especially steel H-piles, beneath a buried pile cap.

4.2.4 Seismic Wave Reflection Survey

<u>Basic Concept:</u> In this method, the seismic survey involved setting up shot points on one side of the pier and two horizontal component geophones on the other side of the pier. The toe of the pier will diffract seismic waves passing under it. The depth of the pier can be found from the position of the diffractions on the seismic records.

<u>Data Acquisition:</u> Reflection seismic data are usually considered to image a point midway between the shot and the receiver. This point is called the Common Depth Point (CDP). Seismic traces that occur at a CDP are usually summed together to form a single trace. For each shot, the geophones record the ground surface motion in two orthogonal directions. Repeating a number of hammer blows at the same

location allows the data recorded at the geophones to be stacked, thus improving the signal-to-noise ratio.

Advantages and Limitations: Because seismic pulses have a finite wavelength and hence bandwidth, the pulses "see" the bottoms of the piles as a zone that is a fraction of the length of the waves. It would take very high-frequency energy to see the precise locations of the bottoms of the piles. Having a finite wavelength produces some uncertainty about the depth of the pile, with shorter wavelengths providing less uncertainty. Generally, for field conditions, a depth resolution to about one-half of one wavelength can be expected. If the peak frequency is about 550 Hz, and the velocity of the waves is about 500 m/sec, then the wavelength is about 0.9 m. Thus, the resolution will be about 0.45 m.

4.2.5 Transient Forced Vibration Survey

Data Acquisition: For this survey, geophones are mounted on the vertical column that extends above the ground surface. A hammer is used to strike the column in a horizontal direction, producing shear waves; the shear wave data are recorded by geophones. This process is repeated by impacting the opposite side of the column. This allows the compressional wave component of the signal to be removed from the total signal, thereby isolating just the shear wave signal.

Advantages and Limitations: It is not possible to interpret the data using simple reflection times. The autocorrelation function is needed to identify major periodicities. The method sees the average of pile depths with multiple piers and cannot resolve the number of individual piles or their individual lengths

5 A SPECIAL APPLICATION OF SURFACE NONDESTRUCTIVE TESTING METHOD SONIC INTEGRITY TESTING

Main discontinuities or defaults in foundations which may cause the failure of the pile shaft is detected by the help of low strain integrity tests. Graphs of velocity-time and force-time are presented collectively for the duration of a SIT test, to rapidly and reliably judge the quality of a foundation. Results can be obtained in both the time or frequency domain.

5.1 Theoretical Background

A disturbance (modeled as an incident wave), is introduced to the pile, when a hammer loads a foundation pile during driving or testing. On account of discontinuities in the pile and interactions with the surrounding soil, reflected waves, which are traveling in the opposite direction to the incident wave, are also introduced. The incident and reflected waves, and any following reflections, interfere all the way through the length of the pile. This process can be compared numerically to one-dimensional wave theory. (Klingmuller, 1996)

5.1.1 One-Dimensional Wave Theory

As a preface to wave theory, consider the case of a cylindrical bar, not subject to any internal damping or soil interaction as shown in Figure 5.1. The bar has a cross sectional area *A,* a mass density ρ, and modulus of elasticity *E.* A time-dependent force *F(t)* is acting to the bar at the top as a load. (TNO, 2003)

Therefore, from Newton's 2nd Law *(F* = m *a).* the equation of motion for an infinitesimal piece of bar, *dx,* is given by

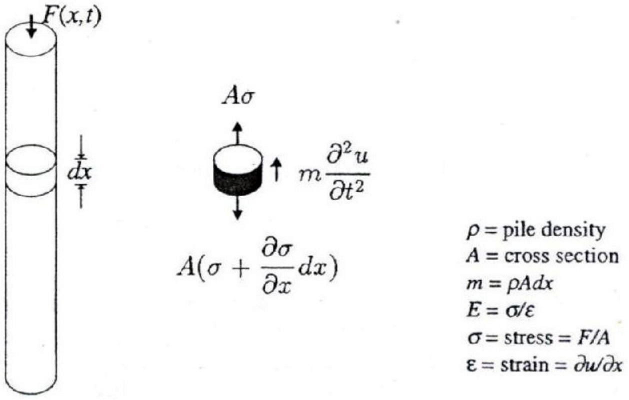


Figure 5.1: Cylindrical bar loaded at one end (TNO, 2003)

$$-\frac{\partial F}{\partial z}dz = (\rho A dz)\frac{\partial^2 u}{\partial t^2} \qquad (5.1)$$

where F : the axial force in the bar (compression = positive)

x : the Lagrangian coordinate

u : displacement

t : time

According to Hooke's Law, the force F can also be related to the strain ε

$$F = -EA \in = -EA\frac{\partial u}{\partial z} \qquad (5.2)$$

From Eq. 5.1 and Eq. 5.2

$$EA\frac{\partial^2 u}{\partial x^2} = \rho A\frac{\partial^2 u}{\partial t^2} \qquad (5.3)$$

or

$$\frac{\partial^2 u}{\partial x^2} - \frac{1}{c^2}\frac{\partial^2 u}{\partial t^2} = 0 \qquad (5.4)$$

where c is the wave propagation velocity

$$c = \sqrt{\frac{E}{\rho}} \qquad (5.5)$$

A partial differential equation in displacement and time, which is the general solution for Eq. 5.4 is

$$u = u^{\downarrow}(x - ct) + u^{\uparrow}(x + ct) \qquad (5.6)$$

That is, the general solution consists of two traveling waves, both propagating with velocity c, but in opposite directions. Along the lines *(x ± ct),* called characteristics, the values of $u^{\downarrow}$ and $u^{\uparrow}$ are steady and are determined from boundary conditions.

For the axial force *F* and the particle velocity *v,* the following equations can be resultant

$$v = \frac{\partial u}{\partial t} = \frac{\partial u^{\downarrow}}{\partial(x-ct)}(-c) + \frac{\partial u^{\uparrow}}{\partial(x+ct)}(+c) = v^{\downarrow} + v^{\uparrow} \qquad (5.7)$$

$$F = -EA\frac{\partial u}{\partial t} = -EA(\frac{\partial u^{\downarrow}}{\partial(x-ct)} + \frac{\partial u^{\uparrow}}{\partial(x+ct)}) = F^{\downarrow} + F^{\uparrow} \qquad (5.8)$$

Therefore, as $v^{\downarrow}$ and $F^{\downarrow}$ are functions of *(x-ct)* only and $v^{\uparrow}$ and $F^{\uparrow}$ are functions of *(x+ct)* only, the force and velocity can also be interpreted as the addition of a downward and upward traveling wave.

From Eq. 5.7 and Eq. 5.8

$$F^{\downarrow} = Zv^{\downarrow} \quad ; \quad F^{\uparrow} = -Zv^{\uparrow} \qquad (5.9)$$

where Z is the impedance of the bar, described as the ratio of the driving force to the associated velocity.

$$Z = \frac{EA}{c} = A\sqrt{E\rho} \qquad (5.10)$$

5.1.2 Boundary Conditions at Top and Toe

A half sine pulse load is now introduced on a pile, which has a finite length *L.* (A half sine is assumed to be formed by a series of pulses. Any time-dependent load can straightforwardly be represented as a series of pulses.)

A compression wave starts traveling downward. To be kept in mind, only the part of the pile between the two characteristics is in motion (with $v = F/Z$); the remainder of the pile is at rest. Once the front of the wave reaches the pile toe at $t = L/c$, it is reflected.

The sort of reflection depends on the character of the pile end-either a free end or a fixed end.

As shown in Figure 5.2, for a free end, the boundary condition is such that at any time *t* the force at the toe is zero. (There is no resistance at a free toe). That is, $F(x=L,t) = 0$. Therefore

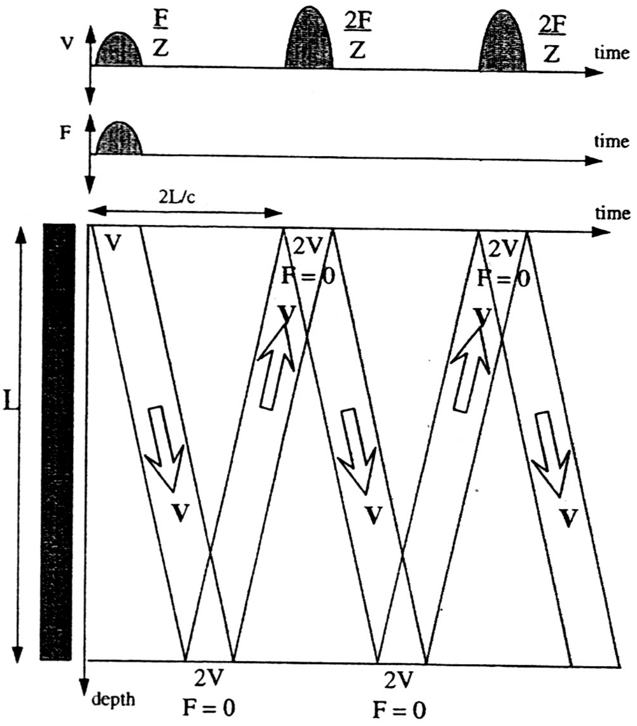


Figure 5.2: Response from a pulse load on a free end pile (TNO, 2003)

$$F(L, t) = F^{\downarrow}(L, t) + F^{\uparrow}(L, t) = 0 \qquad (5.11)$$

or

$$F^{\downarrow}(L, t) = - F^{\uparrow}(L, t) \qquad (5.12)$$

Therefore, a reflected wave of the same but opposite magnitude is introduced at the toe, for a free end at $t = L/c$. The reflected wave is a tension wave.

In the small triangle in the x-t-diagram, both waves overlap and the resulting force is 0. The upward and downward traveling wave's velocities are given by Eq. 5.13.

$$v^{\downarrow} = \frac{F^{\downarrow}}{Z} = \frac{F}{Z} \; ; \; v^{\uparrow} = -\frac{F^{\uparrow}}{Z} = \frac{F}{Z} \; , \quad v(L,t) = v^{\downarrow}(L,t) + v^{\uparrow}(L,t) = \frac{2F}{Z} \qquad (5.13)$$

Therefore, the upward and downward wave velocities have the same sign and magnitude. The value of the velocity is doubled in the overlap area.

At *t* = *2L/c,* the upward traveling wave reaches the free top of the pile and another reflection occurs; a downward traveling compression wave. To be kept in mind, the time average of *v* is given by *v* = *FΔ t/ρAL.*

As shown in Figure 5.3, for a fixed end, the boundary condition is such that at any time *t* the displacement and, therefore, velocity at the toe is zero. (There is no movement at a fixed toe.) That is *v(x=L,t)* = 0. Therefore,

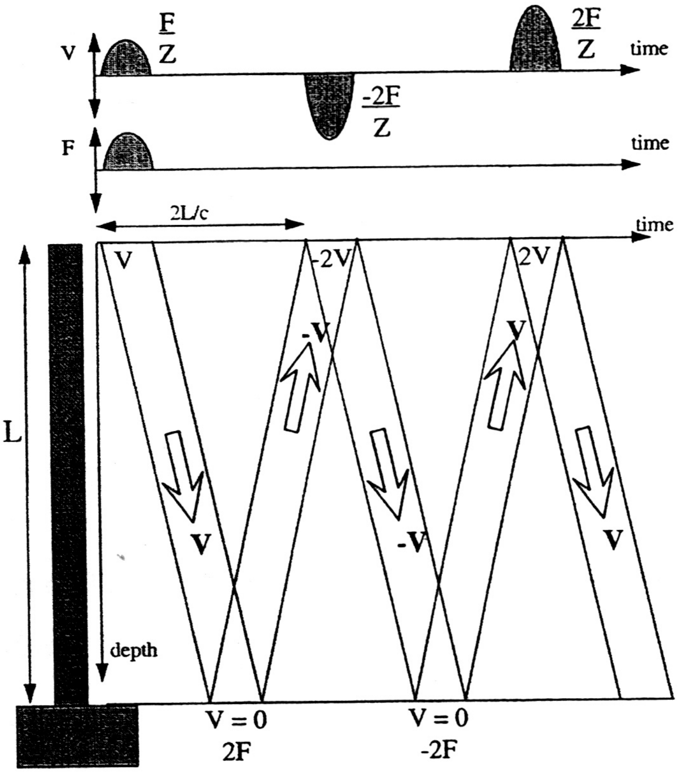


Figure 5.3: Response from a pulse load on a fixed-end pile (TNO, 2003)

$$v(L,t) = v^{\downarrow}(L,t) + v^{\uparrow}(L,t) = 0 \;\; ; \;\; v^{\uparrow}(L,t) = -v^{\downarrow}(L,t) = -\frac{F^{\downarrow}}{Z} = -\frac{F}{Z} \qquad (5.14)$$

$$F^{\uparrow}(L,t) = -Zv^{\uparrow} = F^{\downarrow}(L,t) = F \qquad (5.15)$$

Therefore, for a fixed end, a reflected wave of the same sign and magnitude is introduced at the toe. In that situation, the reflected wave is a compression wave.

In the overlap triangle, the axial force is *2F* and the velocity is zero.

At *t* = *2L/c,* the upward traveling wave reaches the free top of the pile and a further reflection occurs; a downward traveling tension wave.

With the displacement and velocity at the top, it can be concluded that the pile is vibrating with an essential frequency f and a period T according to

$$T = \frac{2\pi}{f} = \frac{4L}{c} = 4L\sqrt{\frac{\rho}{E}} \tag{5.16}$$

The period of vibration can also be found from an eigenvalue analysis as being the lowest eigenfrequency.

5.1.3 Pile Discontinuities

In pile impedance Z, the free and fixed end piles are particular cases of discontinuities. The common case is shown in Figure 5.4. At a discontinuity, the forces, F, are in equilibrium and the velocity, v, on both sides is the same. Therefore,

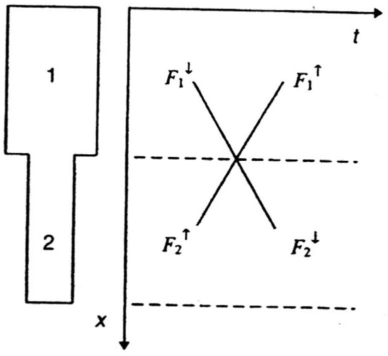


Figure 5.4: General discontinuity

$$F_1^{\downarrow} + F_1^{\uparrow} = F_2^{\downarrow} + F_2^{\uparrow} \tag{5.17}$$

$$v_1^{\downarrow} + v_1^{\uparrow} = v_2^{\downarrow} + v_2^{\uparrow} \tag{5.18}$$

Recalling Eq. 5.9, Eq. 5.18 is rewritten as

$$\frac{F_1^{\downarrow}}{Z_1} - \frac{F_1^{\uparrow}}{Z_1} = \frac{F_2^{\downarrow}}{Z_2} + \frac{F_2^{\uparrow}}{Z_2} \tag{5.19}$$

If some precise point in time is thought, the upward travelling wave in part 2 and the downward traveling wave in part 1 are known. Eq. 5.17 and Eq. 5.19 enable us to calculate the downward traveling wave in part 2 and the upward traveling wave in part 1.

$$F_2^{\downarrow} = \frac{2Z_2}{Z_1 + Z_2} F_1^{\downarrow} + \frac{Z_1 - Z_2}{Z_1 + Z_2} F_2^{\uparrow} \tag{5.20}$$

$$F_1^{\uparrow} = \frac{Z_2 - Z_1}{Z_1 + Z_2} F_1^{\downarrow} + \frac{2Z_1}{Z_1 + Z_2} F_2^{\uparrow} \qquad (5.21)$$

Therefore, for instance, if a pile's cross sectional area is reduced by half ($Z_1 = 2Z_2$), Eq. 5.20 and Eq. 5.21 are written as

$$F_2^{\downarrow} = \frac{2}{3} F_1^{\downarrow} + \frac{1}{3} F_2^{\uparrow} \qquad (5.22)$$

$$F_1^{\uparrow} = -\frac{1}{3} F_1^{\downarrow} + \frac{4}{3} F_2^{\uparrow} \qquad (5.23)$$

The reflections can be seen in Figure 5.5 for a pile with a discontinuity.

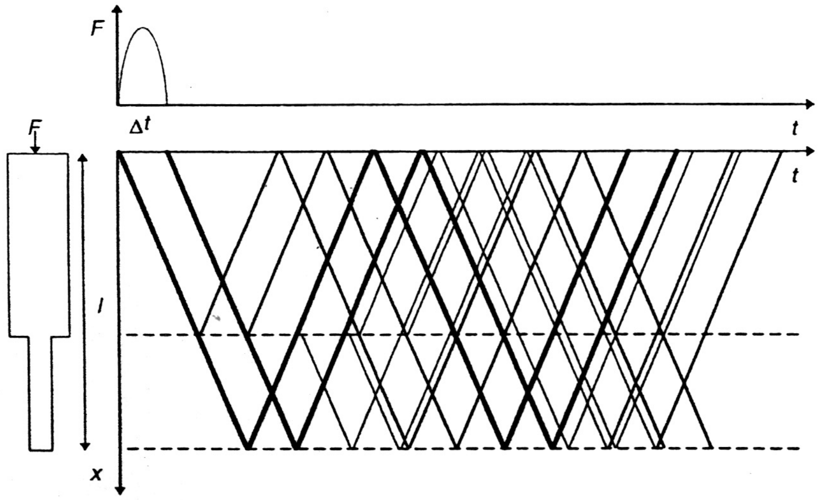


Figure 5.5: Reflections in a pile with a discontinuity

The principles of integrity testing can be explained from the impact of a single ball on a row of balls. There are two circumstances: the last ball can move freely (free end) and the last ball is restricted (fixed end). In both cases, the energy of the impact ball is transferred to the first ball, then to the next, and so on down the row as shown in Figure 5.6a.

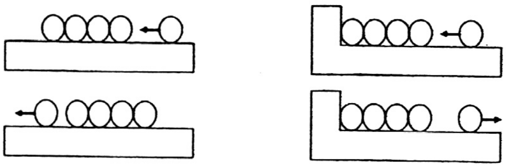


Figure 5.6a: Impact balls: Free end and Fixed end (TNO, 2003)

For a free end, the last ball moves forward, due to can not transferring its energy. To be kept in mind the impact ball remains at rest after impact. For a fixed end, the last ball's energy is reflected backwards, doe to can not transferring its energy. The energy is transferred to the next ball and finally to the impact ball again which move backwards. Therefore, even if the end is not visible, by noting the impact ball alone, one can determine a free end or fixed end condition.

In the same way, it is easy to see a free end or fixed end condition for a pile in soil. As described above, a hammer impact sets the top ball in motion. On the other hand, a pile has both mass and elastic properties and is better modeled as a series of balls joined by springs as shown in Figure 5.6b. Therefore, the spring under the top ball compresses, transferring energy to the next ball. To be kept in mind, the top baJl now has zero velocity. Following the impact, energy is transferred down the pile.

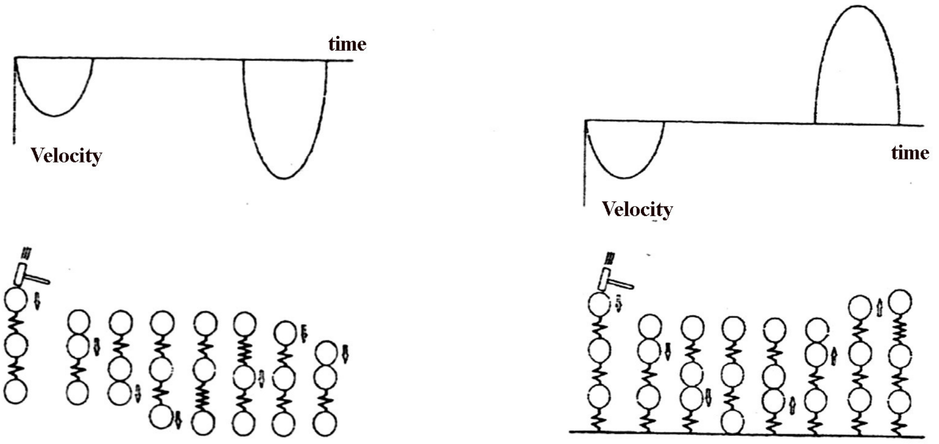


Figure 5.6b: Masses connected by springs: Free end and Fixed end (TNO, 2003)

For a free end, the unrestricted last ball will tension the spring above it, which in turn pulls down the ball above it and so on up the pile. The top ball again moves downward. Therefore, the motion of the top ball is downward at impact, a moment of rest, and downward again. For a fixed end, the restricted last ball will compress the spring below it and push the last ball upward and so on up the pile. The top ball moves upward. Therefore the motion of the top ball is downward at impact, a moment of rest, and then upward.

The speed of propagation of the disturbance moving a ball and compressing or tensioning a spring and moving the next ball can be defined as the stress wave velocity c. The stress wave velocity will be lower for heavier balls and higher for stiffer springs. For a free end, the springs are compressed during the downward propagation and tensioned during the upward propagation. For a fixed end, the springs are compressed during both the downward and upward propagation. The reflections in six general cases are shown in Figure 5.7.

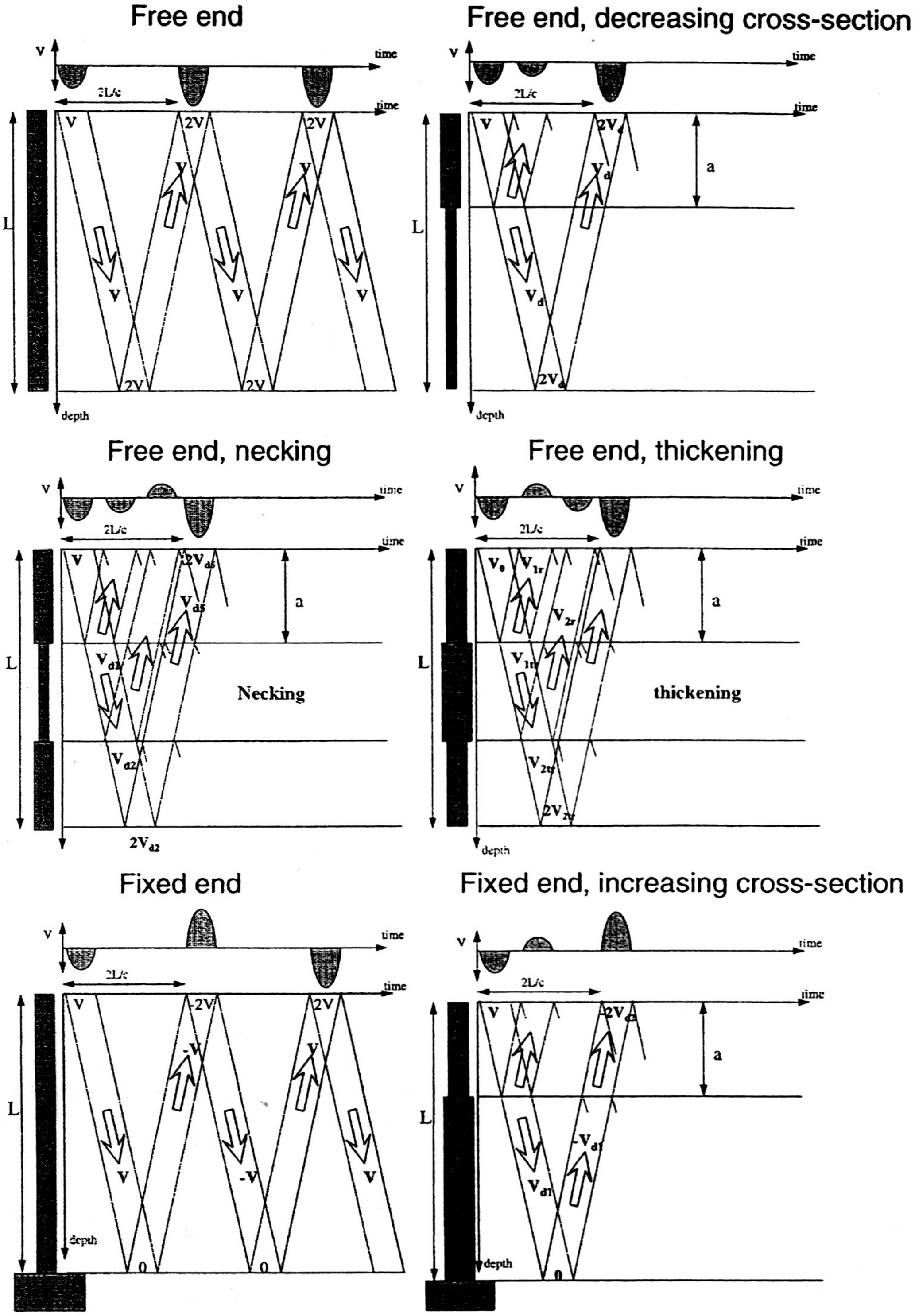


Figure 5.7: Reflections in six general cases (TNO, 2003)

5.1.4 Pile-Soil Interaction

In a free pile, the velocity is shown as a same sign reflection. Free pile wave is shown in Figure 5.8.

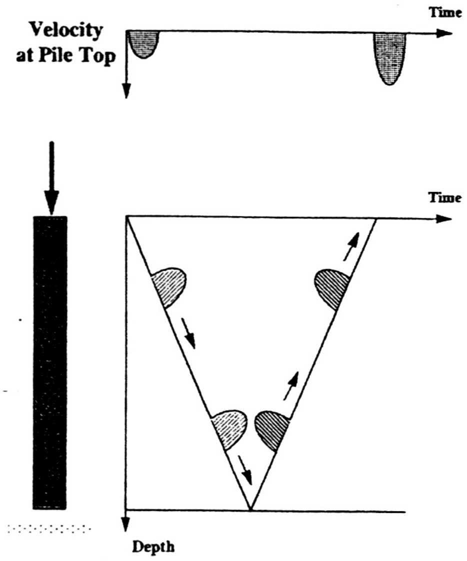


Figure 5.8: : Free pile wave (TNO, 2003)

The velocity is reduced under the influence of shaft friction. Changes from a stiff layer to a soft layer yield the same type of signals as a decrease in cross-section. Changes from a soft layer to a stiff layer yield the same type of signals as an increase in cross-section. Reduced wave from pile-soil interaction is shown in Figure 5.9.

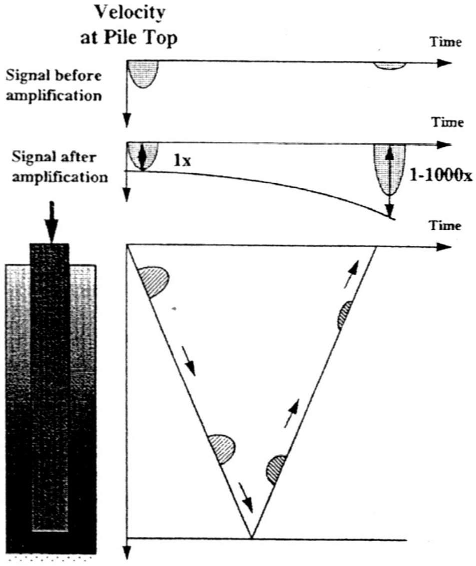
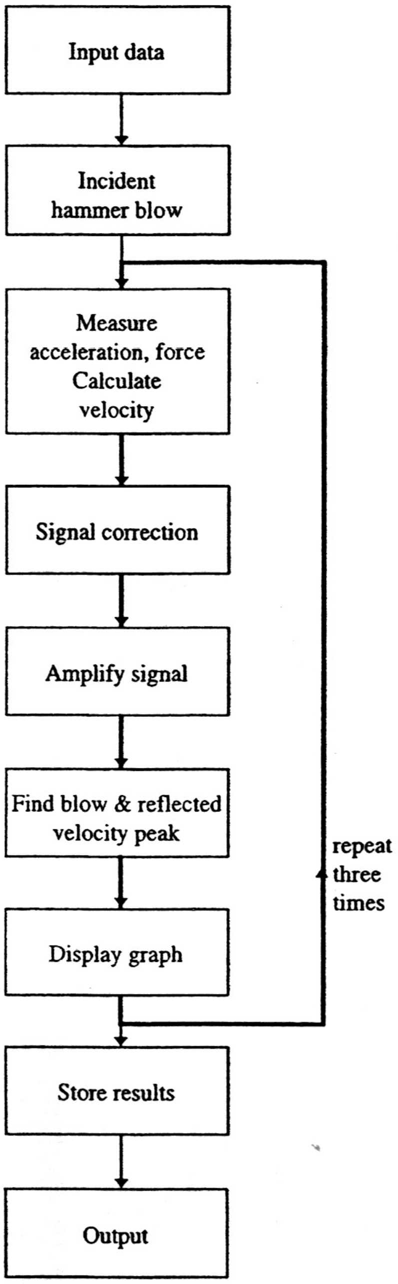


Figure 5.9: Reduced wave from pile-soil interaction (TNO, 2003)

5.1.5 SIT Flow Diagram

The steps are made by SIT through a measurement are shown in Figure 5.10;



- Input data is supplied.
- A hammer blow is introduced to the pile. (For analysis in the frequency domain, the hammer is instrumented.)
- The stress wave calculation begins for all levels along the pile and for each time step.
- Acceleration and force (optional) are measured. Velocity is calculated.
- The signal is corrected to remove any drift from zero level.
- The signal is amplified to reduce the effects of soil resistance (shaft friction or toe resistance).
- The blow and reflected peaks are indicated by markers on screen.
- Each SIT blow is immediately presented on screen.
- The calculation results are stored.
- Results can be output as graphs or text.

Figure 5.10: SIT flow diagram (TNO, 2003)

5.1.6 SIT Input and Output

In a SIT test, as an accelerometer is pressed onto the pile top, the pile is hit with a small hand-held hammer. The acceleration signal is integrated to a velocity signal. Estimated pile length and stress wave velocity is required as an input data for Sonic

Integrity Testing. The incident hammer blow and the resulting reflections are shown by an output graph of velocity versus time.

Input Data

- estimated pile length
- stress wave velocity
- SIT hammer blow

Output Data

- velocity of the pile top as a function of time (or pile length)
- force (instrumented hammer) at the pile top as a function of time (or pile length)

A sample output of a SIT is shown in Figure 5.11.

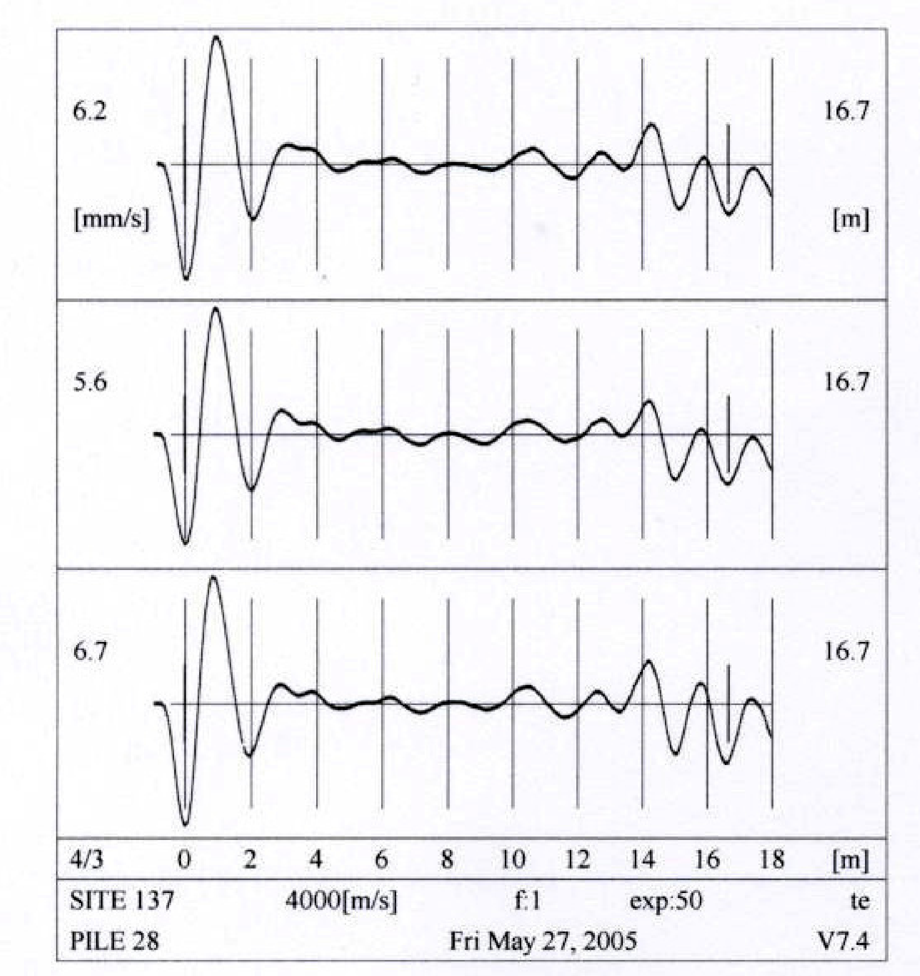


Figure 5.11: SIT sample output

SIT results for four general cases on a Teflon pile are shown in figure 5.12a and 5.12b.

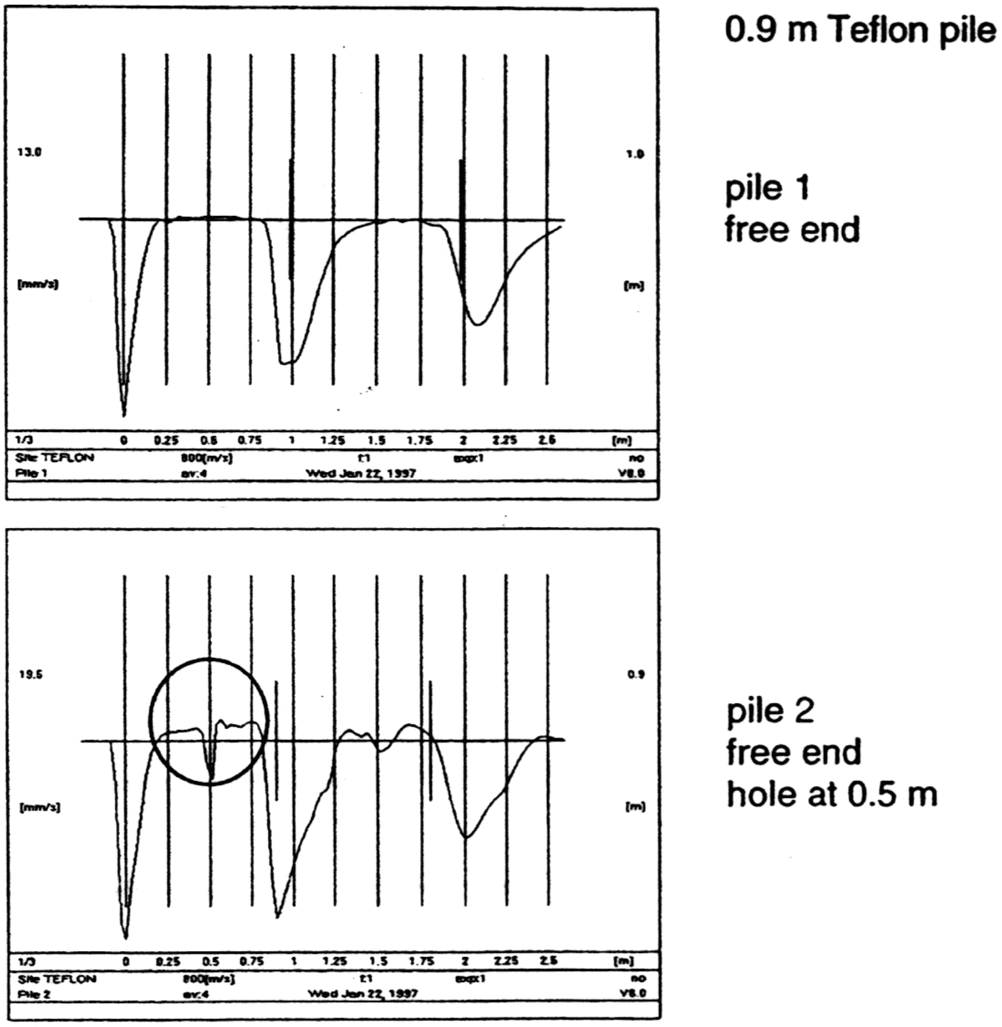


Figure 5.12a: Two different SIT sample outputs (TNO, 2003)

5.2 Concept of Sonic Integrity Testing

In sonic integrity testing, while the pile head movement is recorded with an accelerometer, the pile head is struck with a small hand held hammer. (Also an instrumented hammer can be used to record the impact blow). The simple SIT test is shown in Figure 5.13.

Sonic integrity testing is a straightforward means to help detect major discontinuities or defaults within a pile. Furthermore it identifies piles requiring further examination and is particularly valuable in detecting early integrity problems. Sonic integrity testing should not, though, be used to estimate bearing capacity.

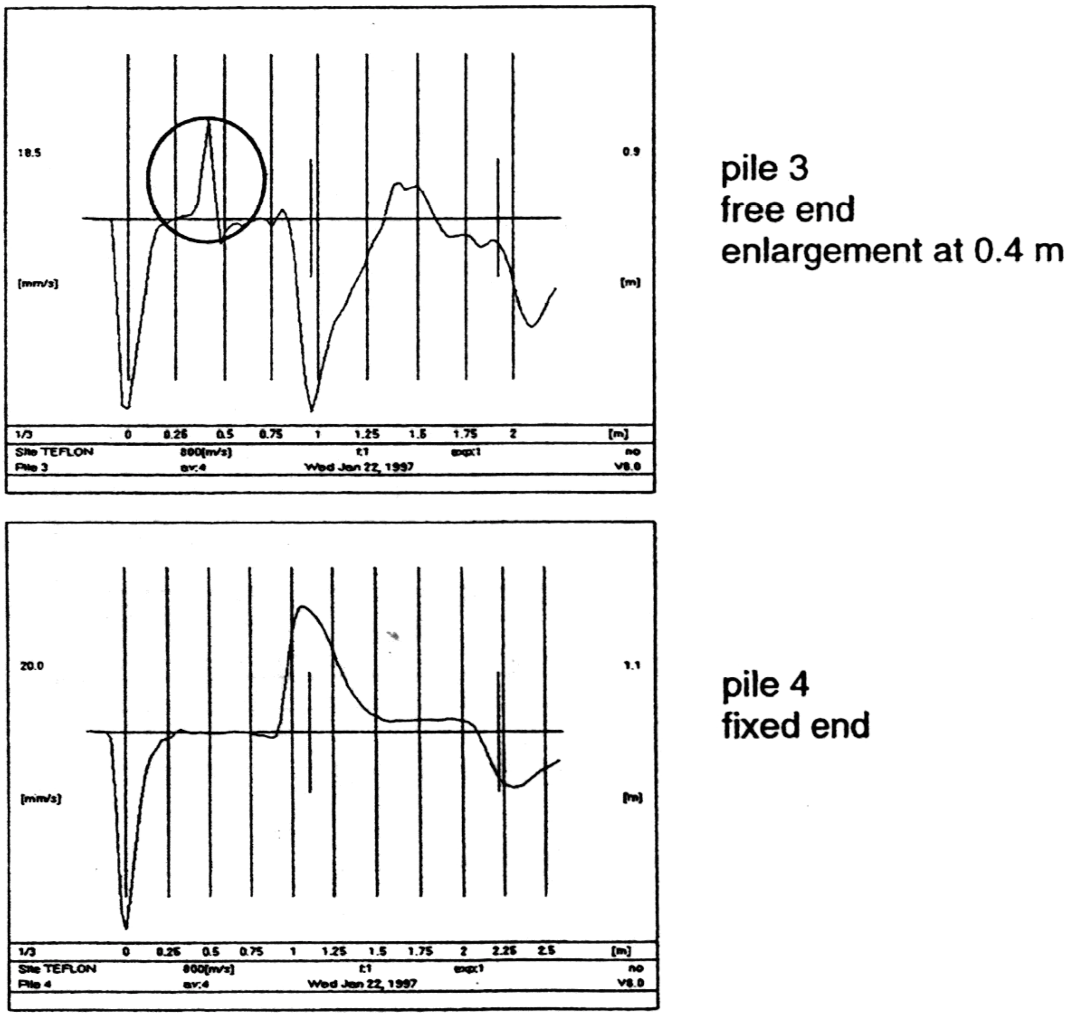


Figure 5.12b: Two different SIT sample outputs (TNO, 2003)

Figure 5.13: The SIT test

Sonic integrity testing provides rapid and reasonably priced results compared to core drilling, inspection by excavation, and load tests, which are time-consuming and pricey.

Millions of piles have been tested using sonic integrity testing, revealing defects in several piles since 1968. Piles have been excavated for additional inspection, where pile defects have been revealed in the top few meters. In other situations, core drilling has been used. In both cases, the excavation has shown the convenience of SIT. (TNO, 2003)

For the most efficient results, the pile head should be dirt free, accessible, sound, and free from standing water. Cast-in-situ piles should be made of known materials and at least 5 days old. SIT is generally not suitable for pre-cast segmental pile or piles discontinuous permanent casings.

5.2.1 The History of Sonic Integrity Testing

At first, SIT was used to find cracks in pre-fabricated piles caused by driving. However, as the use of cast-in-situ piles increase, SIT has been successfully used to help find major faults.

In 1973, TNO made a research to find out the shape and length of cast-in-situ piles for the Department of Public Works. The first results were very hopeful. The measured response of the pile top was displayed on an oscilloscope and recorded on Polaroid film. By the end of the 70s, the FPDS-0 system for Sonic Integrity Testing was developed. Cementation Piling and Foundations of England was one of the first commercial users. The FPDS-0 system is composed of an oscilloscope with storage capabilities and a signal processor. The analogue acceleration signal is amplified, integrated, and filtered. Hard copy is by Polaroid camera. The accelerometer was from Bruel & Kjaer with a frequency range of 2-9000 Hz. (TNO, 2003)

In the mid 80s, the FPDS-l was developed, founded on a UNIX workstation and a 16 bit micro-processor chip, with built-in keyboard, graphical display, disk drive and printer/plotter. The signal was digitized at an early stage subsequent to retrieving the signal to ensure that all following processing was of the highest possible standard.

The system had certain automatic functions, but the skilled operator could also override, magnify, or clarify a result to facilitate interpretation.

With FPDS-2, the signal conditioner and computer were separated, a philosophy sustained with following systems. The FPDS-2 system, which introduced in 1986, was founded on a portable IBM-PC/AT compatible computer running under MS-DOS, having a 20 MB hard disk for program and data storage and a floating point co-processor. For each FPDS application, signal conditioning subsystem, specific sensors and software like Pile Driving Analysis and Dynamic Load Testing of foundation piles (PDA/DLT) were available. The FPDS-2 system was requiring only a limited number of components and no oscilloscope or tape recorder. It did not require knowledge of electronics. Automatic controls, warnings, immediate presentation of signals, and error messages were a big improvement on previous systems. (TNO, 2003)

At the beginning of the 90s, the FPDS-3 system was developed, based on a grid computer and a subsystem with an analogue-lo-digital converter board and a particularly designed board for integrity testing. As with FPDS-2, the subsystem could contain different FPDS applications, like PDA/DLT, the Statnamic load test (STN), vibration measurements (VIBRA), and others. Each option consisted of particularly designed sensors, boards, software, and accessories, an improvement begun with the FPDS-2 system. The operator is freed from bridge balancing and scale selection with automatic signal conditioning. The signal conditioner was built into the grid computer to reduce weight while conducting multiple SIT tests.

With FPDS-3, SIT was also extended to contain an optional instrumented hammer (an accelerometer fixed in a hammer head) to record the applied impact blow. The design of the hammer head, impact cap, accelerometer sensitivity, and hammer shaft stiffness produces a load pulse signal completely representative of the impact amplitude and phase. Due to the fact that force is proportional to velocity at the pile head, the shape of the force and velocity traces are identical, until a reflection is recorded. Defects in the first 2 meters of the pile are revealed by comparing the force signal to the velocity signal. The force is equal to the pile head impedance x velocity. Figure 5.14 shows pile with no defect and pile with defect in upper part.

FPDS-4 was developed in 1993 as a very robust and inexpensively designed lunch box type system. The FPDS-4 system includes 80-200 MB hard disk, a 80486 processor, and a built-in graphical screen and key board, which are both sealed to protect against rain and dust. (TNO, 2003)

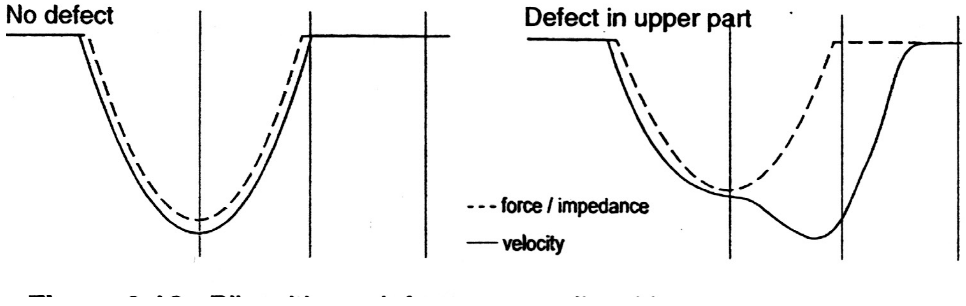


Figure 5.14: Pile with no defect versus pile with defect in upper part

In recent times, the FPDS-5 system was developed using a notebook computer and separate signal conditioning and A/D card subsystem. Data communication between subsystem and notebook is via a PCMCIA card and cable. The FPDS-5 system is not only suited for PDA/DLT and STN, but also it can be used for SIT.

Additionally, TNO's newest FPDS system, the FPDS-6, represents a new approach to foundation testing equipment. The FPDS-6 is a hardware/software package for SIT, designed to work with a wide range of IBM compatible PC computers equipped with a PCMCIA card slot as shown in Figure 5.15. The SIT hardware consists of a sensor, hammer, cable, and PCMCIA card. The PCMCIA card, measuring just 8.5 cm by 5.5 cm by 5 mm, incorporates all gained data (300 kHz, 14 bit) and signal conditioning electronics. Signals and data are stored automatically and can be effortlessly recalled. Automatic reporting capabilities decrease reporting time. (TNO, 2003)

Figure 5.15: SIT with PCMCIA card

The main advantages of SIT are:

- first line quality assurance against major faults
- hundreds of piles can be tested in a single day
- testing on any accessible pile
- defects discovered at an early stage
- quick and economical compared to other methods
- equipment is portable and easy to operate

The limitations of sonic integrity testing include:

- can not estimate bearing capacity
- minor defects are not easily seen (local loss of cover to steel or small inclusions)
- length is difficult to determine for very long piles with very high shaft friction
- debris at the pile toe can not be determined

5.2.2 Signal Processing

A SIT signal contains information about changes in impedance caused by variations in pile cross-section, soil characteristics, or pile density as well as holes, soil inclusions, and cracks. (Hassanein, 1988 and TNO, 2003)

The accelerometer signal is amplified and converted to digital form, preserving all details, particularly signals with weak toe reflections. By integrating the acceleration signal, velocity is calculated. For additional involved analysis, the hammer blow can also be recorded with an instrumented hammer. The velocity-time and force-time graphs are presented on screen and can be printed or stored for additional analysis. Three sample SIT signals are shown in Figure 5.16. (Hassanein, 1988 and TNO, 2003)

For constancy, the results of a number of hammer blows are averaged, reducing noise and highlighting features unique to the test pile. By identifying toe reflections as reflected pulses, pile length can be determined.

Results can be further analyzed using SIT, to estimate the dimensions of any discontinuities.

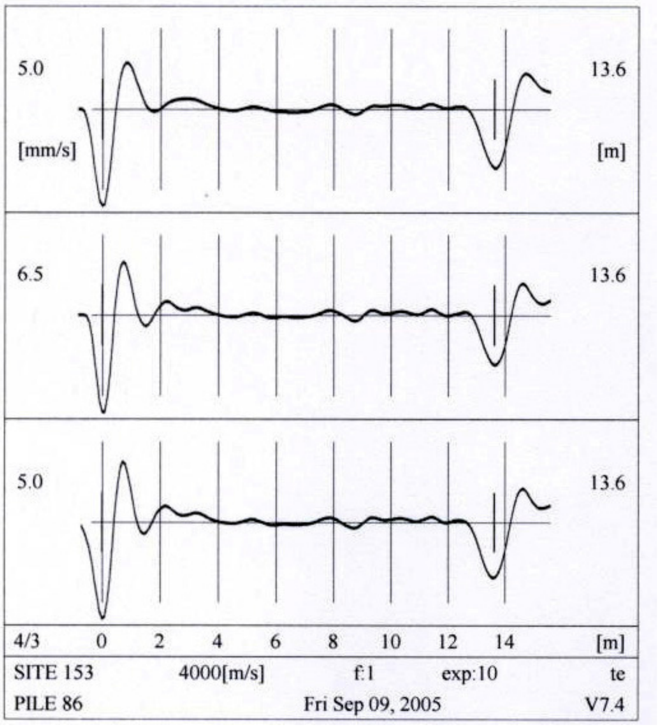


Figure 5.16: Sample SIT signals

5.2.3 The Instrumented Hammer

As described above, a SIT test measures the resulting pile movement from a hammer blow, using an accelerometer pressed onto the pile head.

To improve reliability, a SIT test can also be conducted with a specially-instrumented hammer (an accelerometer fitted in a hammer head) which records the impact blow. (To analyze a SIT test in the frequency domain, impact data must be recorded.) The instrumented hammer is shown in Figure 5.17. The design of the hammer head, impact cap, accelerometer sensitivity, and hammer shaft stiffness produce a load pulse signal perfectly representative of the impact amplitude and phase. (Middendorp and Reiding, 1998)

Since force is proportional to velocity at the pile head, the shape of the force and velocity traces are identical until a reflection is recorded with the accelerometer. Defects can be revealed, by comparing the force signal to the velocity signal. *(*From Equation 5.9, the ratio of force to velocity equals impedance and from Equation 5.10, impedance is a function only of a pile's material properties; specifically, the modulus of elasticity, stress wave velocity, and cross-section.) /TNO, 2003)

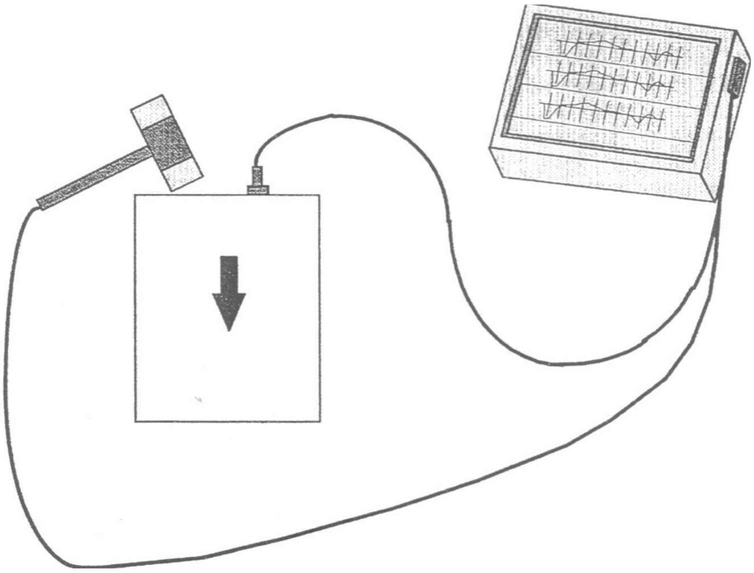


Figure 5.17: The instrumented hammer (TNO, 2003)

Therefore, from independent measurements of force and velocity, any discrepancy is attributed to a change in impedance. As the modulus of elasticity and the stress wave velocity are constant for a given test pile, the discrepancy results from a change in pile cross-section caused either by a change in soil resistance or a pile discontinuity. *(*Interpreting SIT signals with reflections is more involved.)

5.2.4 Mechanical Admittance and the Frequency Domain

For the reason that a single blow to a pile also produces data over a wide frequency range, integrity testing can also be analyzed in the frequency domain. Developed by J. Paquet [1968], early tests were conducted by placing an electro-dynamic vibrator on top of a pile and vibrating the pile (with a steady amplitude sinusoidal force) in a frequency range from 20-1000 Hz and measuring the velocity at the pile head with an accelerometer attached to the pile head. Nowadays, in the frequency domain, the instrumented hammer is used to analyze pile integrity. (TNO, 2003)

Mechanical admittance (the ratio of velocity to force or the reciprocal of impedance) is plotted versus frequency, revealing pile information similar to integrity testing in the time domain. To determine mechanical admittance, the velocity signal (calculated by integrating the acceleration signal from the accelerometer) and the force signal (measured by the instrumented hammer) are transformed into the frequency domain using fast Fourier transfer analysis. Mechanical admittance is then obtained as a function of frequency. A typical mechanical admittance graph is shown in Figure 5.18. (Piwu, 1991 and TNO, 2003)

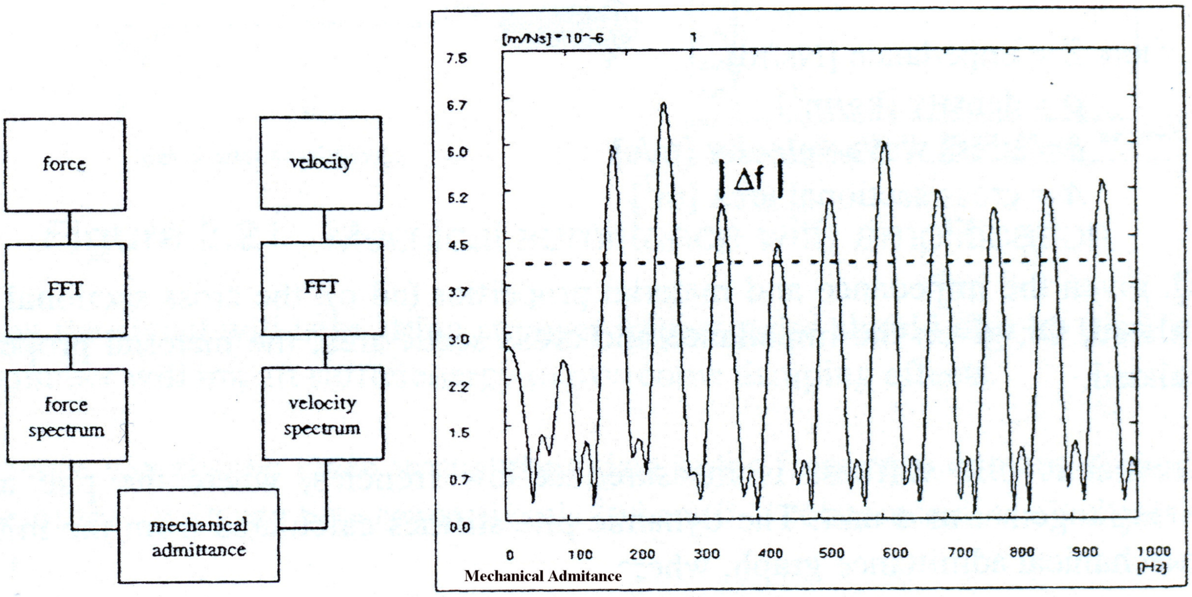


Figure 5.18: Mechanical admittance

In the time domain, pile length is determined by identifying toe reflections as reflected pulses. In the frequency domain, the pile length is calculated from the distance between resonance peaks,

$$l = \frac{c}{2\Delta f} \qquad (5.24)$$

where l: pile length [m]

c : stress wave velocity [m/s]

Δf : distance between two successive peaks [s^{-1}]

Any irregularities in the pile shaft are also seen as resonance peaks, corresponding to the depth of the irregularity and, therefore, the resonance peaks will appear closer together. As in the time domain, differences in cross-section and soil resistance are seen as a change in impedance. (Stiff and weak layers are shown as an increase and decrease, respectively, in impedance.)

The pile impedance, *Z*, can be calculated from the pile admittance, *N,* where

$$Z = \frac{1}{N} \qquad (5.25)$$

where Z : impedance [Ns/m]

N : admittance [m/Ns]

Admittance is averaged in the resonant range, as shown in Figure 5.18.

From Eq. 5.5 and Eq. 5.10,

$$Z = \rho c A \qquad (5.26)$$

where Z : impedance [Ns/m]

ρ : density [kg/m^3]

c : stress wave velocity [m/s]

A : cross sectional area [m^2]

Therefore, given the impedance and material properties (ρ and c), the cross sectional area can be calculated. Or, given the impedance and cross sectional area, the material properties can be calculated.

Where the pile and soil are vibrating together as a unit, the dynamic pile stiffness is measured at low frequencies. The dynamic pile stiffness is calculated from the initial slope of the mechanical admittance graph, where

$$k = \frac{2\pi f_o}{Z_o} \qquad (5.27)$$

where k : dynamic pile stiffness [N/m]

f_0 : frequency at end of initial slope [s^{-1}]

Z_0 : impedance at end of initial slope [s/kg]

Dynamic pile stiffness is often used as the initial static stiffness to calculate static behavior under working load conditions, on the other hand, maximum pile displacement in this range (on the order of μm) is well below that of the working load (on the order of mm). Only when the pile-soil system is perfectly elastic up to the working load, should the dynamic pile stiffness be used as the static stiffness. For instance, cast-in-situ piles, often show a soft initial behavior. (TNO, 2003)

In numerous cases, insufficient energy is returned from the toe due to damping (from shaft friction and toe resistance) and, therefore, the pile length can not be determined from resonance peaks. As shown in Figure 5.19, the mechanical admittance is reduced to a horizontal line.

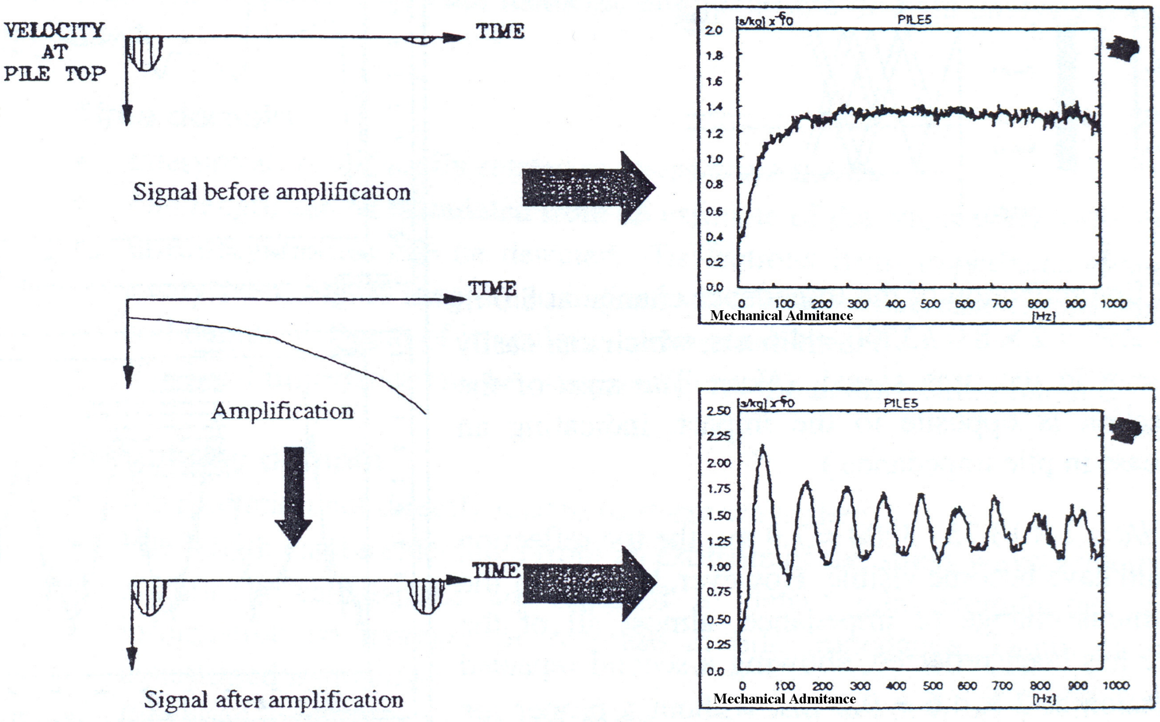


Figure 5.19: Mechanical admittance with amplification

Although, a stronger impact pulse should be tried, amplifying the signal will help. Using a heavier hammer will impart satisfactory energy to overcome damping effects.

The consistency can also be given while analyzing data in the frequency domain. Consistency is a measure of the similarity between three signals (where 0≥ coherence ≤1; a coherence of 1 identical.)

Results of a theoretical pile (used to simulate a pile with defects) are shown below to highlight SIT output of a defective pile.

- theoretical pile with properties:

  length = 13.75 m

  stress wave velocity = 3500 m/s

    modulus of elasticity = 5.82x10^4 MPa

    pile density = 2300 kg/m^3)

- pile enlargement

  enlargement = 8.0 m from the pile top

length of enlargement = 1.0 m

nominal pile diameter = 0.615 m

pile diameter at enlargement = 2.0 m

The first reflection of the impedance change at 8.0 m is at *2x/c* = 2 x 8.0/ 3500 = 4.6 ms, which can effortlessly be seen in the time signal. *(*The sign of the reflection is opposite to the impact, indicating an increase in pile impedance.). Result of a theoretical pile is shown in Figure 5.20.

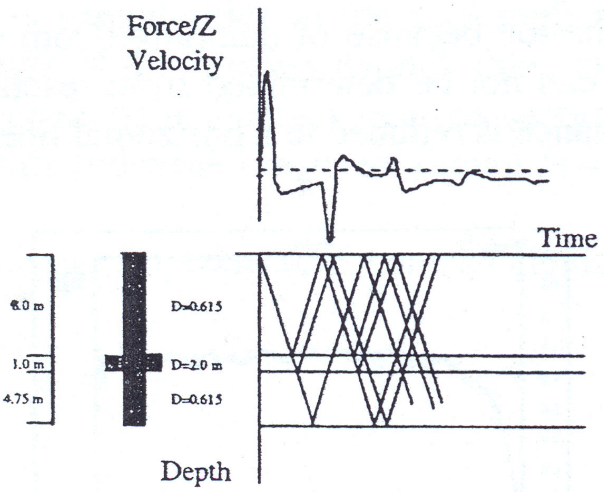


Figure 5.20: Results of a theoretical pile

At *2l/c* = 2 x 13.75/3500 = 7.9 ms, the toe reflection should have turn out to be visible. Nevertheless, owing to the enormous change of impedance, almost all of the wave has been reflected, showing a second repeated reflection at 2 x 4.6 = 9.2 ms, without a proper toe reflection.

In the frequency domain, the mechanical admittance graph shows sharp peaks at intervals of about 220 Hz and, therefore, the corresponding length is

$$l = \frac{c}{2\Delta f} \cong \frac{3500m/s}{2.200s^{-1}} = 7.9m$$

As in the time domain analysis, the pile shows an imperfection at about 7.9 m. The sort of the imperfection (reduction or enlargement in impedance) can not be determined.

A resonance frequency of 127 Hz, corresponding to a pile length of 13.75 m is shown by the reference pile. The defective pile shows a resonance frequency of 220

Hz. There is dissimilarity in the calculated average impedances, (4,180,000-4,550,000 m/Ns*),* but this dissimilarity gives no important indication about the type of defect at 8.0 m. the mechanical admittance of reference versus defective pile is shown in Figure 5.21.

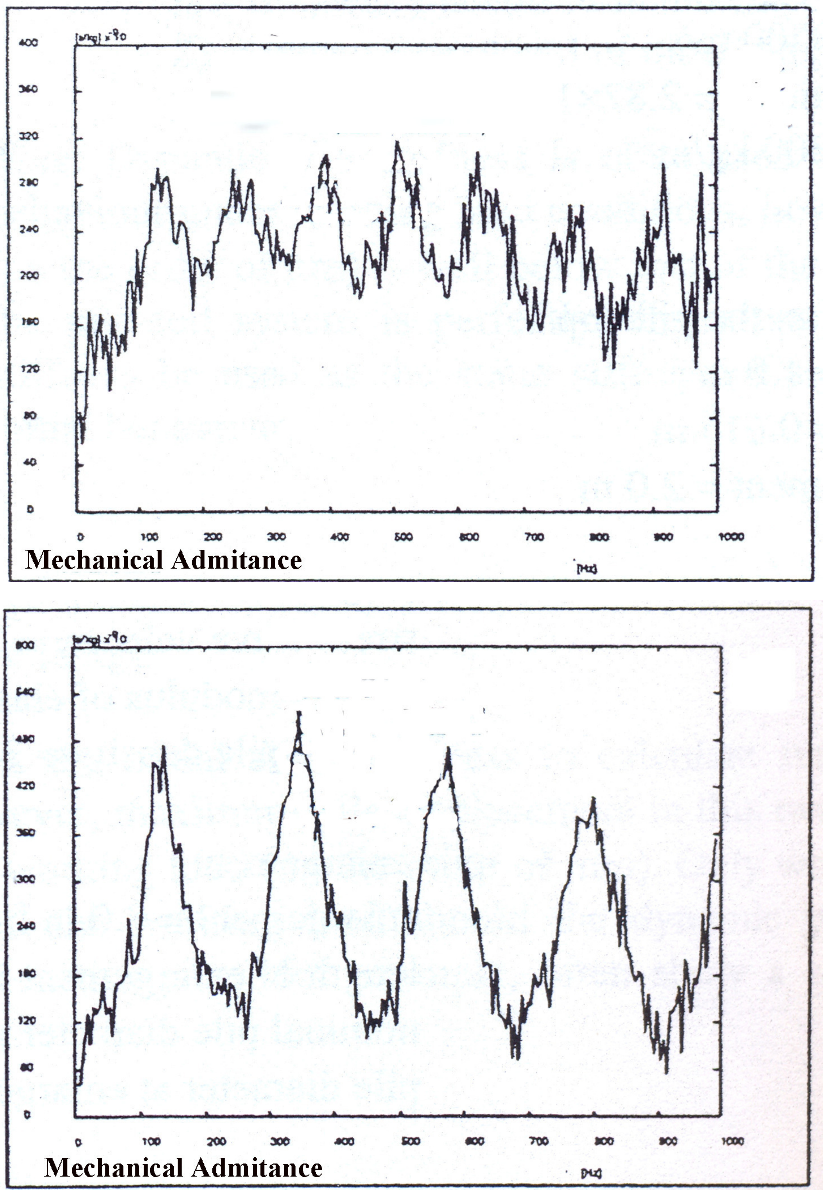


Figure 5.21: Mechanical admittance: reference versus defective pile (TNO, 2003)

5.2.5 Time Domain versus Frequency Domain

Analyzing a SIT test in frequency domain or the time domain is a matter of preference. Although time domain analysis is typically less complicated, helpful pile information is provided by both methods.

Time domain

- Interpretation is directly related to stress wave theory.
- Pile length can be calculated from an estimate of the stress wave velocity.
- Pile irregularities can be detected. (Reflections from impedance changes can be easily seen. Signal magnitude indicates increase or decrease in pile impedance or soil damping. Depth of irregularity determined directly.)
- Pile head impedance can be estimated (with the instrumented hammer).

Frequency domain

- Interpretation not directly related to stress wave theory.
- Pile length can be calculated from an estimate of the stress wave velocity.
- Additional calculations are required to detect pile irregularities. (No direct information on -increase or decrease in pile impedance. Depth of irregularity determined indirectly.)
- Pile head impedance can be estimated.
- Pile stiffness under an extremely small dynamic load can be determined.
- Processing time is longer.

A comparison of both methods is shown in Table 5.1

Table 5.1: Time domain versus frequency domain analysis (TNO, 2003)

	Time Domain	Frequency Domain
Stress wave theory interpretation?	Yes	No
Calculate pile length?	Yes	Yes
Detect pile irregularities?	Yes	Yes
Detect increase or decrease in cross-section?	Yes	No
Calculate pile head impedance?	Indirect	Direct
Initial stiffness under extremely small dynamic load?	No	Yes
Static capacity information?	No	No

5.2.6 Accuracy of Results

The accuracy of any result, certainly, depends on the input values. Estimates of pile length and stress wave velocity are input for all SIT tests. (The modulus of elasticity and cross sectional area are input with the instrumented hammer and transducer distance with two accelerometers.)

Estimated pile length: The length is measured directly for pre-cast and steel piles. For cast-in-situ piles, length is estimated from construction drawings. (Length can be estimated by the supervisor or from site information during construction as well.)

Stress wave velocity : Depending on age of concrete, the stress wave velocity in concrete varies between 3600 m/s and 4400 m/s.

Modulus of elasticity: Depending on the age and quality of the concrete, the modulus of elasticity varies between 20,000 MPa and 40,000 MPa. To be kept in mind, there is dissimilarity between the static and dynamic modulus of elasticity. Table 5.2 gives an indication of typical values.

Table 5.2 E, ρ, and c of pile materials (TNO, 2003)

Material	Modulus of elasticity [MPa]	Density [kg/m^3]	Stress wave Velocity(m/s)
Steel piles	210,000	7850	5200
Old precast piles (prestressed)	> 40,000	2500	>4000
New precast piles (prestressed)	40,000	2500	4000
Compacted concrete case-in-situ	35,000	2500	3800
Uncompacted concrete	30,000	2300	3600
Poor concrete quality	20,000	< 2300	<3000

Cross sectional area: The design diameter is never the same as the real diameter, mainly near the pile top. A 10% error (Δd) in the estimated pile diameter produces a 20% error (2 x Δd) in the cross sectional area.

5.2.7 Situations That are Not Detectable with SIT

Situations that are not detectable with SIT are as fallows

- Gradual increases or decreases in cross-section
- Curved forms
- Small inclusions of foreign materials
- Local loss of cover
- Debris at the toe of the pile
- Cracks parallel to the pile axis

5.2.8 The Characteristic Signal

The SIT signal from a test pile is compared to a "characteristic signal" deemed to be representative of similar piles in similar soil conditions on site to differentiate between a change in soil resistance and a pile discontinuity,. (The characteristic signal can either be an average of a number of piles on site or the SIT signal of a reference pile chosen prior to testing.) If the test signal is different than the characteristic signal, then any impedance changes are owing to the changing pile impedance and not characteristic of the site. Changes not found in the characteristic signal require additional analysis to determine the cause.

5.2.9 Basic Signal Interpretation

A number of piles at several locations on a site are tested. Testing only piles which appear suspect is never been satisfactory. The characteristic signal for piles of the same sort on a site is determined. The characteristic signal should be compatible with that for other piles of the same kind, and should usually correspond with the majority of piles tested on a particular site. The characteristic signal can be established intuitively or made by averaging a number of pile results together, excluding any piles which deviate from the norm.

The characteristic signal is compared with the available soil data. The causes of deviations-most often a change in pile cross-section caused by soft layers, fill materials, voids in ground, old foundation bases, entry into hard layers, casing lengths, or deliberate pile base enlargements is tried to understand. *(*If a pile enters a rock material, damping will be very high on account of greatly increased shaft friction and will show as an apparent increase in cross-section and no reaction from the pile toe.)

If possible, determining the pile length from the characteristic signal is tried. using all individual signals. Flag as "suspect" signals with important deviations from normal is carried out. The level and type of deviation from normal and physically examine the pile is determined.

5.2.10 Requirements for Reliable Testing

The requirements for a reliable cast-in-situ pile include:

- high quality soil investigation

- well designed
- experienced piling contractor
- good piling equipment
- experienced equipment operator
- high quality concrete
- supervision during construction
- reliable testing method
- test at least 10 % of piles on site
- for the highest confidence, test all piles
- determine average site signal
- check reference signal with soil investigation data
- check signals that deviate from reference signal
- perform qualitative interpretation

If required, quantitative interpretation with integrity testing signal matching using TNOWAVE (TNOSIT) is performed. If there are still doubts, the pile is excavated.

6 FIELD APPLICATION OF SONIC INTEGRITY TESTING

The tests were performed on piles, which were constructed under coverage of a housing project at Esenyurt district in Istanbul, which is composed of 156 residences having different sizes and number of stairs. A number of those residences, planned to be constructed in scope of the housing project, were designed to have pile foundations. However the project consists of 156 residences, in this study, only piles constructed under residences numbered as 136,137, 138, 139, 140, 141, 142, 143, 144, 147, 148, 149, 151, 152, 153 and 154 were tested and are discussed. The plan view of these residences is given in Appendix A.

Piles constructed under residences mentioned above are drilled and cast-in-situ piles, which were tested with Sonic Integrity Test (SIT) for quality control and to determine the possible discontinuities occurred during construction of the piles. Three records have been taken for each pile being tested and were evaluated.

6.1 Soil Conditions of the Area

Soil profile of the area was derived by the help of boring logs obtained from soil investigation, which was performed to design foundations of structures planned to be constructed. Four sections were chosen on the map and four soil profiles were derived from the logs of borings, which are close to these sections. Location of borings and these sections A-A’, B-B’, C-C’ and D-D’ are given in Appendix B. Additinaly, boring logs are given in Appendix C.

Regarding to boring logs, it is observed that at eastern part of the area, fist layer is Medium Stiff Clay, which has an average SPTN-30 value of 5, with a thickness of 2-5 meters (SK-8, SK-9, SK-16 and SK-12). At South-East, the second layer is observed as Stiff Clay, which has an average SPTN-30 value of 13 with a thickness of 2-5 meters (SK-8 and SK-9). This layer was also experienced at SK-12 located at North-East of the area as a tiny layer with a thickness of less than 2 meters. Hard Clay layer is the third layer, if stiff clay is available, or the second layer, if stiff clay is not available, with SPTN-30 values ranging from 12 to 34 (SK-8, SK-16 and SK-

12). At the end of borings SK-16 and SK-12, Clay Stone was observed with a SPTN-30 value higher than 50.

At western part of the area, the soil profile is slightly different than the eastern part. The only difference is, Medium Stiff and Stiff Clays are not seen at North-West of the area (SK-13) but these layers were again experienced at southern part of west side (SK-6 and SK-4) with a thickness of less than eastern part. Soil profiles A-A', B-B', C-C' and D-D' derived from boring logs are given in Appendix D.

6.2 SIT Field Testing and Its Evaluation

In this study, chosen piles constructed under residences numbered as 136,137, 138, 139, 140, 141, 142, 143, 144, 147, 148, 149, 150, 151, 152, 153 and 154 were tested on the field with SIT equipment described in Section 5. After testing completed, data obtained was evaluated according to SIT knowledge. Piling plan of residences, pile information and evaluation of SIT test results are given below.

Residence 136

Piling plan of residence numbered as 136 is shown in Figure 6.1. At this residence, 8 out of 41 piles (1, 8, 11, 15, 22, 25, 32 and 34) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.1.

Table 6.1: Evaluation of SIT results for residence 136

Residence No: 136		Total Num. Of Piles:		41
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
1	50 cm	15 m	B	4 m
8	50 cm	15 m	C	6 m
11	50 cm	15 m	B	12 m
15	50 cm	15 m	B	3 m
22	50 cm	15 m	A	
25	50 cm	15 m	B	3 m
32	50 cm	15 m	B	2,5 m
34	50 cm	15 m	B	4,5 m

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

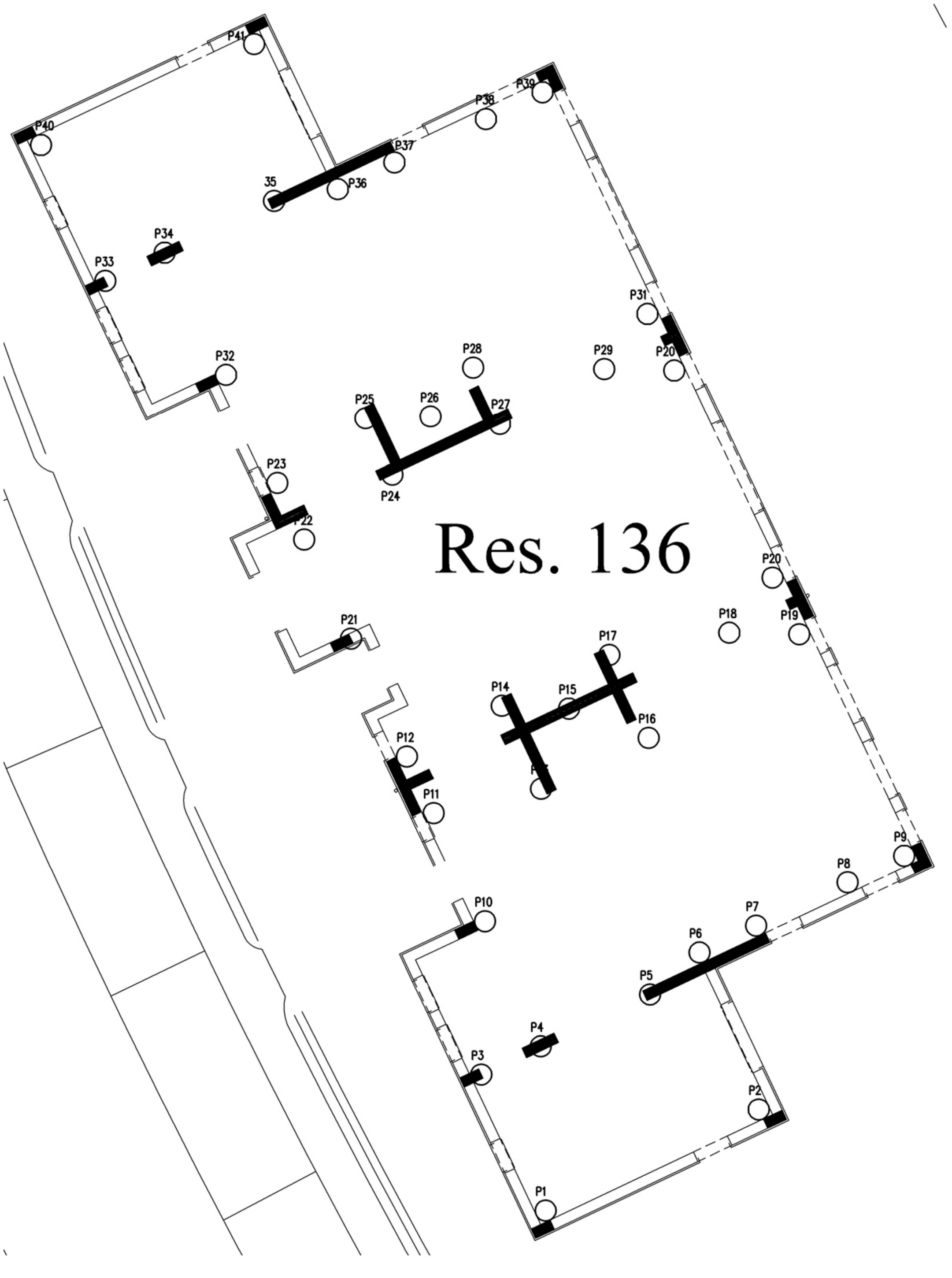


Figure 6.1: Piling plan of Residence 136

Residence 137

Piling plan of residence numbered as 137 is shown in Figure 6.2. At this residence, 15 out of 41 piles (5, 9, 11, 16, 17, 18, 19, 20, 23, 24, 28, 30, 32, 38 and 41) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.2.

Table 6.2: Evaluation of SIT results for residence 137

Residence No: 137		Total Num. Of Piles:		41
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
5	50 cm	15 m	B	5,5 m
9	50 cm	15 m	B	5 m
11	50 cm	15 m	A	
16	50 cm	15 m	B	3 m
17	50 cm	15 m	B	3 m
18	50 cm	15 m	A	
19	50 cm	15 m	B	2 m
20	50 cm	15 m	B	2 and 6 m
23	50 cm	15 m	A	
24	50 cm	15 m	A	
28	50 cm	15 m	B	4 m
30	50 cm	15 m	A	
32	50 cm	15 m	A	
38	50 cm	15 m	B	7 m
41	50 cm	15 m	B	4 m

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

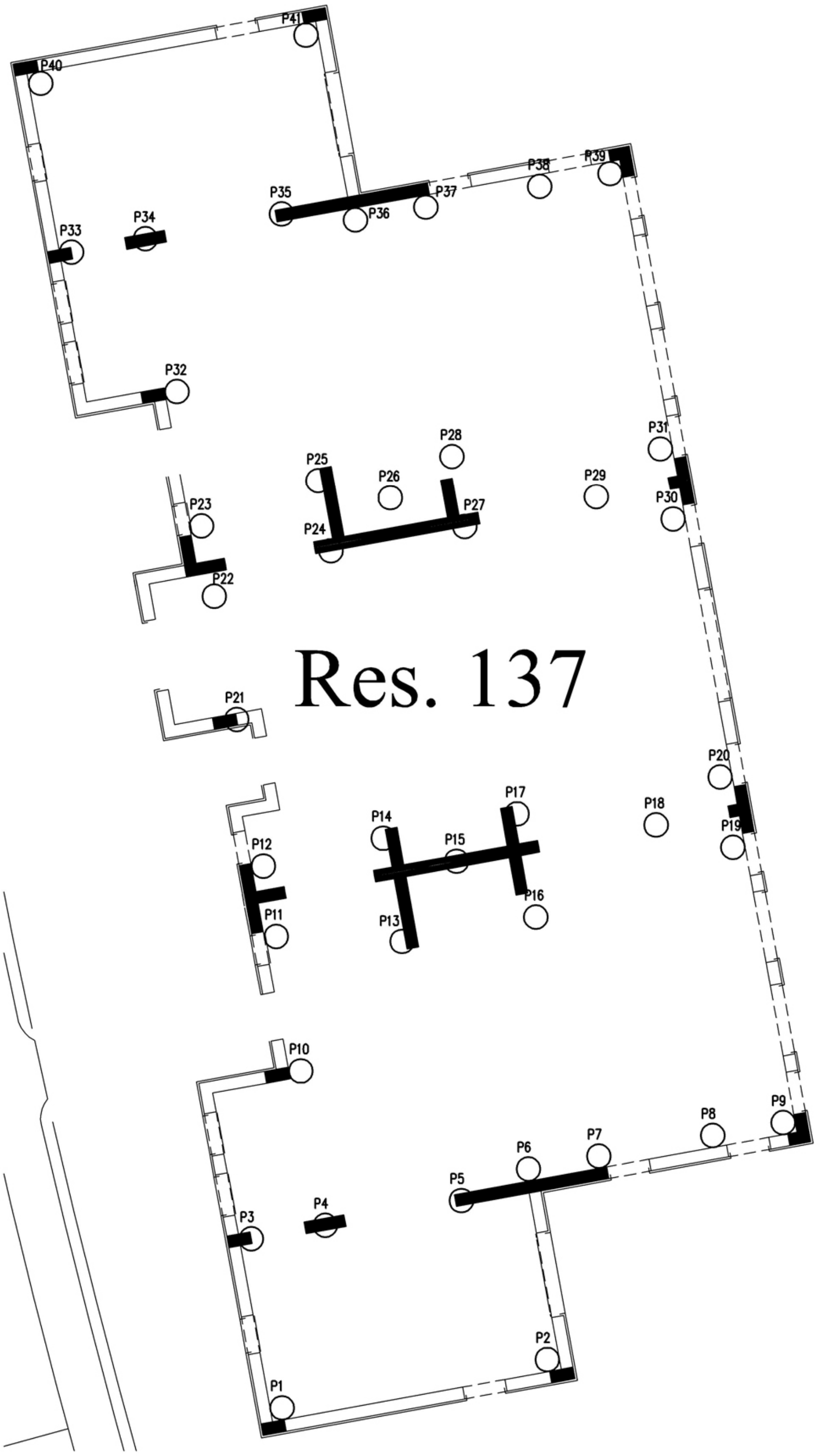


Figure 6.2: Piling plan of Residence 137

Residence 138

Piling plan of residence numbered as 138 is shown in Figure 6.3. At this residence, 14 out of 52 piles (2, 13, 15, 20, 29, 30, 36, 39, 40, 41, 45, 46, 47 and 52) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.3.

Table 6.3: Evaluation of SIT results for residence 138

<table>
<tr><td colspan="2">Residence No: 138</td><td colspan="2">Total Num. Of Piles:</td><td>52</td></tr>
<tr><td colspan="5">Tested Piles</td></tr>
<tr><td>Pile No.</td><td>Pile Diameter</td><td>Pile Length</td><td>Discontinuity Type</td><td>Discontinuity Loc. (Distance from Top)</td></tr>
<tr><td>2</td><td>50 cm</td><td>15 m</td><td>B</td><td>3 and 9 m</td></tr>
<tr><td>13</td><td>50 cm</td><td>15 m</td><td>B</td><td>3 m</td></tr>
<tr><td>15</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>20</td><td>50 cm</td><td>15 m</td><td>B</td><td>2,5 m</td></tr>
<tr><td>29</td><td>50 cm</td><td>15 m</td><td>B</td><td>2 m</td></tr>
<tr><td>30</td><td>50 cm</td><td>15 m</td><td>B</td><td>8 m</td></tr>
<tr><td>36</td><td>50 cm</td><td>15 m</td><td>B</td><td>6 m</td></tr>
<tr><td>39</td><td>50 cm</td><td>15 m</td><td>B</td><td>2 m</td></tr>
<tr><td>40</td><td>50 cm</td><td>15 m</td><td>B</td><td>3 and 7 m</td></tr>
<tr><td>41</td><td>50 cm</td><td>15 m</td><td>B</td><td>2 and 8 m</td></tr>
<tr><td>45</td><td>50 cm</td><td>15 m</td><td>B</td><td>3 m</td></tr>
<tr><td>46</td><td>50 cm</td><td>15 m</td><td>B</td><td>7 m</td></tr>
<tr><td>47</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>51</td><td>50 cm</td><td>15 m</td><td>B</td><td>7 m</td></tr>
</table>

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

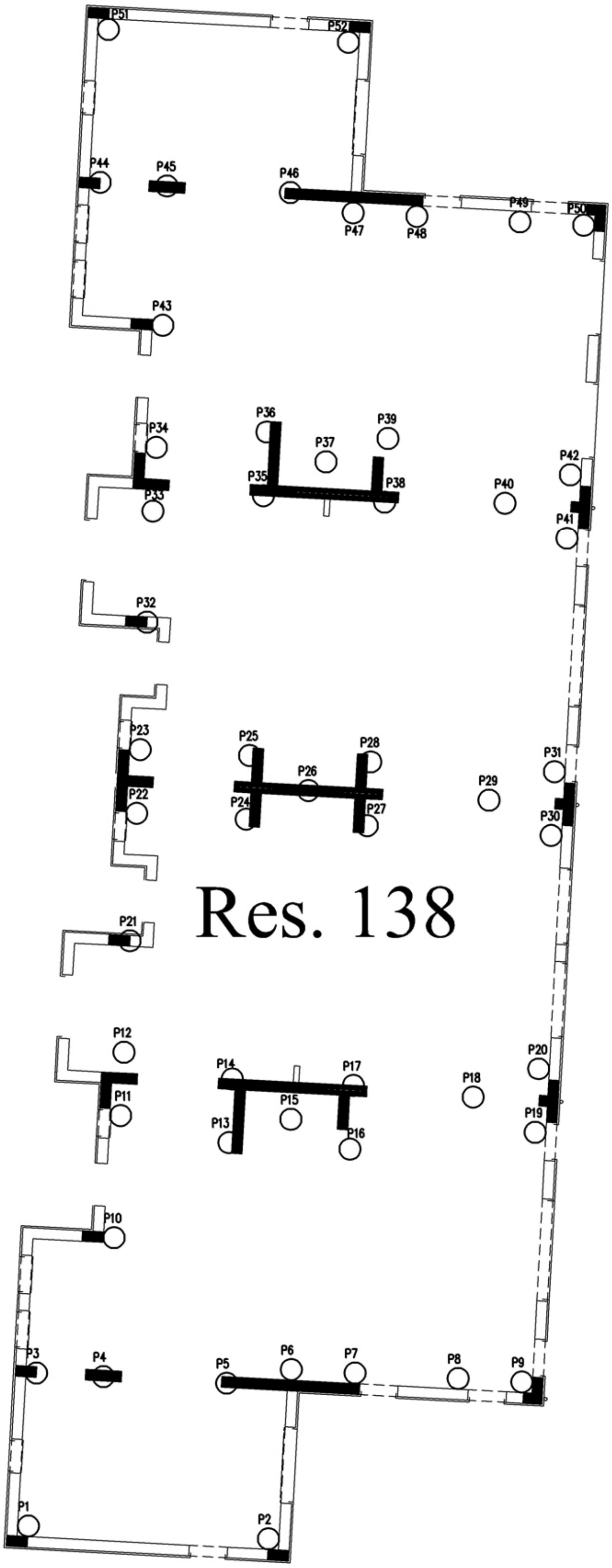


Figure 6.3: Piling plan of Residence 138

Residence 139

Piling plan of residence numbered as 139 is shown in Figure 6.4. At this residence, 5 out of 19 piles (1, 3, 5, 8 and 13) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.4.

Table 6.4: Evaluation of SIT results for residence 139

Residence No:	139	Total Num. Of Piles:		19
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
1	50 cm	12 m	A	
3	50 cm	12 m	B	4 m
5	50 cm	12 m	B	3 m
8	50 cm	12 m	A	
13	50 cm	12 m	B	5 and 8 m

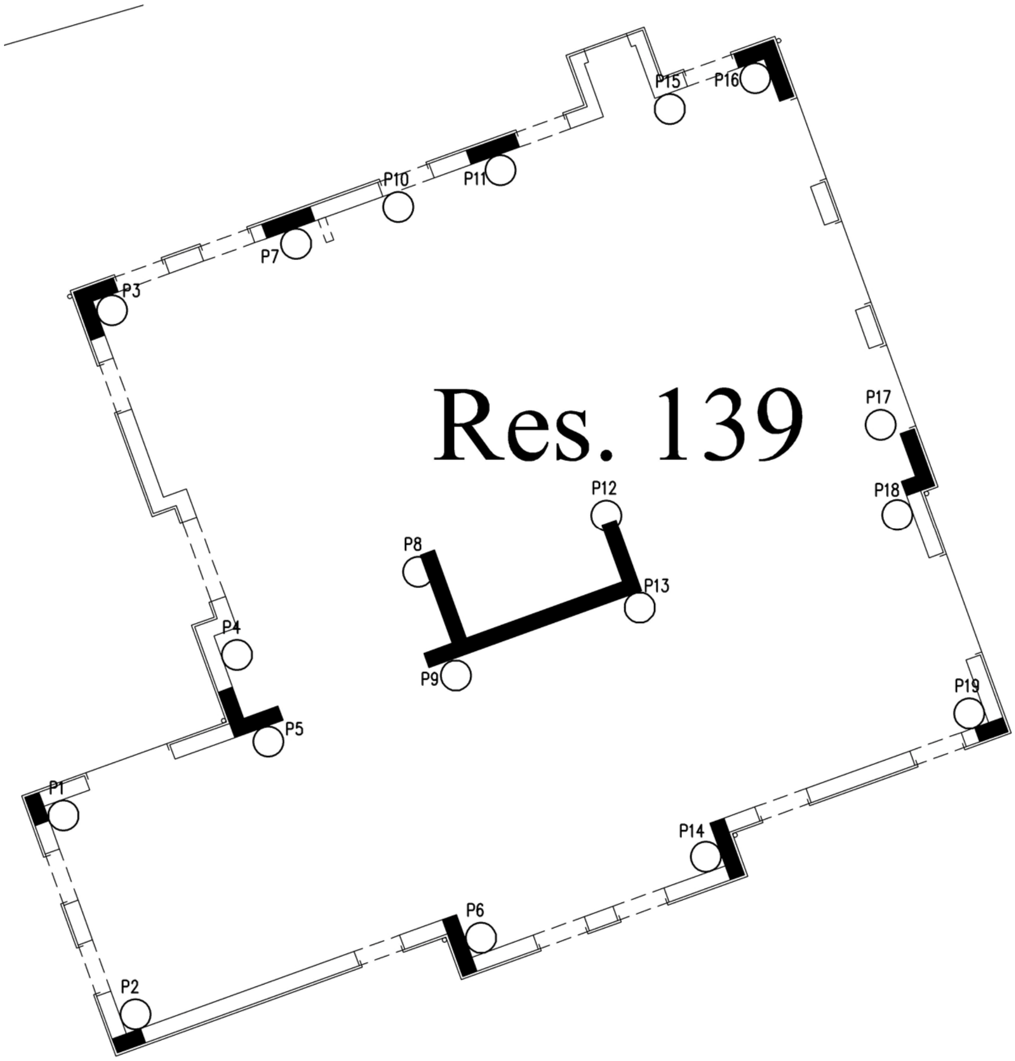


Figure 6.4: Piling plan of Residence 139

Residence 140

Piling plan of residence numbered as 140 is shown in Figure 6.5. At this residence, 4 out of 19 piles (1, 4, 9 and 17) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.5.

Table 6.5: Evaluation of SIT results for residence 140

Residence No: 140		Total Num. Of Piles:	19	
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
1	50 cm	12 m	B	3 m
4	50 cm	12 m	B	2,5 m
9	50 cm	12 m	B	2 m
17	50 cm	12 m	B	3 m

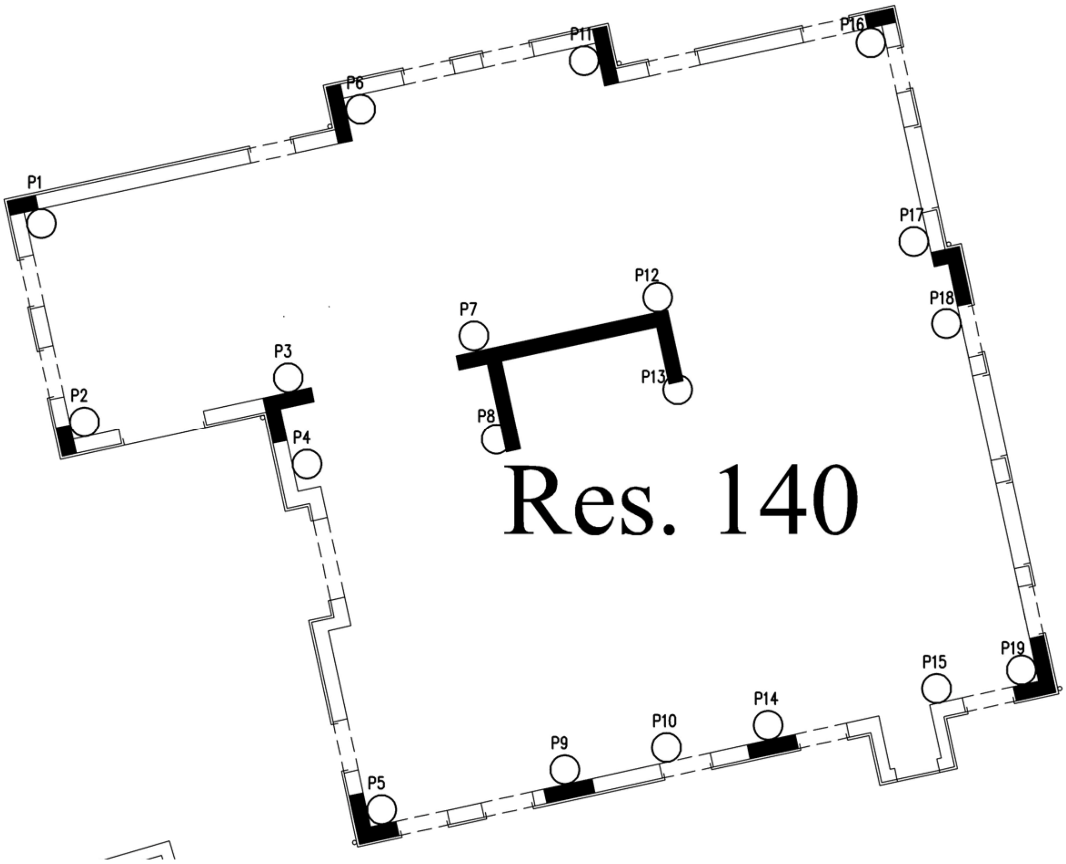


Figure 6.5: Piling plan of Residence 140

Residence 141

Piling plan of residence numbered as 141 is shown in Figure 6.6. At this residence, 5 out of 19 piles (6, 8, 11, 17 and 18) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.6.

Table 6.6: Evaluation of SIT results for residence 141

Residence No: 141		Total Num. Of Piles:	19	
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
6	50 cm	12 m	A	
8	50 cm	12 m	A	
11	50 cm	12 m	B	2 m
17	50 cm	12 m	A	
18	50 cm	12 m	B	3 m

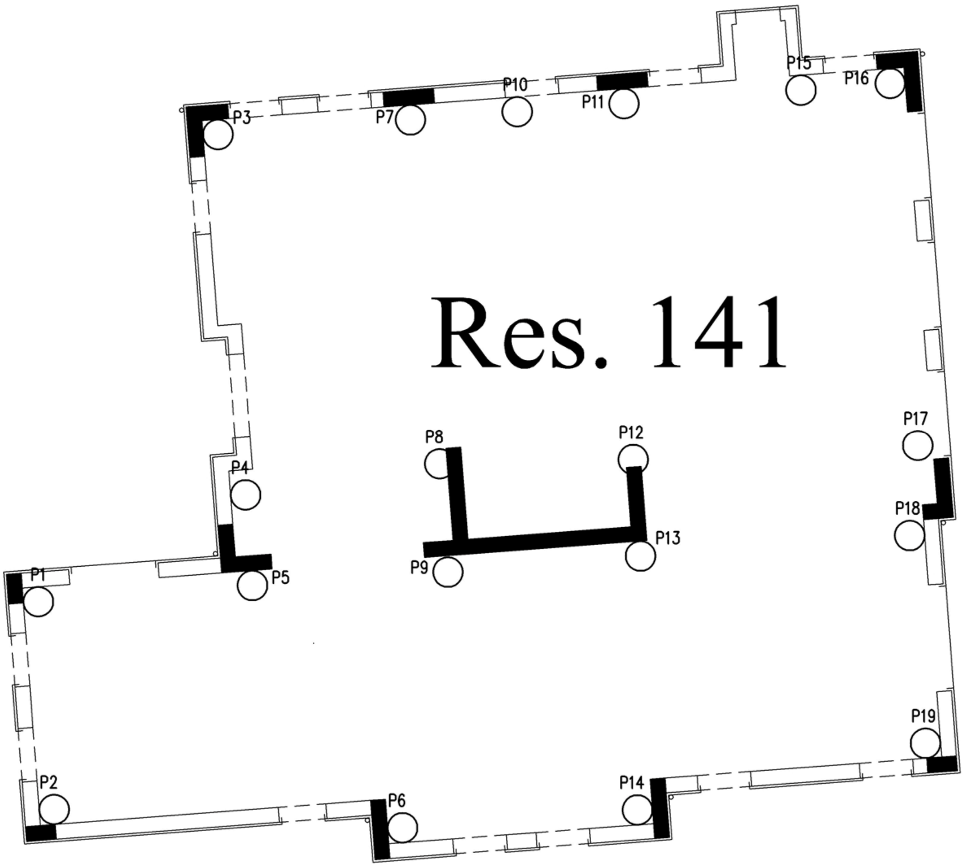


Figure 6.6: Piling plan of Residence 141

Residence 142

Piling plan of residence numbered as 142 is shown in Figure 6.7. At this residence, 4 out of 19 piles (4, 5, 12 and 19) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.7.

Table 6.7: Evaluation of SIT results for residence 142

Residence No: 142		Total Num. Of Piles:		19
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
4	50 cm	12 m	A	
5	50 cm	12 m	A	
12	50 cm	12 m	A	
19	50 cm	12 m	A	

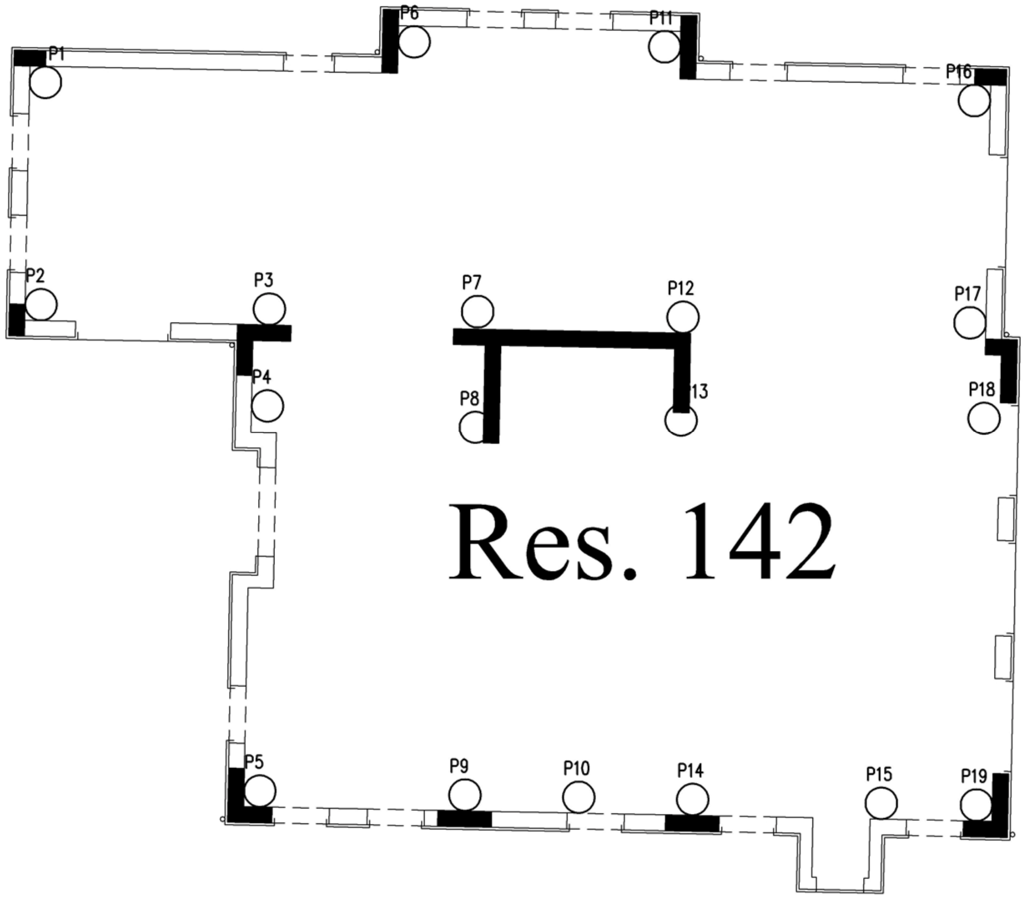


Figure 6.7: Piling plan of Residence 142

Residence 143

Piling plan of residence numbered as 143 is shown in Figure 6.8. At this residence, 12 out of 52 piles (2, 5, 7, 8, 9, 11, 18, 25, 32, 35, 42 and 50) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.8.

Table 6.8: Evaluation of SIT results for residence 143

Residence No:	143	Total Num. Of Piles:		52
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
2	50 cm	12 m	B	3 m
5	50 cm	12 m	B	2,5 m
7	50 cm	12 m	A	
8	50 cm	12 m	A	
9	50 cm	12 m	A	
11	50 cm	12 m	B	4 and 7 m
18	50 cm	12 m	B	3 m
25	50 cm	12 m	A	
32	50 cm	12 m	B	3,5 m
35	50 cm	12 m	B	3,5 m
42	50 cm	12 m	B	3,5 m
50	50 cm	12 m	A	

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

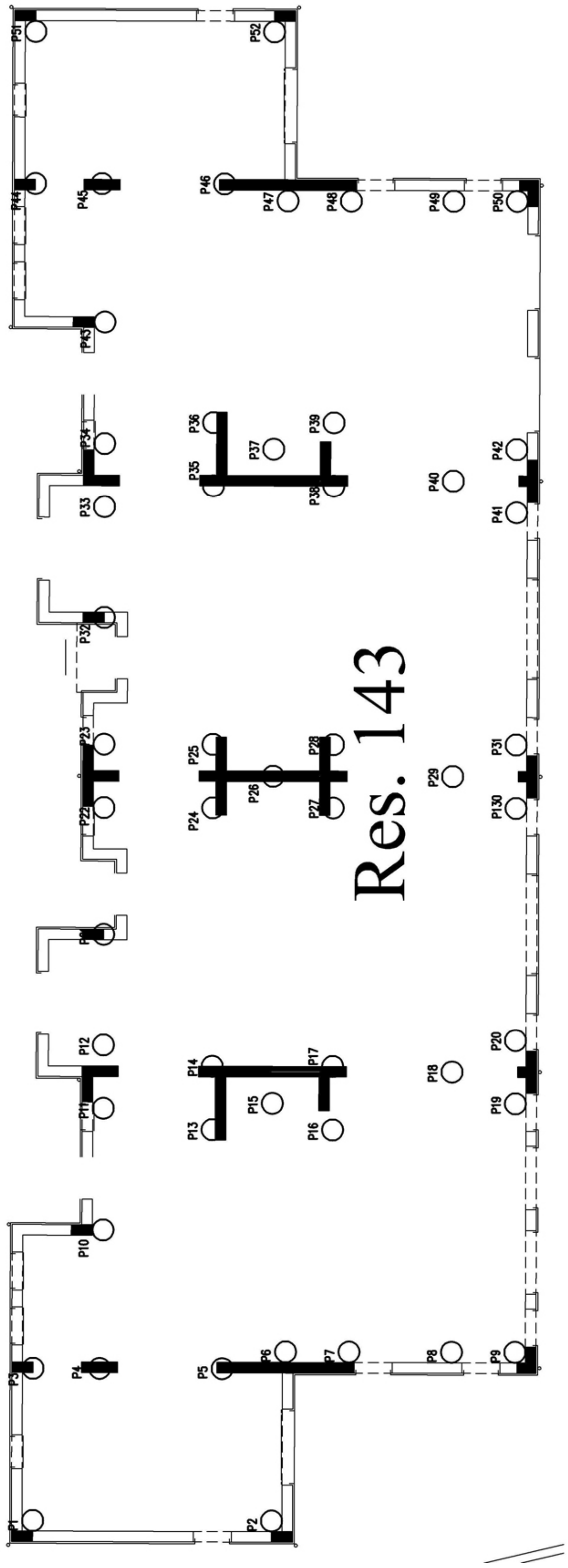


Figure 6.8: Piling plan of Residence 143

Residence 144

Piling plan of residence numbered as 144 is shown in Figure 6.9. At this residence, 18 out of 63 piles (2, 3, 4, 5, 6, 7, 8, 11, 13, 15, 17, 26, 29, 38, 42, 53, 54 and 61) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.9.

Table 6.9: Evaluation of SIT results for residence 144

Residence No: 144		Total Num. Of Piles:		63
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
2	50 cm	12 m	A	
3	50 cm	12 m	A	
4	50 cm	12 m	A	
5	50 cm	12 m	A	
6	50 cm	12 m	A	
7	50 cm	12 m	A	
8	50 cm	12 m	A	
11	50 cm	12 m	B	4 m
13	50 cm	12 m	B	3,5 m
15	50 cm	12 m	B	4,5 m
17	50 cm	12 m	A	
26	50 cm	12 m	B	4 m
29	50 cm	12 m	A	
38	50 cm	12 m	B	3,5 m
42	50 cm	12 m	B	3,5 m
53	50 cm	12 m	A	
54	50 cm	12 m	A	
61	50 cm	12 m	A	

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

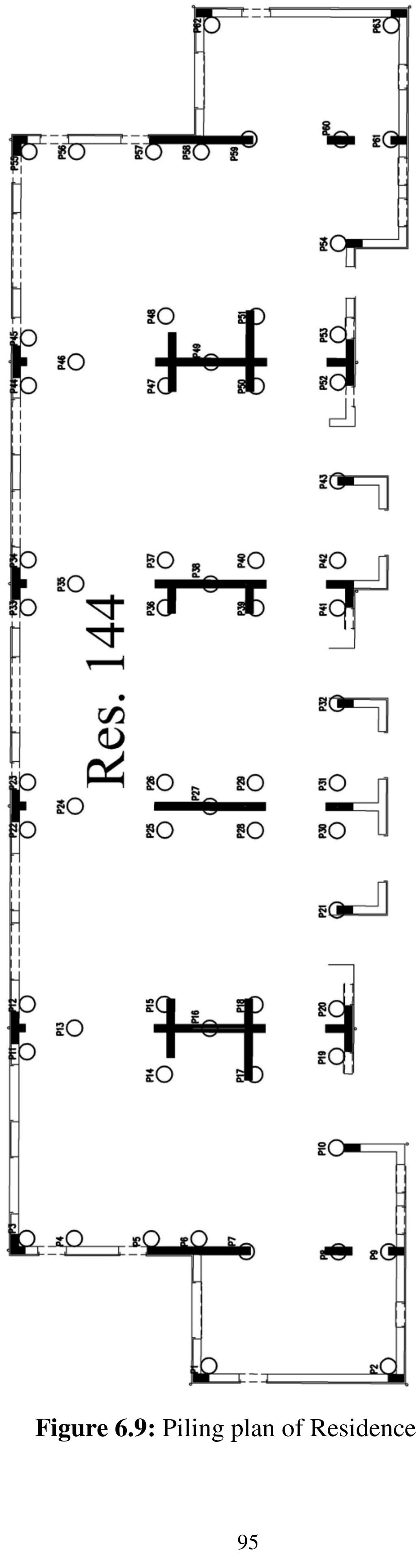


Figure 6.9: Piling plan of Residence 144

Residence 147

Piling plan of residence numbered as 147 is shown in Figure 6.10. At this residence, 7 out of 27 piles (1, 2, 5, 13, 19, 21 and 25) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.10.

Table 6.10: Evaluation of SIT results for residence 147

Residence No:	147	Total Num. Of Piles:		27
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
1	50 cm	12 m	A	
2	50 cm	12 m	A	
5	50 cm	12 m	A	
13	50 cm	12 m	B	4,5 m
19	50 cm	12 m	A	
21	50 cm	12 m	B	4,5 m
25	50 cm	12 m	B	6 m

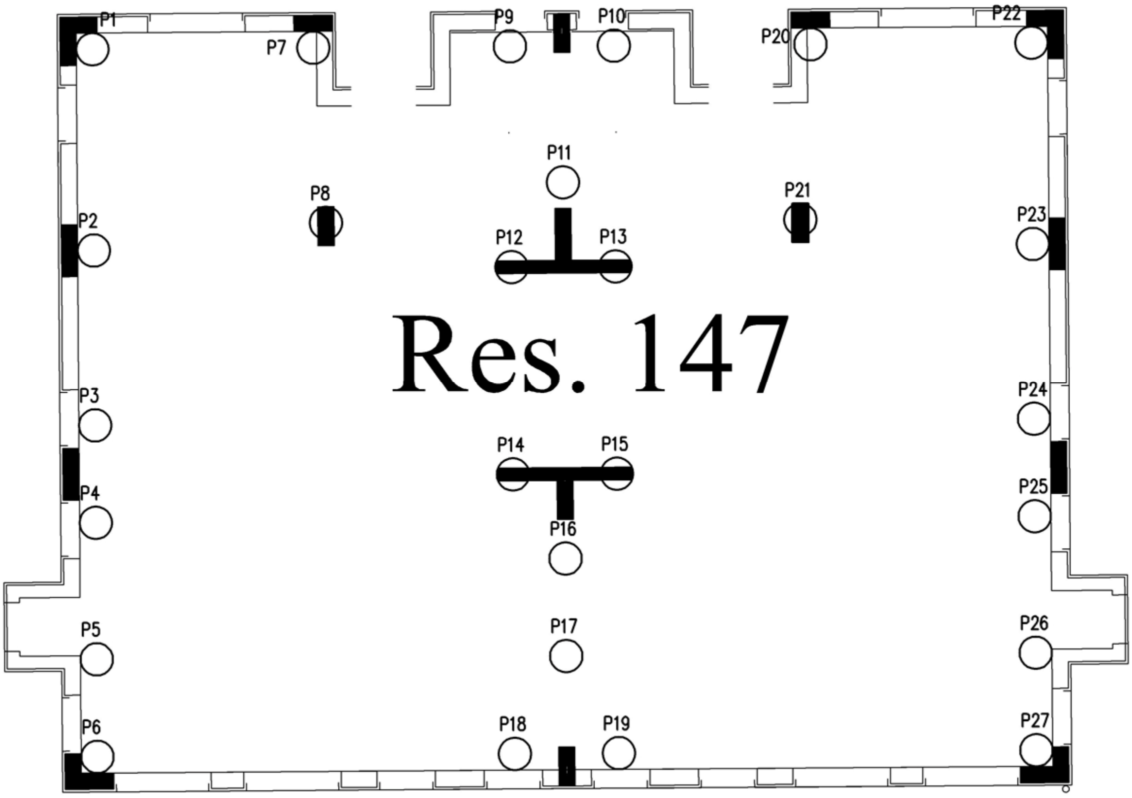


Figure 6.10: Piling plan of Residence 147

Residence 148

Piling plan of residence numbered as 148 is shown in Figure 6.11. At this residence, 7 out of 27 piles (1, 4, 5, 6, 8, 21 and 25) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.11.

Table 6.11: Evaluation of SIT results for residence 148

Residence No:	148	Total Num. Of Piles:		27
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
1	50 cm	12 m	B	6 m
4	50 cm	12 m	A	
5	50 cm	12 m	B	4,5 m
6	50 cm	12 m	A	
8	50 cm	12 m	A	
21	50 cm	12 m	B	4,5 m
25	50 cm	12 m	B	6 m

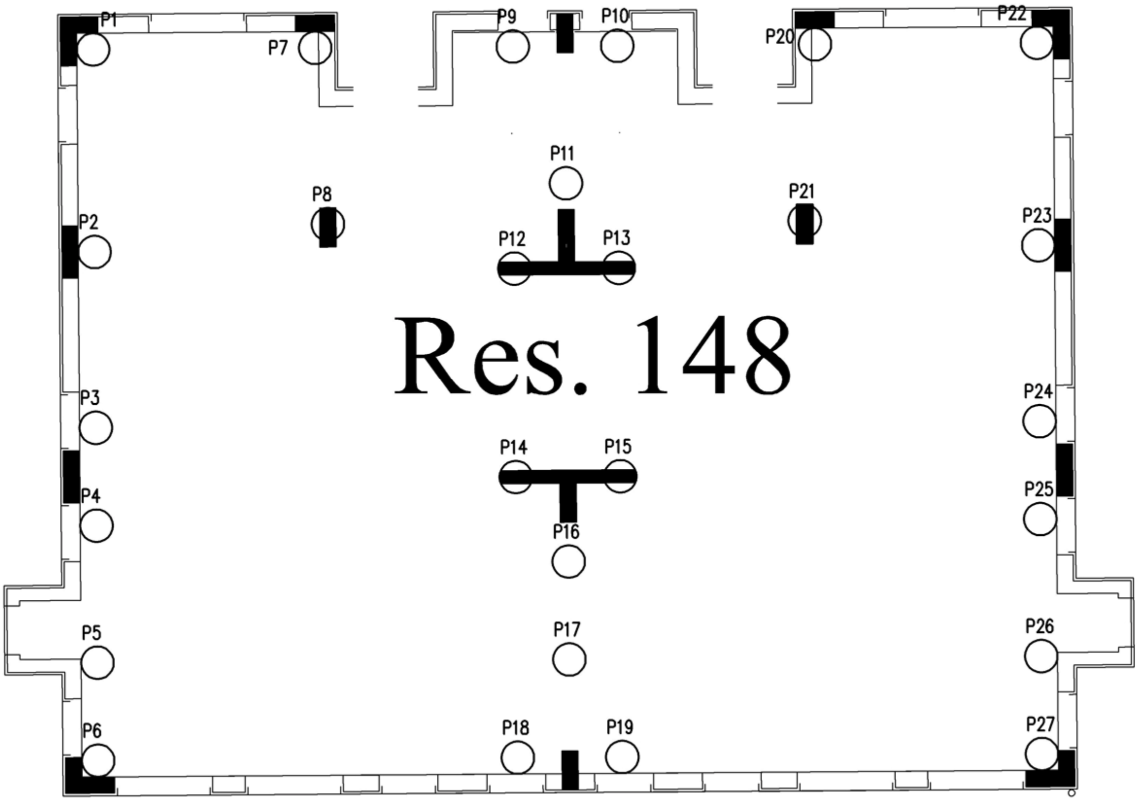


Figure 6.11: Piling plan of Residence 148

Residence 149

Piling plan of residence numbered as 149 is shown in Figure 6.12. At this residence, 7 out of 38 piles (1, 4, 5, 6, 7, 10 and 13) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.12.

Table 6.12: Evaluation of SIT results for residence 149

Residence No:	149	Total Num. Of Piles:		38
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
1	50 cm	15 m	B	10,5 m
4	50 cm	15 m	A	
5	50 cm	15 m	A	
6	50 cm	15 m	A	
7	50 cm	15 m	B	10 m
10	50 cm	15 m	A	
13	50 cm	15 m	B	15 m

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

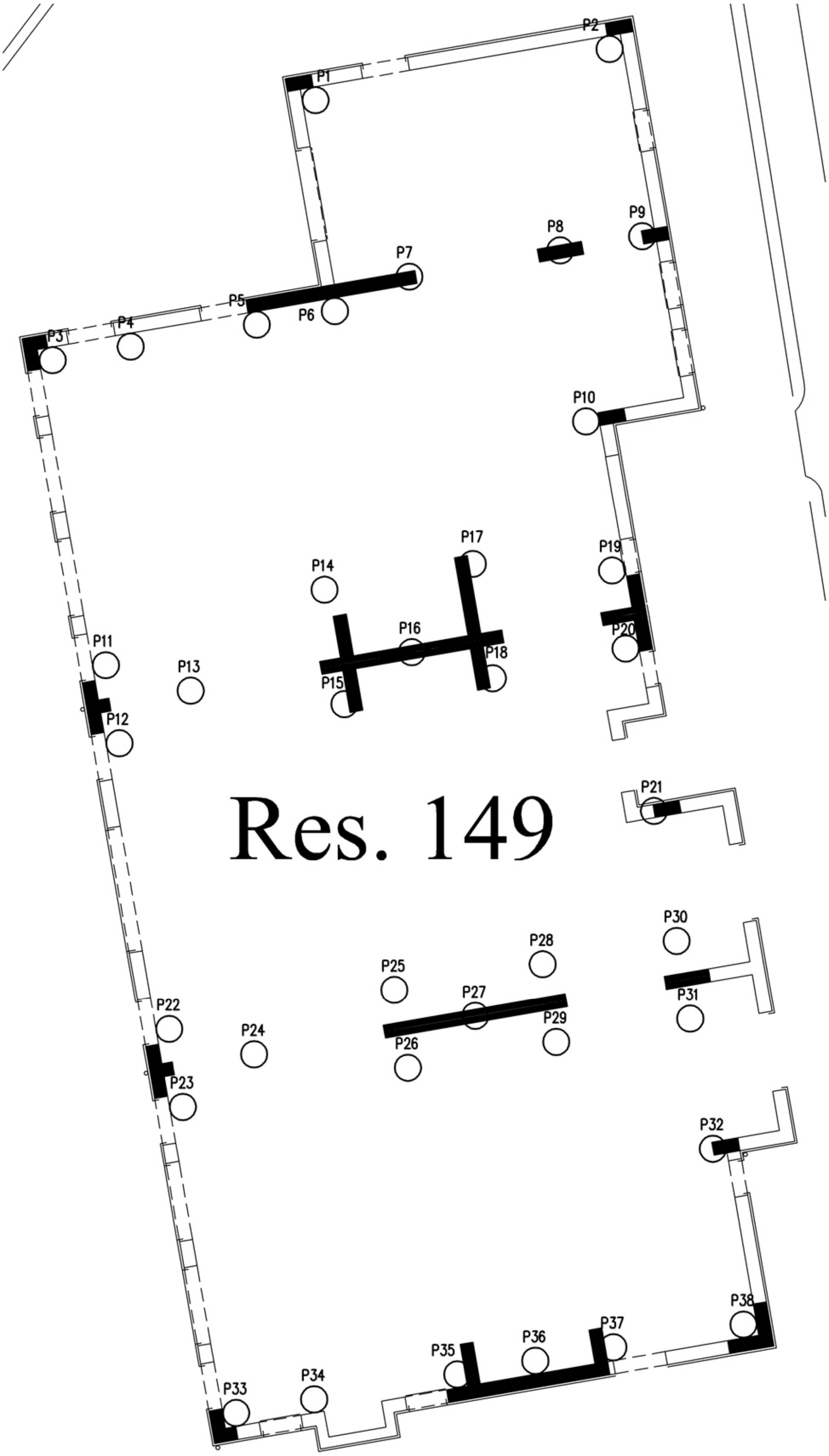


Figure 6.12: Piling plan of Residence 149

Residence 151

Piling plan of residence numbered as 151 is shown in Figure 6.13. At this residence, 11 out of 52 piles (5, 7, 10, 11, 12, 14, 23, 27, 35, 45 and 51) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.13.

Table 6.13: Evaluation of SIT results for residence 151

<table>
<tr><td colspan="2">Residence No: 151</td><td colspan="2">Total Num. Of Piles:</td><td>52</td></tr>
<tr><td colspan="5">Tested Piles</td></tr>
<tr><td>Pile No.</td><td>Pile Diameter</td><td>Pile Length</td><td>Discontinuity Type</td><td>Discontinuity Loc. (Distance from Top)</td></tr>
<tr><td>5</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>7</td><td>50 cm</td><td>15 m</td><td>B</td><td>12 m</td></tr>
<tr><td>10</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>11</td><td>50 cm</td><td>15 m</td><td>B</td><td>8 m</td></tr>
<tr><td>12</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>14</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>23</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>27</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>35</td><td>50 cm</td><td>15 m</td><td>A</td><td></td></tr>
<tr><td>45</td><td>50 cm</td><td>15 m</td><td>B</td><td>9 m</td></tr>
<tr><td>51</td><td>50 cm</td><td>15 m</td><td>B</td><td>5,5 m</td></tr>
</table>

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

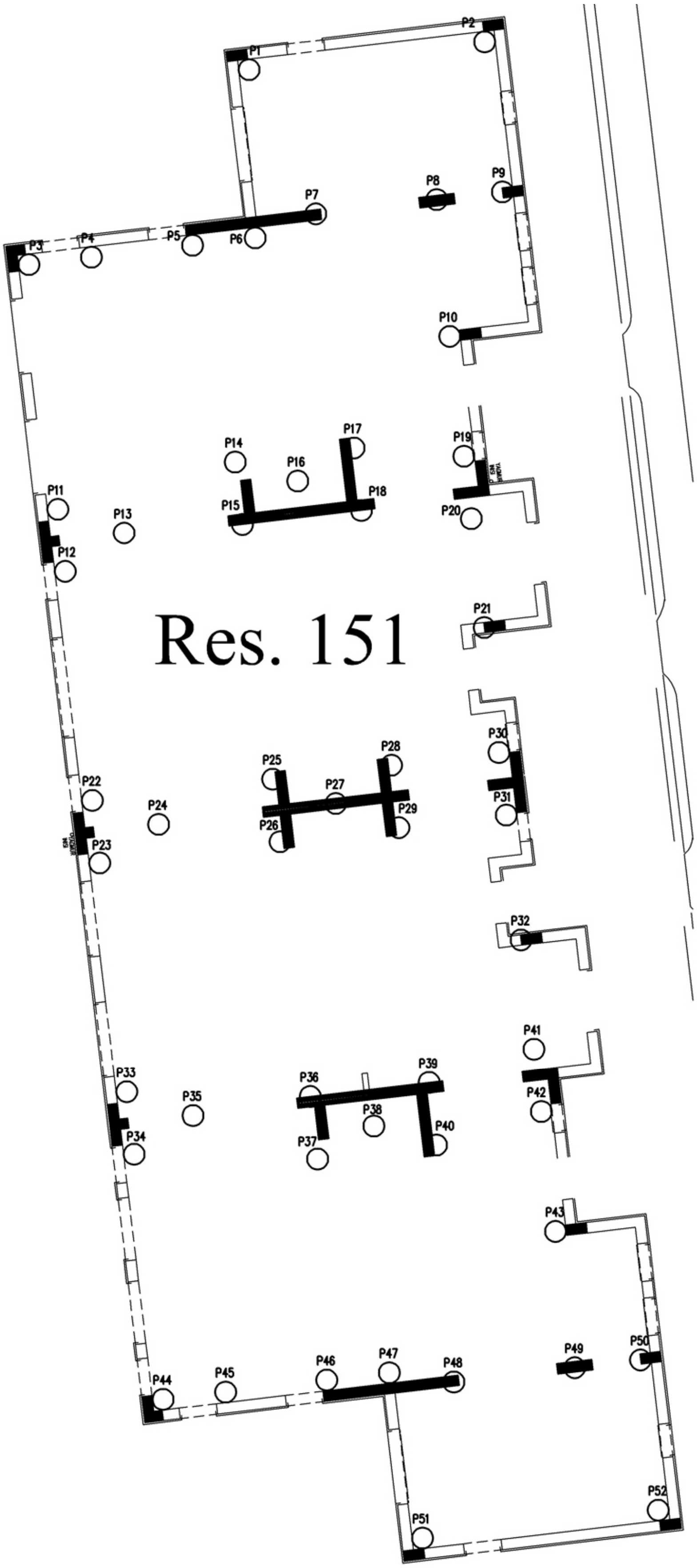


Figure 6.13: Piling plan of Residence 151

Residence 152

Piling plan of residence numbered as 152 is shown in Figure 6.14. At this residence, 14 out of 52 piles (2, 4, 7, 9, 10, 15, 17, 19, 24, 27, 32, 34, 39 and 42) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.14.

Table 6.14: Evaluation of SIT results for residence 152

Residence No: 152		Total Num. Of Piles:		52
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
2	50 cm	15 m	A	
4	50 cm	15 m	B	11 m
7	50 cm	15 m	B	9 m
9	50 cm	15 m	C	
10	50 cm	15 m	A	
15	50 cm	15 m	B	7 m
17	50 cm	15 m	B	7,5 m
19	50 cm	15 m	A	
24	50 cm	15 m	B	8 m
27	50 cm	15 m	B	7 m
32	50 cm	15 m	B	10 m
34	50 cm	15 m	B	7 m
39	50 cm	15 m	A	
42	50 cm	15 m	B	11 m

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

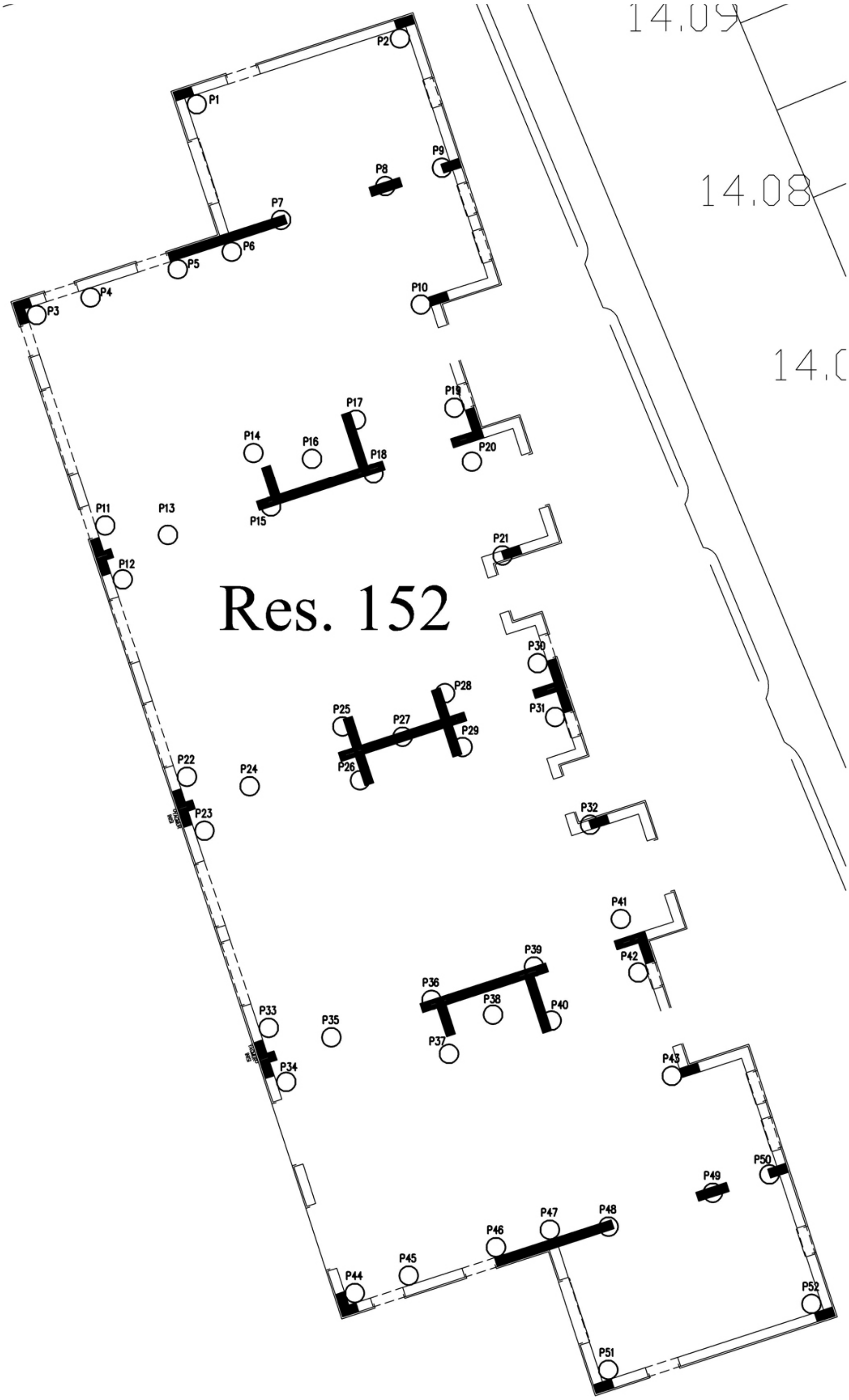


Figure 6.14: Piling plan of Residence 152

Residence 153

Piling plan of residence numbered as 153 is shown in Figure 6.15. At this residence, 20 out of 92 piles (8, 12, 17, 24, 26, 30, 34, 35, 41, 42, 46, 53, 62, 64, 68, 73, 81 83, 86 and 91) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.15.

Table 6.15: Evaluation of SIT results for residence 153

Residence No:	153	Total Num. Of Piles:		92
Tested Piles				
Pile No.	Pile Diameter	Pile Length	Discontinuity Type	Discontinuity Loc. (Distance from Top)
8	50 cm	13 m	A	
12	50 cm	13 m	A	
17	50 cm	13 m	B	6 m
24	50 cm	13 m	A	
26	50 cm	13 m	A	
30	50 cm	13 m	A	
34	50 cm	13 m	A	
36	50 cm	13 m	A	
41	50 cm	13 m	A	
42	50 cm	13 m	A	
46	50 cm	13 m	A	
53	50 cm	13 m	A	
62	50 cm	13 m	A	
64	50 cm	13 m	A	
68	50 cm	13 m	A	
73	50 cm	13 m	A	
81	50 cm	13 m	A	
83	50 cm	13 m	A	
86	50 cm	13 m	A	
91	50 cm	13 m	B	7 m

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

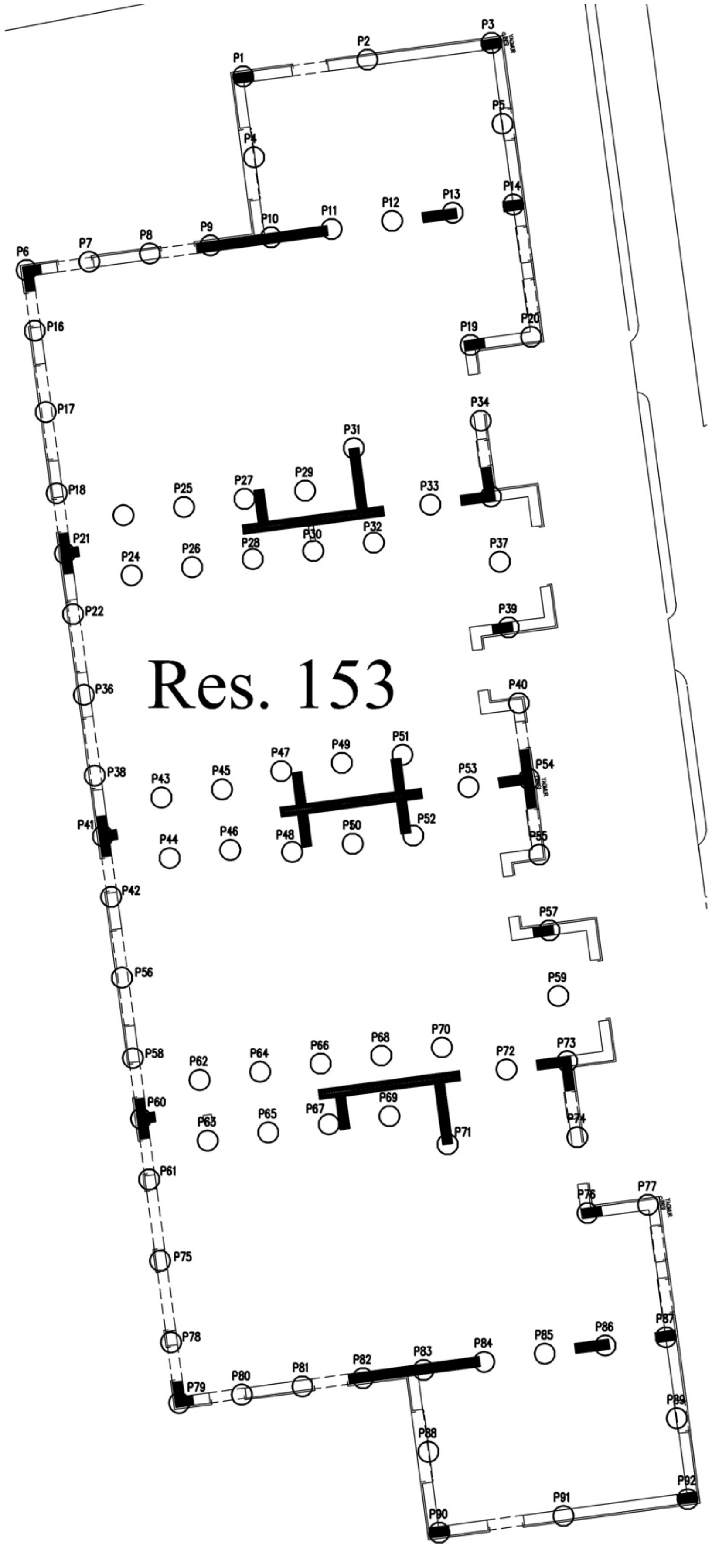


Figure 6.15: Piling plan of Residence 153

Residence 154

Piling plan of residence numbered as 154 is shown in Figure 6.16. At this residence, 15 out of 72 piles (2, 3, 7, 10, 12, 15, 18, 19, 21, 25, 29, 30, 32, 45 and 49) were chosen for Sonic Integrity Testing. Evaluation of SIT results for these piles are given in Table 6.16.

Table 6.16: Evaluation of SIT results for residence 154

<table>
<tr><td colspan="2">Residence
No: 154</td><td colspan="2">Total Num. Of Piles:</td><td>72</td></tr>
<tr><td colspan="5"></td></tr>
<tr><th>Pile No.</th><th>Pile Diameter</th><th>Pile Length</th><th>Discontinuity Type</th><th>Discontinuity Loc. (Distance from Top)</th></tr>
<tr><td>2</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>3</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>7</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>10</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>12</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>15</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>18</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>19</td><td>50 cm</td><td>12 m</td><td>B</td><td>6 m</td></tr>
<tr><td>21</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>25</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>29</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>30</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>32</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
<tr><td>45</td><td>50 cm</td><td>12 m</td><td>B</td><td>4 m</td></tr>
<tr><td>49</td><td>50 cm</td><td>12 m</td><td>A</td><td></td></tr>
</table>

Note : Description of discontinuity types are:

A : No discontinuity

B : Enlargement in cross-section

C : Decrement in cross-section

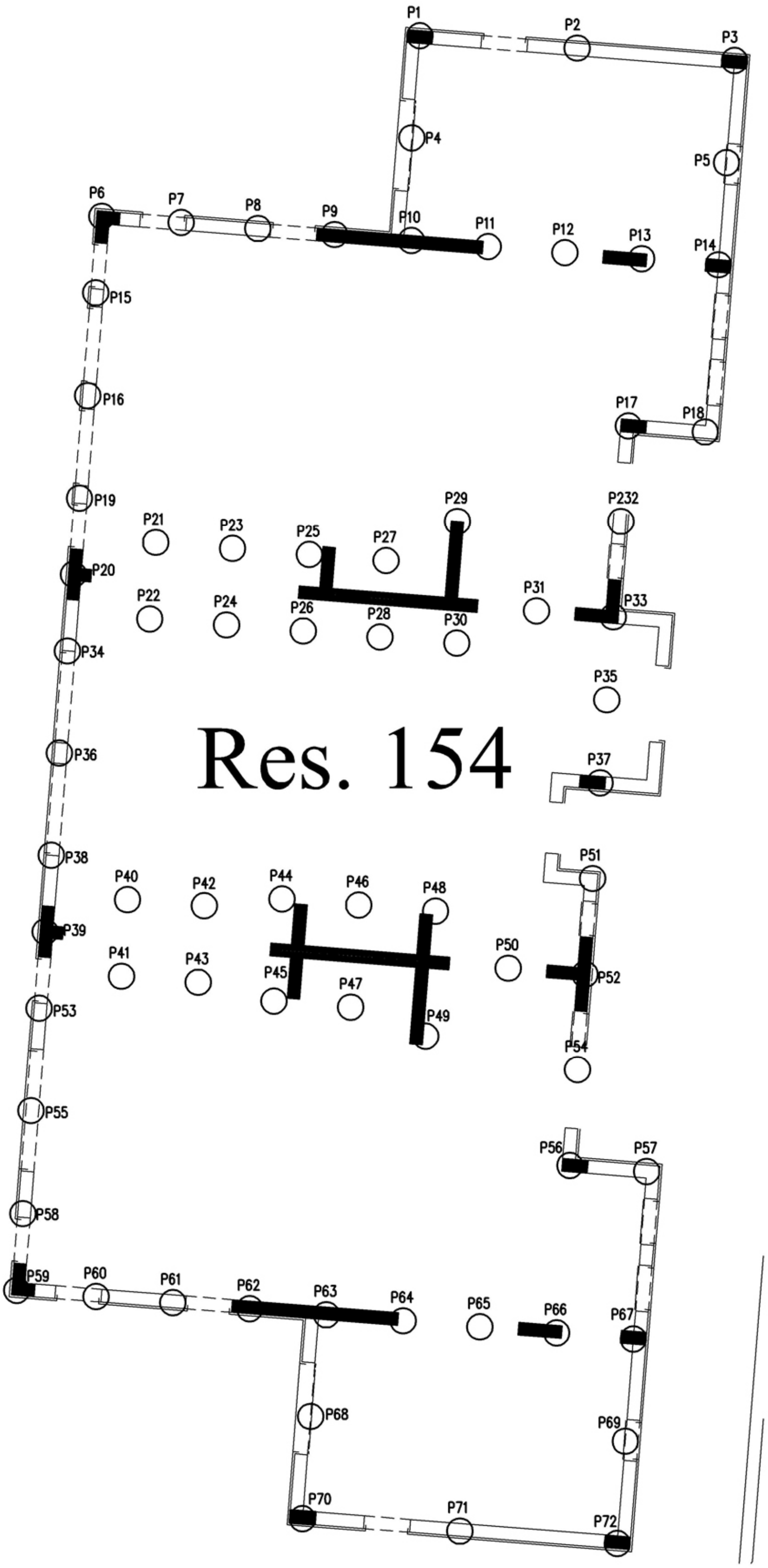


Figure 6.16: Piling plan of Residence 154

The distribution of discontinuities respect to residences determined by SIT test, performed on the field, can be seen in Table 6.17. Furthermore, sum of these discontinuities according to discontinuity type and their percentages, which respect to total number of tested piles, are given in Table 6.18 and shown as a graph in Figure 6.17.

Table 6.17: Discontinuity types and quantities for each residence

Res. No	Total Number of Piles	Number of Tested Piles	Number of Piles According to Discontinuity Type		
			A	B	C
136	41	8	1	6	1
137	41	15	6	9	0
138	52	14	2	12	0
139	19	5	2	3	0
140	19	4	0	4	0
141	19	5	3	2	0
142	19	4	4	0	0
143	52	12	5	7	0
144	63	18	12	6	0
147	27	7	4	3	0
148	27	7	3	4	0
149	38	7	4	3	0
151	52	11	7	4	0
152	52	14	4	9	1
153	92	20	18	2	0
154	72	15	13	2	0
Totals:	**685**	**166**	**88**	**76**	**2**

Table 6.18: Number of discontinuities and percentages according to discontinuity type

Discontinuity Type	Number of Piles	Percentage
A	88	%53,01
B	76	%45,78
C	2	%1,20
Total=	**166**	

With the assistance of soil profiles derived from boring logs, simple soil sections of each residence had been drawn and total number of piles under these residences, number of tested piles and number of “B” (enlargement in cross-section) and “C” (decrement in cross-section) type discontinuities experienced at each soil layer under stated residences were entered to the drawing as shown in Figure 6.18.

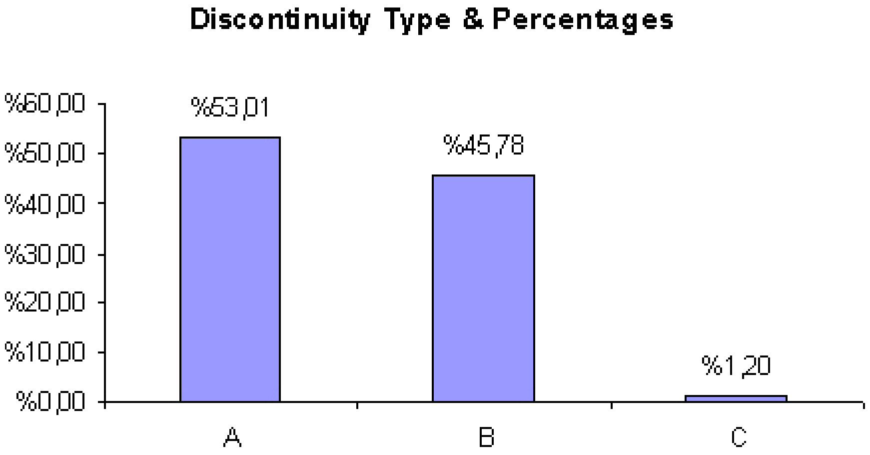


Figure 6.17: Discontinuity type and their percentages

The detailed format of Figure 6.18 is shown in Figure 6.19. At this drawing, discontinuity types, number of discontinuities and their locations can be seen more clearly. The blue color represents the “B” (enlargement in cross-section) type discontinuity and the red color represents the “C” (decrement in cross-section) type discontinuity. Number at the centre of each box represents the number of discontinuity at that location. Orange, gray and green colors represent the soil layers as same with Figure 6.18.

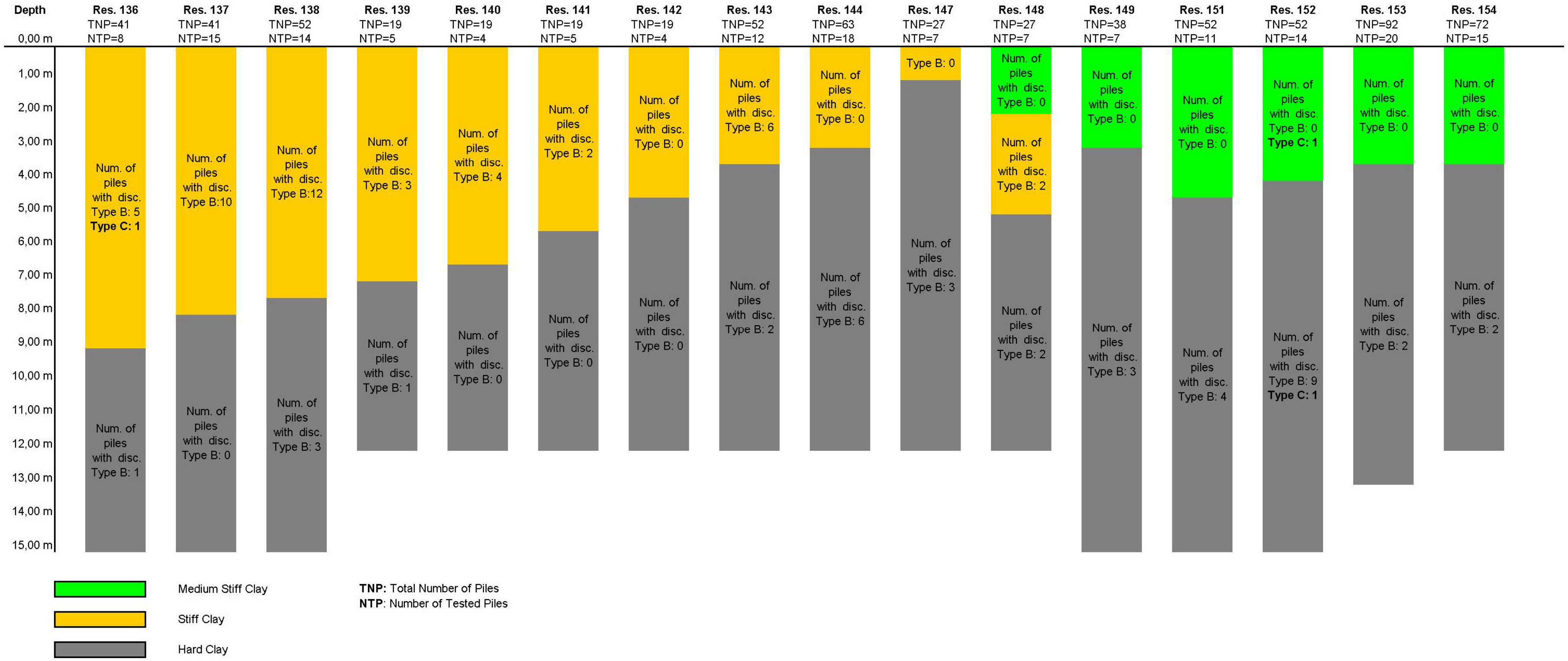


Figure 6.18: Simple soil section of each residence and discontinuities experienced

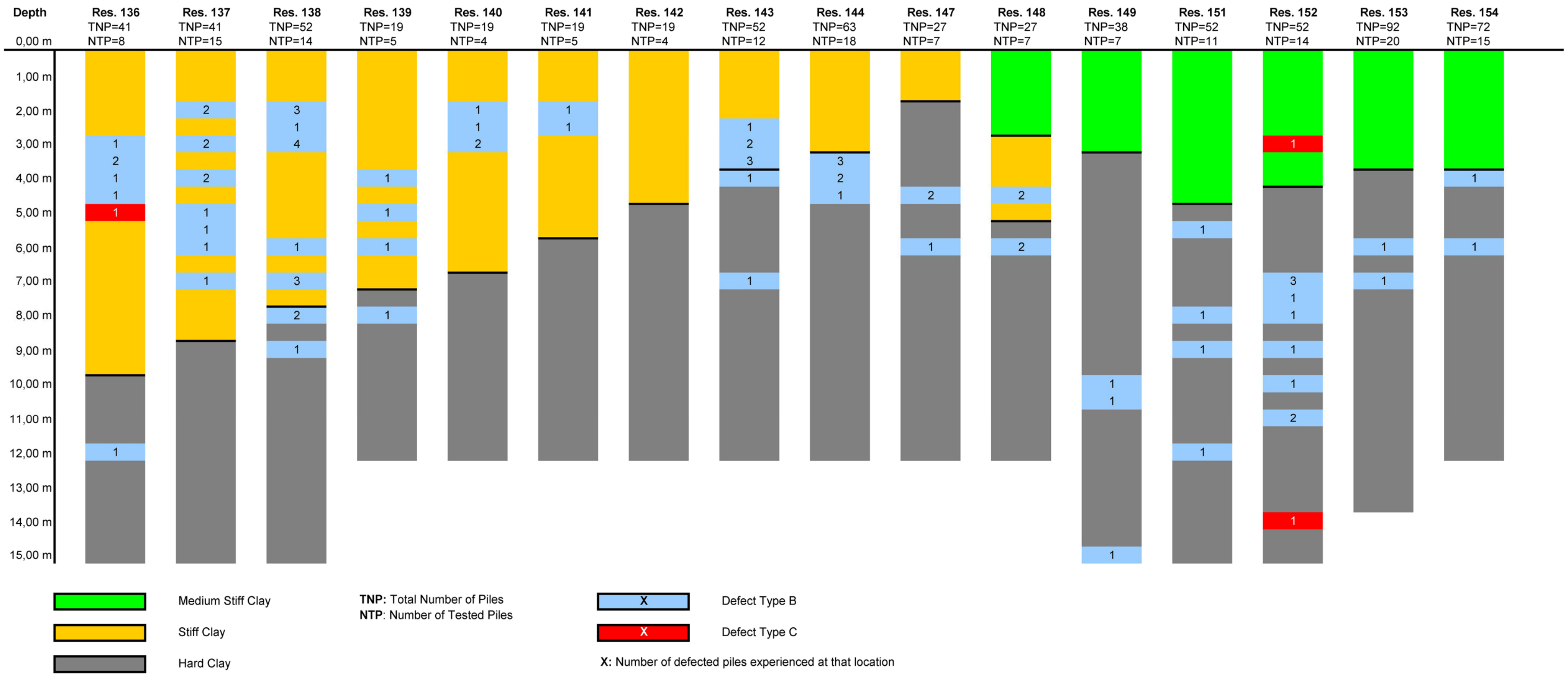


Figure 6.19: Distributions of discontinuities experienced under each residence

From Figure 6.18 and 6.19, number of discontinuities experienced in each soil layer were determined and presented in Figure 6.20. As shown in Figure 6.20, 54% of “B” type discontinuity was experienced in Stiff Clay layers and the rest (46%) was experienced in Hard Clay. No “B” type of discontinuity was experienced in Medium Stiff Clay layer. Three “C” type of discontinuity was experienced in three soil types with the same percentages.

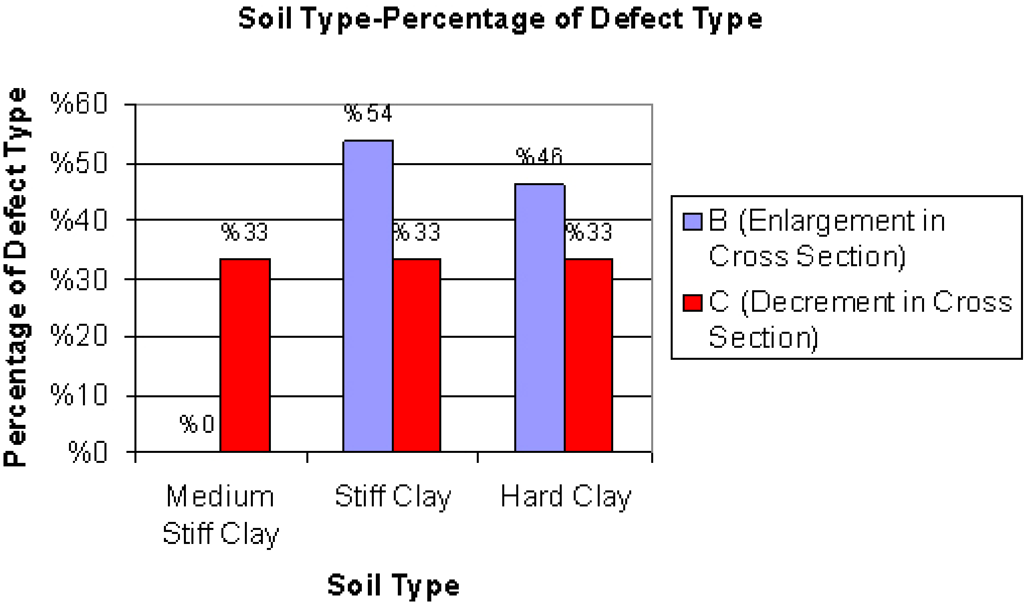


Figure 6.20: Soil Type – Percentages of Defect type experienced

Moreover, the depth-number of discontinuities relation for “B” (enlargement in cross-section) type of discontinuity was investigated. In this investigation, it has been observed that most of the “B” type discontinuities were experienced in depth range of 1,5 m to 4,5 m as shown in Figure 6.21. As in this figure, it can be seen that 44 out of 82 (approximately 54%) of “B” type discontinuities are detected at that depth range between 1,5 m and 4,5 m. The percentages of discontinuities according to depth range are shown in Figure 6.22.

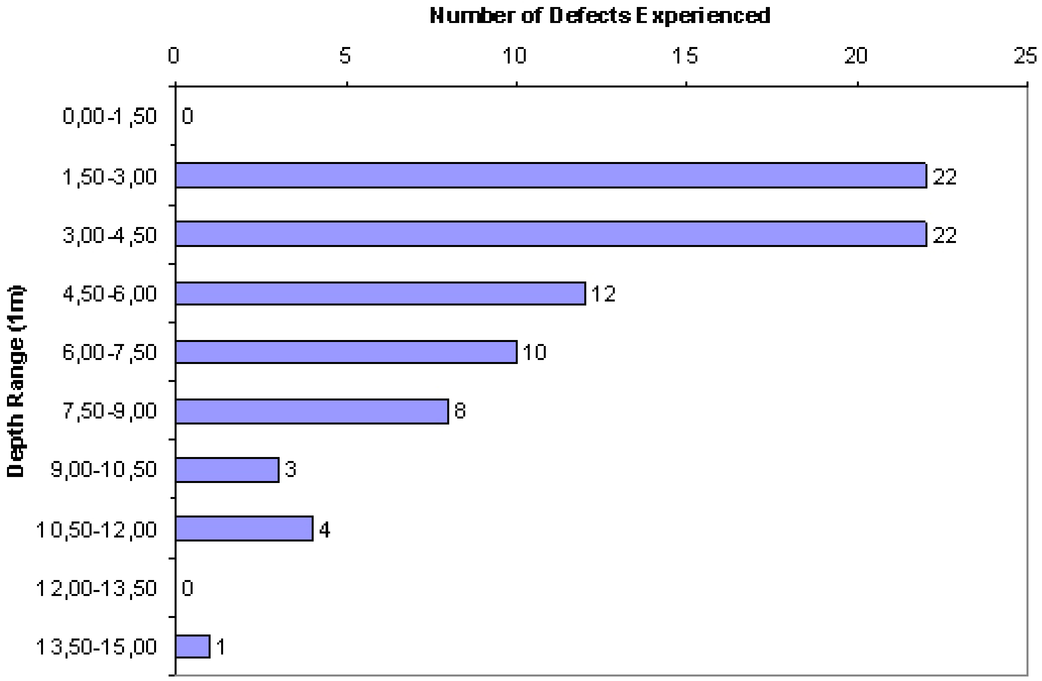


Figure 6.21: Depth Range – Number of Defects Experienced

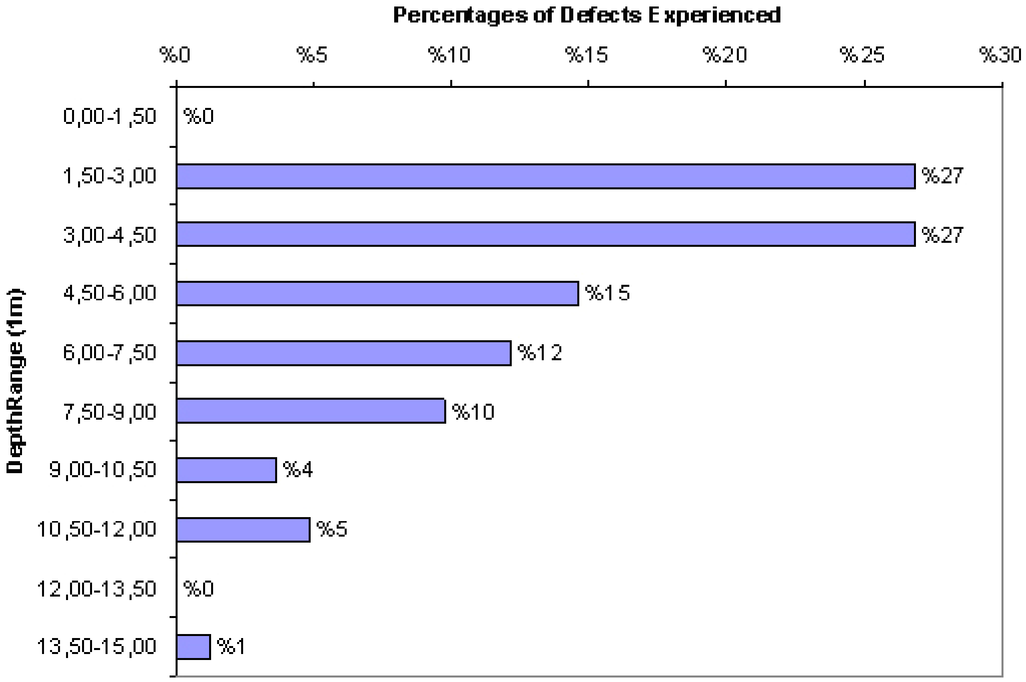


Figure 6.22: Depth Range – Percentages of Defects Experinced

7 CONCLUSION

In this study, 166 of 685 (approximately 24%) piles, constructed under foundations of 16 residences numbered as 136, 137, 138, 139, 140, 141, 142, 143, 144, 147, 148, 149, 151, 152, 154 and 154, which are in scope of housing project at location of Istanbul city, Esenyurt district, were tested with “Sonic Integrity Testing” (SIT) and discontinuities were aimed to be detected. Further analyses were performed to determine discontinuity types-location of discontinuity (depth)-soil type relation.

Soil at the investigated area is composed of clay layers with various stiffnesses. It is observed that at eastern part of the area, fist layer is medium stiff clay, with a thickness of 2-5 meters. At South-East, the second layer is observed as stiff clay with a thickness of 2-5 meter. hard clay layer is the third layer, if stiff clay is available, or the second layer, if stiff clay is not available.

At western part of the area, the soil profile is slightly different than the eastern part. The only difference is, medium stiff and stiff clays are not seen at North-West of the area but these layers were again experienced at southern part of west side with a thickness of less than eastern part.

In respect of soil profiles derived from boring logs, simple soil sections were drawn for each residence as shown in Figure 6.18. This figure also includes summary of discontinuities experienced in each soil layer under each residences. According to this data, it was determined that 54% of “B” (enlargement in cross-section) type discontinuities were experienced in stiff clay layer and 44% of them are experienced in hard clay. “B” type of discontinuity was not experienced in medium stiff clay layer. Moreover, “C” type of discontinuities were experienced in each soil layer with same percentages of 33%.

Further analyses were also performed with data obtained from test results to determine the depth-number of discontinuities relation for “B” (enlargement in cross-section) type discontinuity. In this investigation, it has been observed that most of the “B” type discontinuities were experienced in depth range of 1,5 m to 4,5 m as shown in Figure 6.21. From this figure, it can be seen that 44 out of 82 (approximately 54%) of “B” type discontinuities are detected at that depth range between 1,5 m and 4,5 m. The percentages of discontinuities according to depth range are presented in Figure 6.22. Combining this analyzes with soil layer-discontinuity type relation, resulted that 32 out of this 44 (approximately 73%) discontinuities, which were experienced in that depth range, is in stiff clay layer.

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APPENDIXES

APPENDIX A

PLAN VIEW OF HOUSING PROJECT

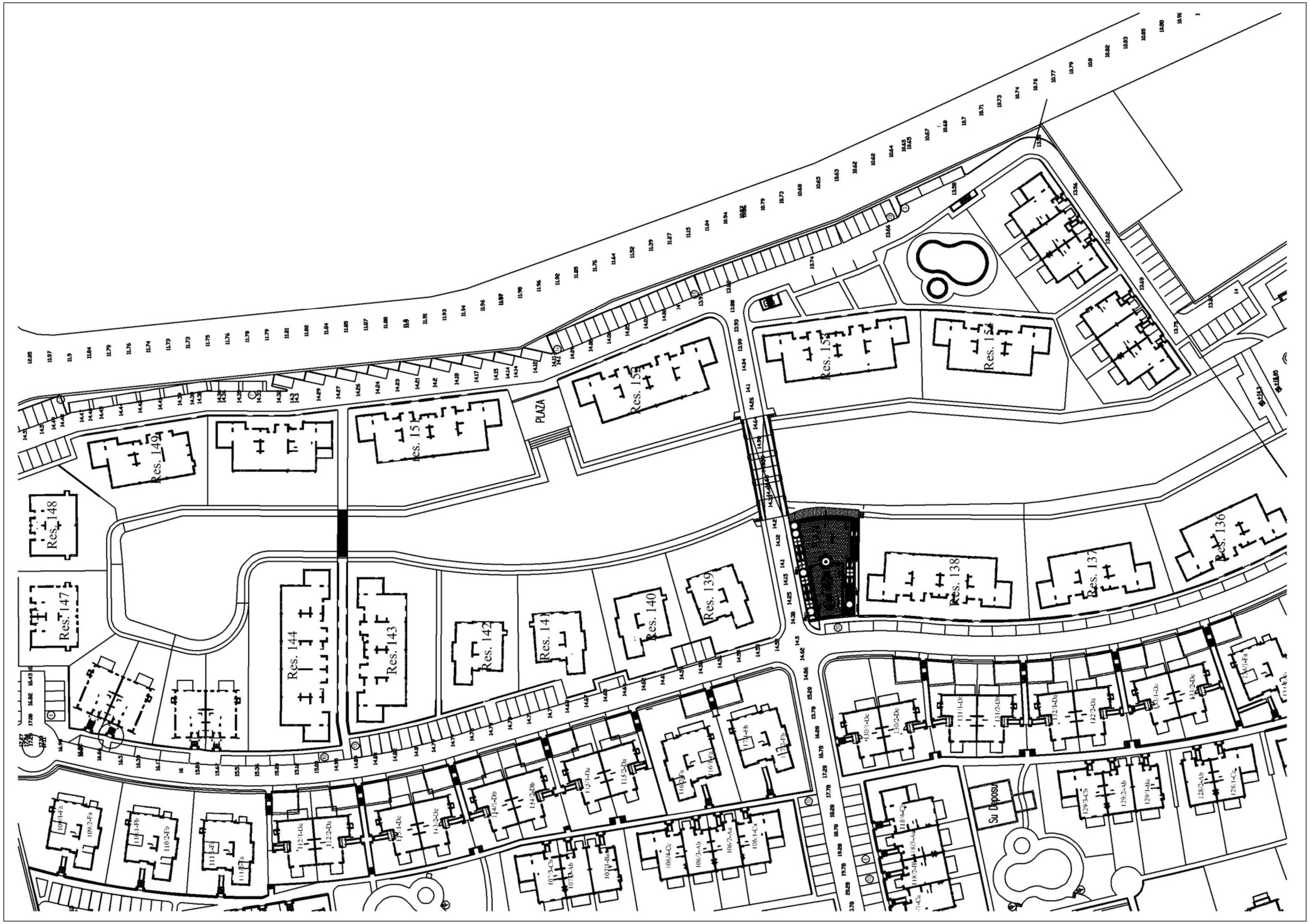


Figure A.1: Plan view of housing project

APPENDIX B
LOCATION OF BORINGS AND SOIL SECTIONS

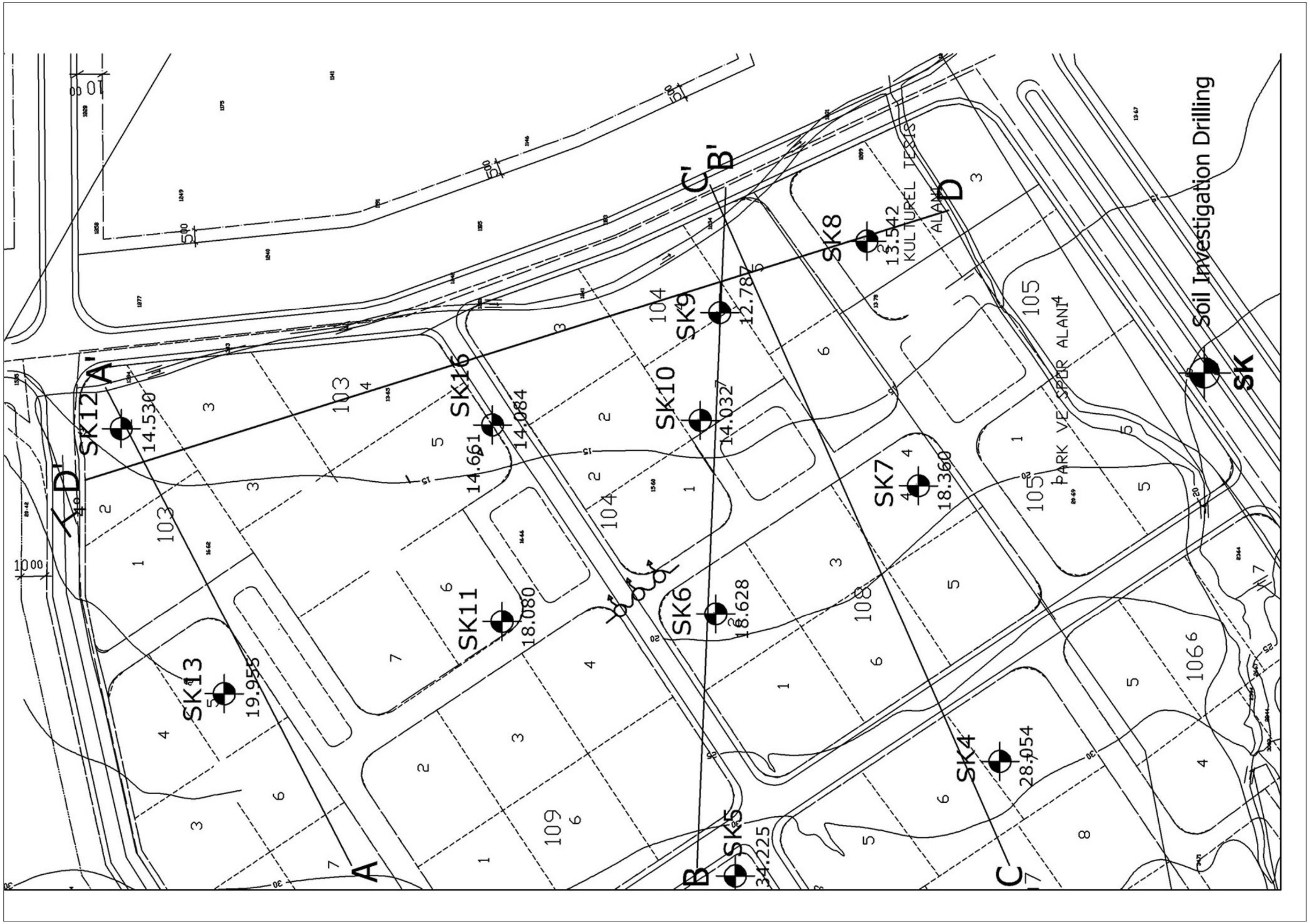
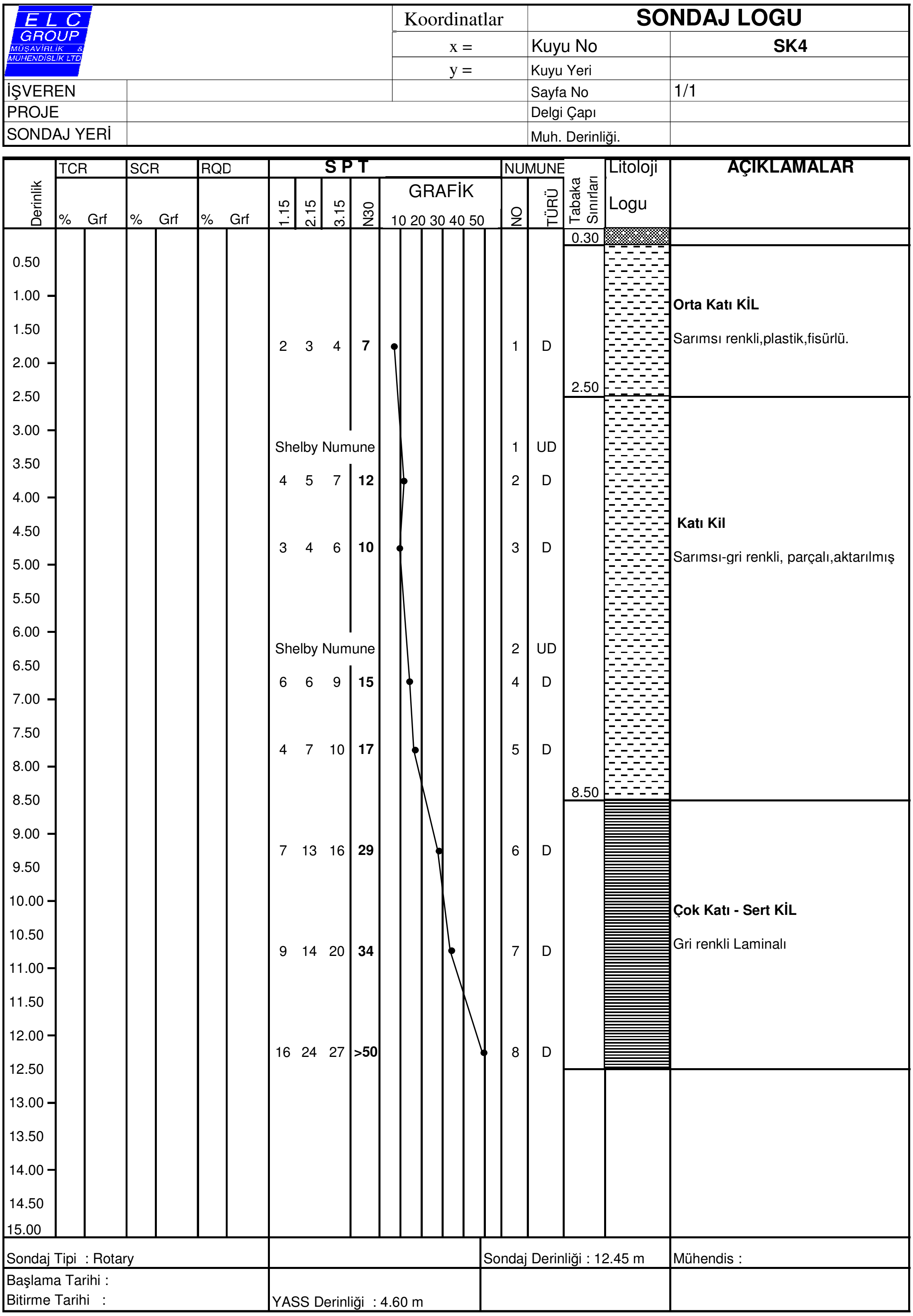


Figure B.1: Location of borings and soil sections

APPENDIX C
BORING LOGS



ELC GROUP MÜŞAVİRLİK & MÜHENDİSLİK LTD	Koordinatlar	SONDAJ LOGU	
	x =	Kuyu No	**SK4**
	y =	Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

TCR %	TCR Grf	SCR %	SCR Grf	RQD %	RQD Grf	SPT 1.15	SPT 2.15	SPT 3.15	N30	NUMUNE NO	NUMUNE TÜRÜ
						2	3	4	**7**	1	D
						Shelby Numune				1	UD
						4	5	7	**12**	2	D
						3	4	6	**10**	3	D
						Shelby Numune				2	UD
						6	6	9	**15**	4	D
						4	7	10	**17**	5	D
						7	13	16	**29**	6	D
						9	14	20	**34**	7	D
						16	24	27	**>50**	8	D

Tabaka Sınırları	AÇIKLAMALAR
0.30	
2.50	**Orta Katı KİL** Sarımsı renkli,plastik,fisürlü.
8.50	**Katı Kil** Sarımsı-gri renkli, parçalı,aktarılmış
	Çok Katı - Sert KİL Gri renkli Laminalı

Sondaj Tipi : Rotary

Sondaj Derinliği : 12.45 m

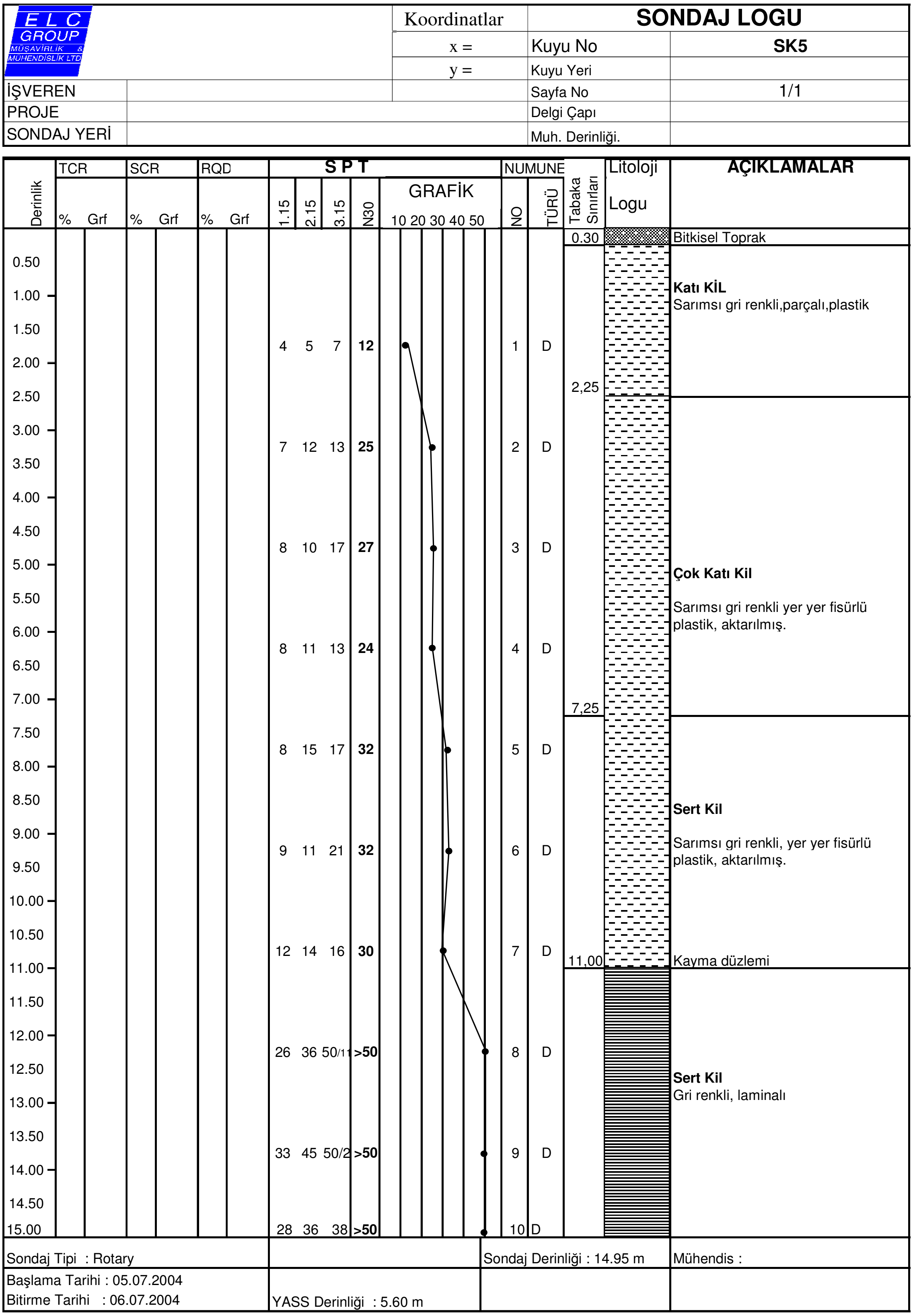
Mühendis :

Başlama Tarihi :

Bitirme Tarihi :

YASS Derinliği : 4.60 m

Figure C.1: Boring log of SK4



ELC GROUP
MÜŞAVİRLİK & MÜHENDİSLİK LTD

SONDAJ LOGU

	Koordinatlar		
	x =	Kuyu No	**SK5**
	y =	Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

Derinlik: 0.50, 1.00, 1.50, 2.00, 2.50, 3.00, 3.50, 4.00, 4.50, 5.00, 5.50, 6.00, 6.50, 7.00, 7.50, 8.00, 8.50, 9.00, 9.50, 10.00, 10.50, 11.00, 11.50, 12.00, 12.50, 13.00, 13.50, 14.00, 14.50, 15.00

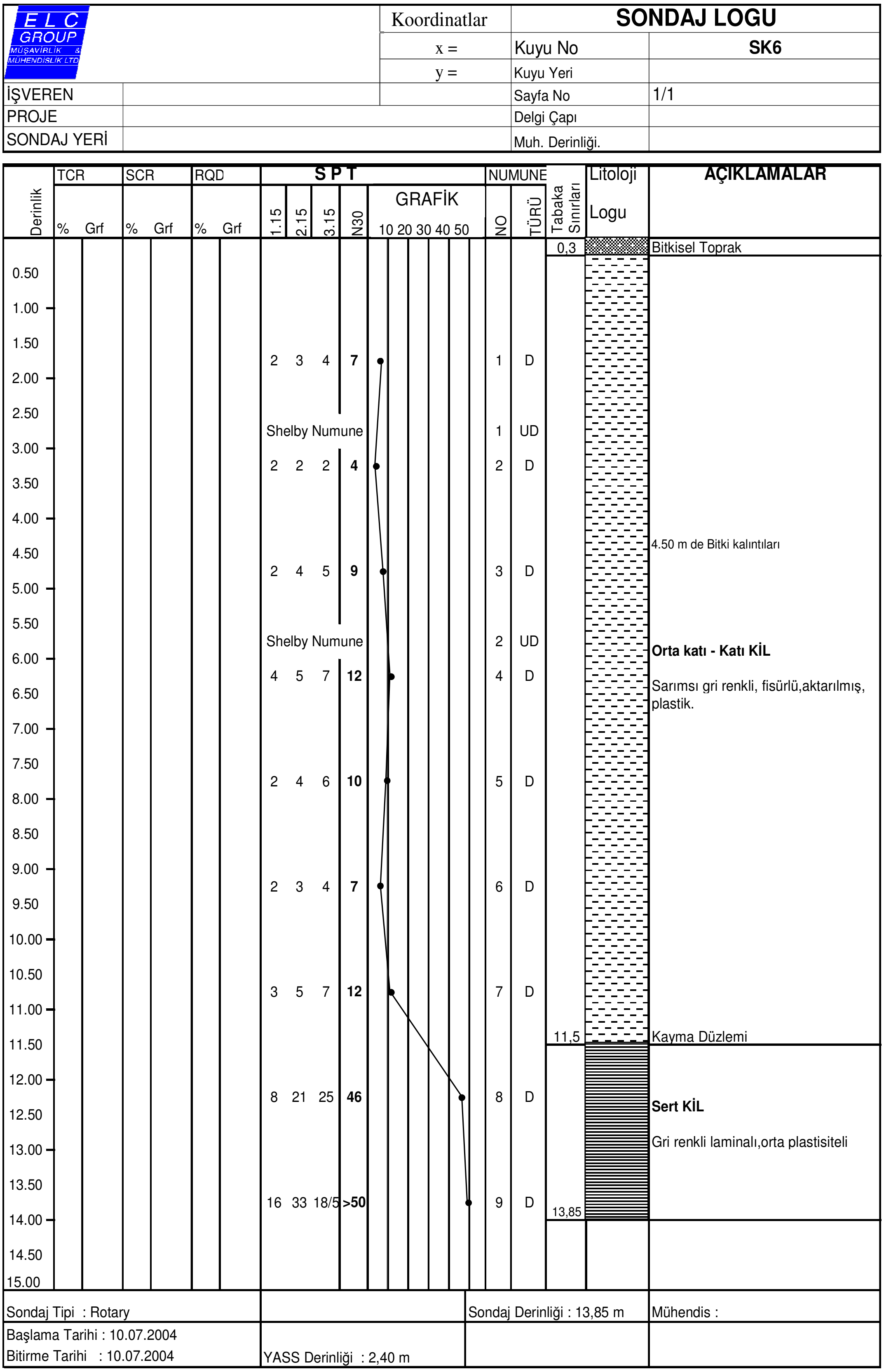
TCR % Grf | SCR % Grf | RQD % Grf

S P T				NUMUNE	
1.15	2.15	3.15	N30	NO	TÜRÜ
4	5	7	**12**	1	D
7	12	13	**25**	2	D
8	10	17	**27**	3	D
8	11	13	**24**	4	D
8	15	17	**32**	5	D
9	11	21	**32**	6	D
12	14	16	**30**	7	D
26	36	50/11	**>50**	8	D
33	45	50/2	**>50**	9	D
28	36	38	**>50**	10	D

Tabaka Sınırları	AÇIKLAMALAR
0.30	Bitkisel Toprak
2,25	**Katı KİL** Sarımsı gri renkli,parçalı,plastik
7,25	**Çok Katı Kil** Sarımsı gri renkli yer yer fisürlü plastik, aktarılmış.
11,00	**Sert Kil** Sarımsı gri renkli, yer yer fisürlü plastik, aktarılmış. Kayma düzlemi
	Sert Kil Gri renkli, laminalı

Sondaj Tipi : Rotary		Sondaj Derinliği : 14.95 m	Mühendis :
Başlama Tarihi : 05.07.2004 Bitirme Tarihi : 06.07.2004	YASS Derinliği : 5.60 m		

Figure C.2: Boring log of SK5



ELC GROUP MÜŞAVİRLİK & MÜHENDİSLİK LTD

Koordinatlar		SONDAJ LOGU	
x =		Kuyu No	SK6
y =		Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

1.15	2.15	3.15	N30	NO	TÜRÜ
2	3	4	7	1	D
Shelby Numune				1	UD
2	2	2	4	2	D
2	4	5	9	3	D
Shelby Numune				2	UD
4	5	7	12	4	D
2	4	6	10	5	D
2	3	4	7	6	D
3	5	7	12	7	D
8	21	25	46	8	D
16	33	18/5	>50	9	D

Tabaka Sınırları	AÇIKLAMALAR
0,3	Bitkisel Toprak
	4.50 m de Bitki kalıntıları **Orta katı - Katı KİL** Sarımsı gri renkli, fisürlü,aktarılmış, plastik.
11,5	Kayma Düzlemi
	Sert KİL Gri renkli laminalı,orta plastisiteli
13,85	

Sondaj Tipi : Rotary

Sondaj Derinliği : 13,85 m

Mühendis :

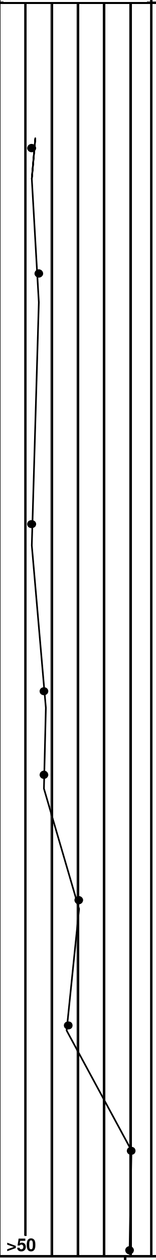
Başlama Tarihi : 10.07.2004

Bitirme Tarihi : 10.07.2004

YASS Derinliği : 2,40 m

Figure C.3: Boring log of SK6

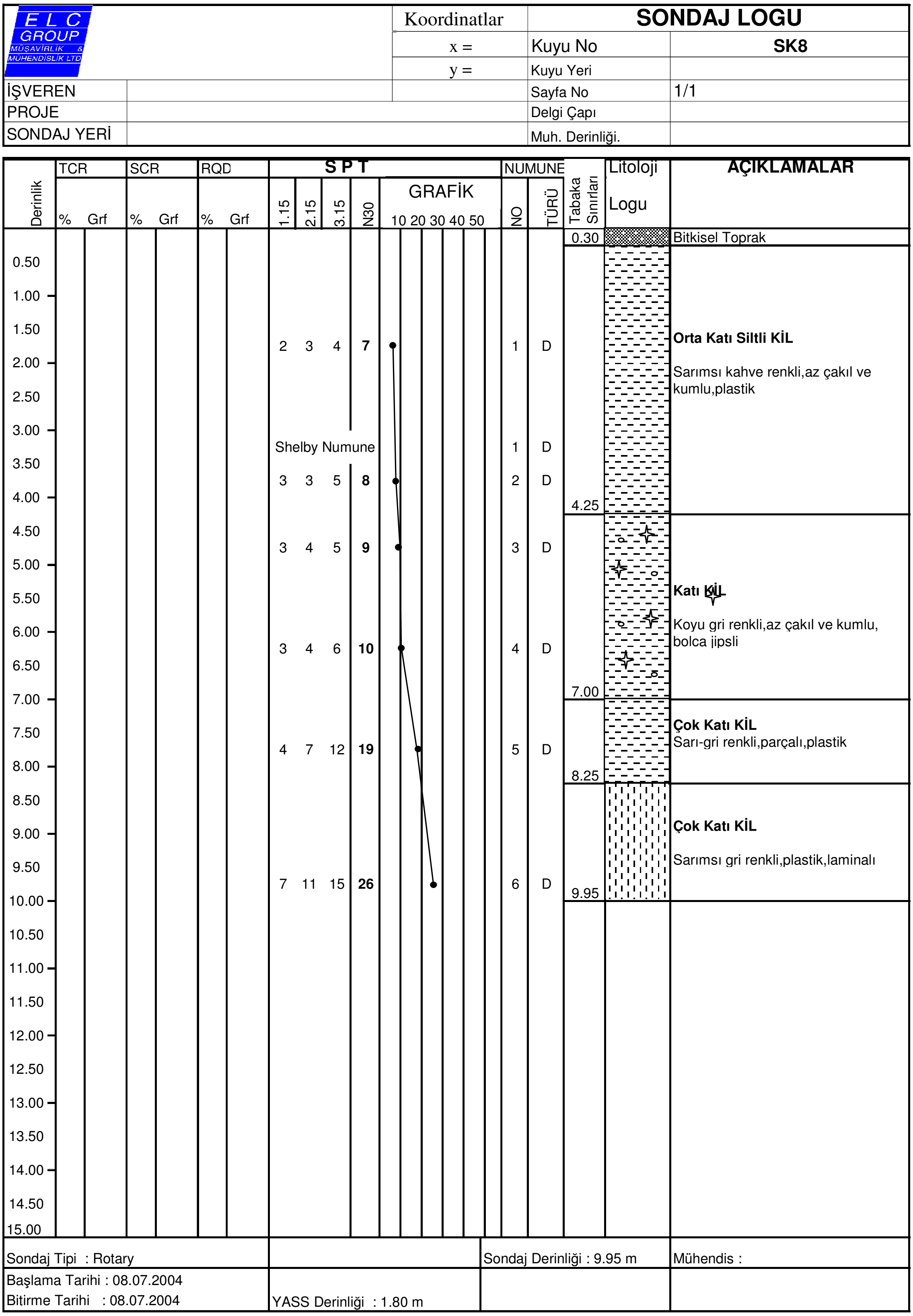
ELC GROUP MÜŞAVİRLİK & MÜHENDİSLİK LTD		Koordinatlar	SONDAJ LOGU	
		x =	Kuyu No	SK7
		y =	Kuyu Yeri	
İŞVEREN			Sayfa No	1/1
PROJE			Delgi Çapı	
SONDAJ YERİ			Muh. Derinliği.	



Derinlik	TCR		SCR		RQD		S P T					NUMUNE		Tabaka Sınırları	Litoloji	AÇIKLAMALAR
	%	Grf	%	Grf	%	Grf	1.15	2.15	3.15	N30	GRAFİK 10 20 30 40 50	NO	TÜRÜ		Logu	
														0,30		Bitkisel Toprak
0.50																
1.00																
1.50																
							3	5	8	**13**		1	D			
2.00																
2.50																**Katı KİL** Sarımsı renli,aktarılmış,plastik
3.00																
							4	5	10	**15**		2	D			
3.50																
4.00																
4.50																
5.00																
5.50																
6.00																
							3	6	6	**12**		3	D			
6.50																
7.00														7.00		
7.50																
							Shelby Numune					1	UD			
8.00																**Çok Katı KİL**
							6	8	10	**18**		4	D			
8.50																Sarımsı gri renkli,parçalı,aktarılmış
9.00																
							4	8	10	**18**		5	D			
9.50																
10.00														10.00		
10.50																
							9	14	16	**30**		6	D			**Çok katı KİL**
11.00																
11.50																Yeşilimsi, sarı- gri renkli laminalı. parçalı
12.00																
							7	12	15	**27**		7	D			
12.50																
13.00																
13.50																
							15	23	31	**>50**		8	D			**Sert KİL**
14.00																Yeşilimsi gri renkli laminalı.
14.50																
15.00							21	26	30/12		**>50**	9	D			

Sondaj Tipi : Rotary		Sondaj Derinliği :15.45 m	Mühendis :
Başlama Tarihi : 07.07.2004 Bitirme Tarihi : 07.07.2004	YASS Derinliği : 4.40 m		

Figure C.4: Boring log of SK7



SONDAJ LOGU

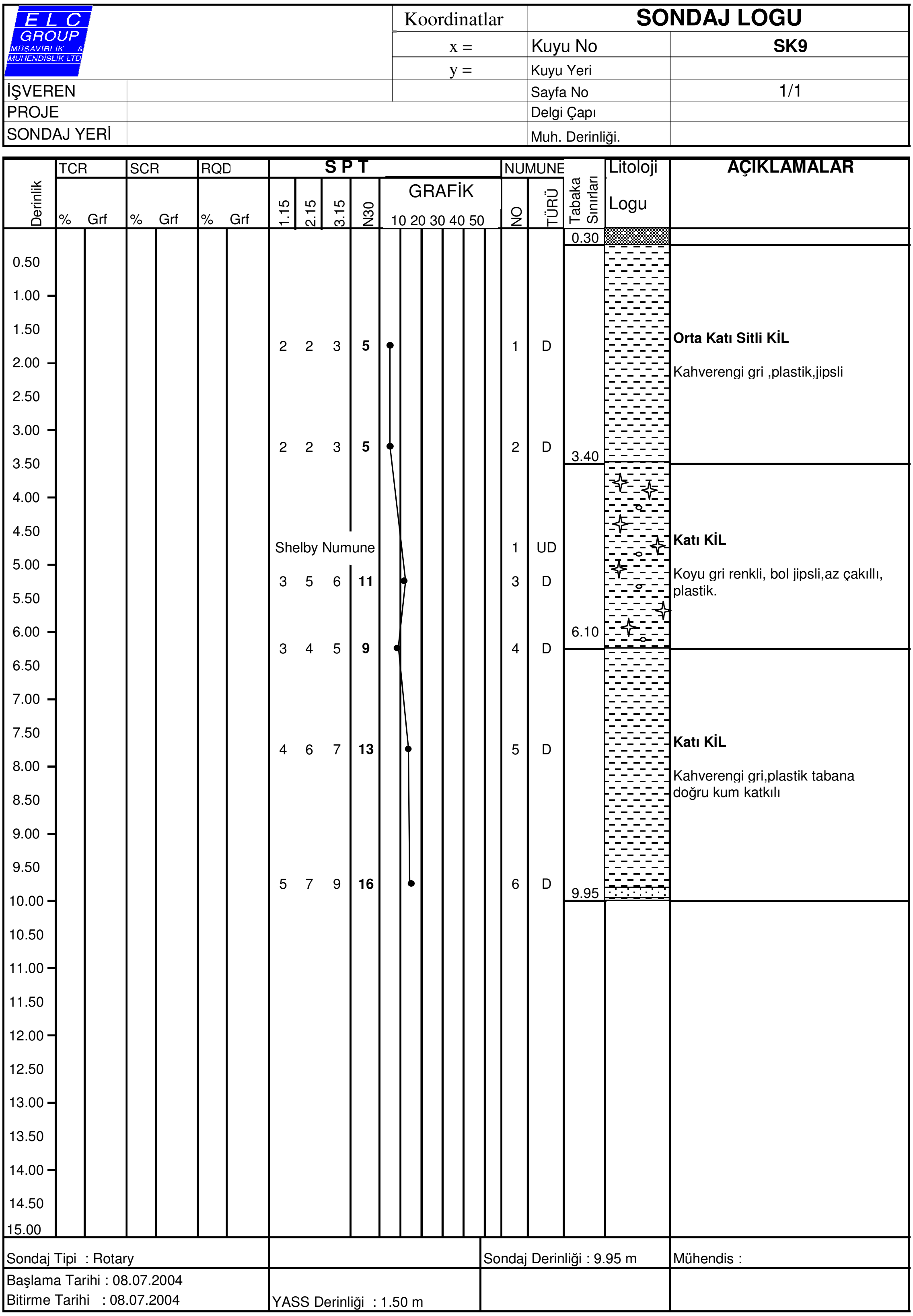
Koordinatlar
x =
y =
Kuyu No SK8
Kuyu Yeri
Sayfa No 1/1
Delgi Çapı
Muh. Derinliği.
İŞVEREN
PROJE
SONDAJ YERİ

TCR %	TCR Grf	SCR %	SCR Grf	RQD %	RQD Grf	SPT 1.15	SPT 2.15	SPT 3.15	SPT N30	NUMUNE NO	NUMUNE TÜRÜ
						2	3	4	**7**	1	D
						Shelby Numune				1	D
						3	3	5	**8**	2	D
						3	4	5	**9**	3	D
						3	4	6	**10**	4	D
						4	7	12	**19**	5	D
						7	11	15	**26**	6	D

Tabaka Sınırları	AÇIKLAMALAR
0.30	Bitkisel Toprak
4.25	**Orta Katı Siltli KİL** Sarımsı kahve renkli,az çakıl ve kumlu,plastik
7.00	**Katı KİL** Koyu gri renkli,az çakıl ve kumlu, bolca jipsli
8.25	**Çok Katı KİL** Sarı-gri renkli,parçalı,plastik
9.95	**Çok Katı KİL** Sarımsı gri renkli,plastik,laminalı

Sondaj Tipi : Rotary
Sondaj Derinliği : 9.95 m
Mühendis :
Başlama Tarihi : 08.07.2004
Bitirme Tarihi : 08.07.2004
YASS Derinliği : 1.80 m

Figure C.5: Boring log of SK8



		Koordinatlar	SONDAJ LOGU
	x =	Kuyu No	SK9
	y =	Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

Derinlik | TCR % Grf | SCR % Grf | RQD % Grf

S P T 1.15	2.15	3.15	N30	NUMUNE NO	TÜRÜ
2	2	3	5	1	D
2	2	3	5	2	D
Shelby Numune				1	UD
3	5	6	11	3	D
3	4	5	9	4	D
4	6	7	13	5	D
5	7	9	16	6	D

Tabaka Sınırları	AÇIKLAMALAR
0.30	
3.40	**Orta Katı Sitli KİL** Kahverengi gri ,plastik,jipsli
6.10	**Katı KİL** Koyu gri renkli, bol jipsli,az çakıllı, plastik.
9.95	**Katı KİL** Kahverengi gri,plastik tabana doğru kum katkılı

Sondaj Tipi : Rotary

Sondaj Derinliği : 9.95 m

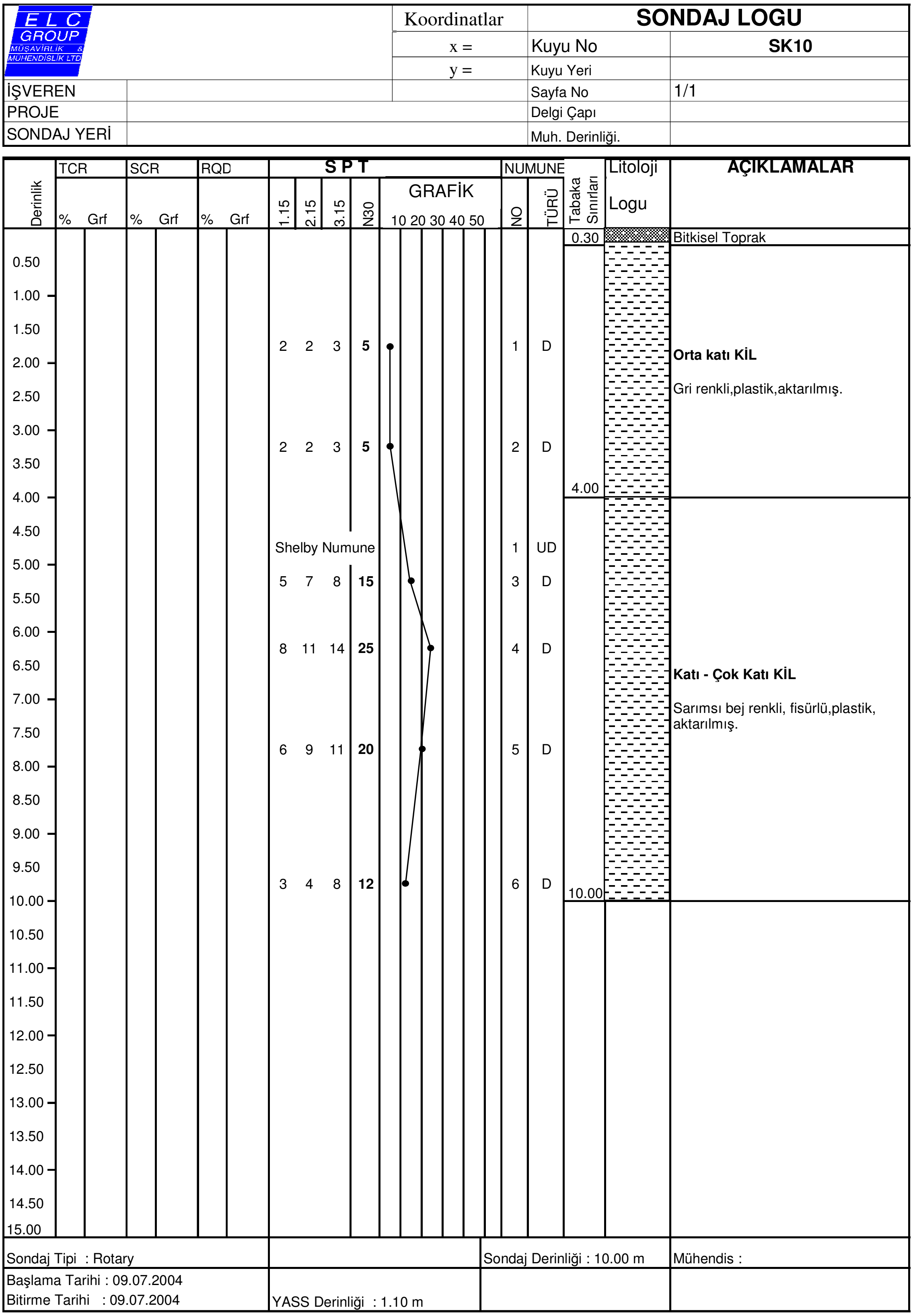
Mühendis :

Başlama Tarihi : 08.07.2004

Bitirme Tarihi : 08.07.2004

YASS Derinliği : 1.50 m

Figure C.6: Boring log of SK9

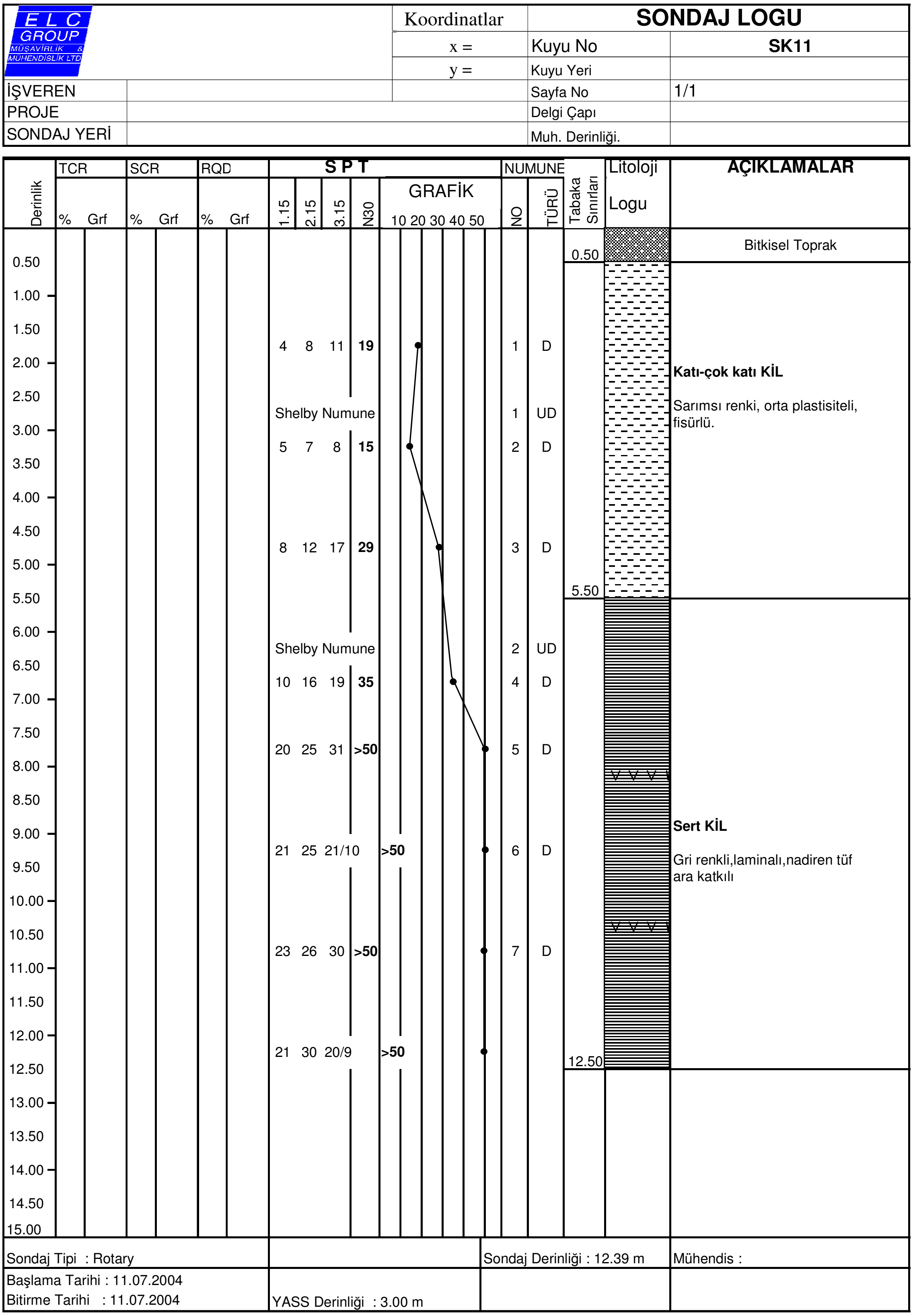


ELC GROUP MÜŞAVİRLİK & MÜHENDİSLİK LTD	Koordinatlar	SONDAJ LOGU	
	x =	Kuyu No	SK10
	y =	Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

S P T 1.15	2.15	3.15	N30	NUMUNE NO	TÜRÜ	Tabaka Sınırları	AÇIKLAMALAR
						0.30	Bitkisel Toprak
2	2	3	**5**	1	D		**Orta katı KİL** Gri renkli,plastik,aktarılmış.
2	2	3	**5**	2	D	4.00	
Shelby Numune				1	UD		
5	7	8	**15**	3	D		
8	11	14	**25**	4	D		**Katı - Çok Katı KİL** Sarımsı bej renkli, fisürlü,plastik, aktarılmış.
6	9	11	**20**	5	D		
3	4	8	**12**	6	D	10.00	

Sondaj Tipi : Rotary
Sondaj Derinliği : 10.00 m
Mühendis :
Başlama Tarihi : 09.07.2004
Bitirme Tarihi : 09.07.2004
YASS Derinliği : 1.10 m

Figure C.7: Boring log of SK10



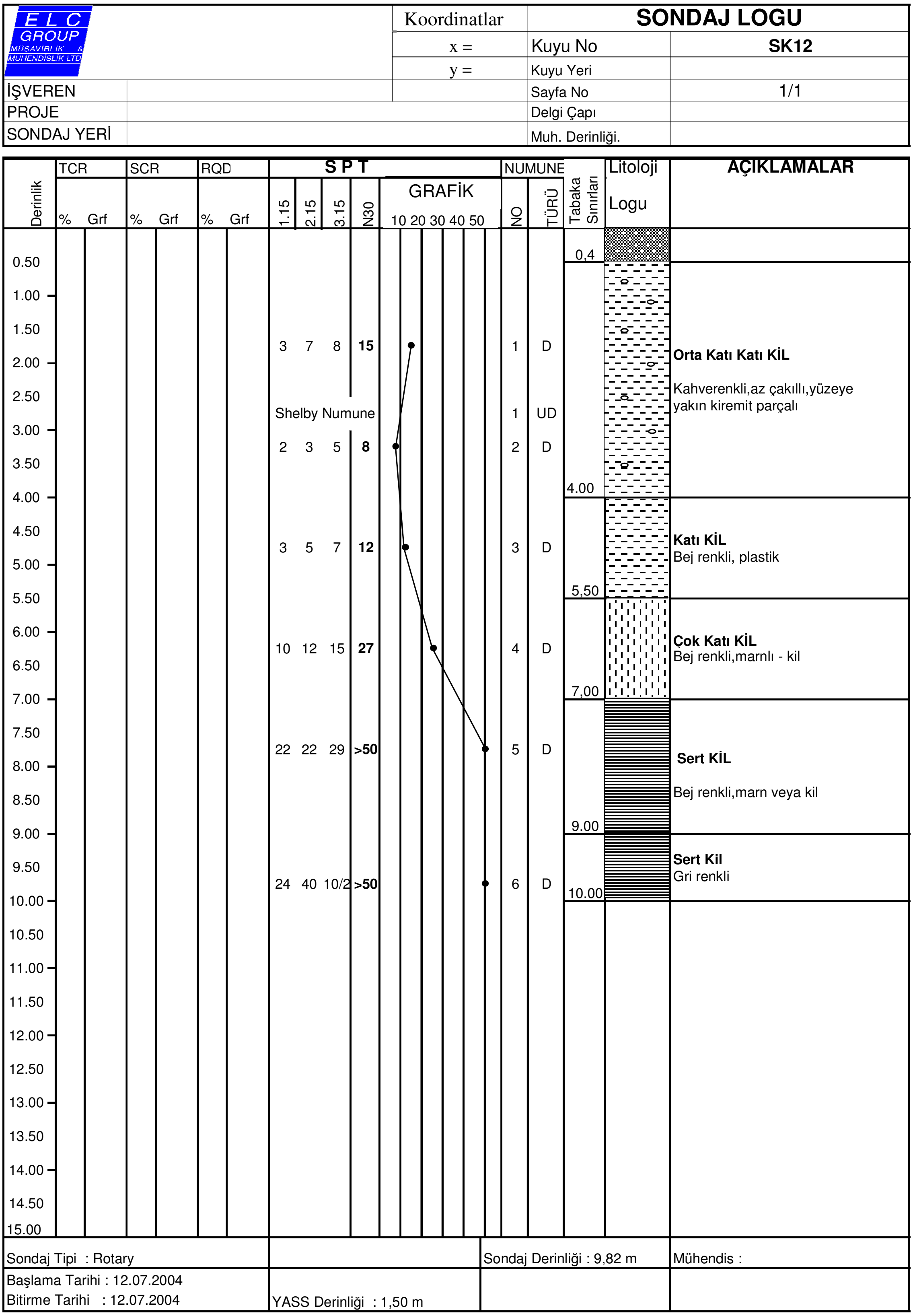
ELC GROUP MÜŞAVİRLİK & MÜHENDİSLİK LTD	Koordinatlar	**SONDAJ LOGU**	
	x =	Kuyu No	**SK11**
	y =	Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

Derinlik	1.15	2.15	3.15	N30	NUMUNE NO	NUMUNE TÜRÜ
1.75	4	8	11	**19**	1	D
2.75	Shelby Numune				1	UD
3.25	5	7	8	**15**	2	D
4.75	8	12	17	**29**	3	D
6.25	Shelby Numune				2	UD
6.75	10	16	19	**35**	4	D
7.75	20	25	31	**>50**	5	D
9.25	21	25	21/10	**>50**	6	D
10.75	23	26	30	**>50**	7	D
12.25	21	30	20/9	**>50**		

Tabaka Sınırları	AÇIKLAMALAR
0.50	Bitkisel Toprak
5.50	**Katı-çok katı KİL** Sarımsı renki, orta plastisiteli, fisürlü.
12.50	**Sert KİL** Gri renkli,laminalı,nadiren tüf ara katkılı

Sondaj Tipi : Rotary		Sondaj Derinliği : 12.39 m	Mühendis :
Başlama Tarihi : 11.07.2004 Bitirme Tarihi : 11.07.2004	YASS Derinliği : 3.00 m		

Figure C.8: Boring log of SK11

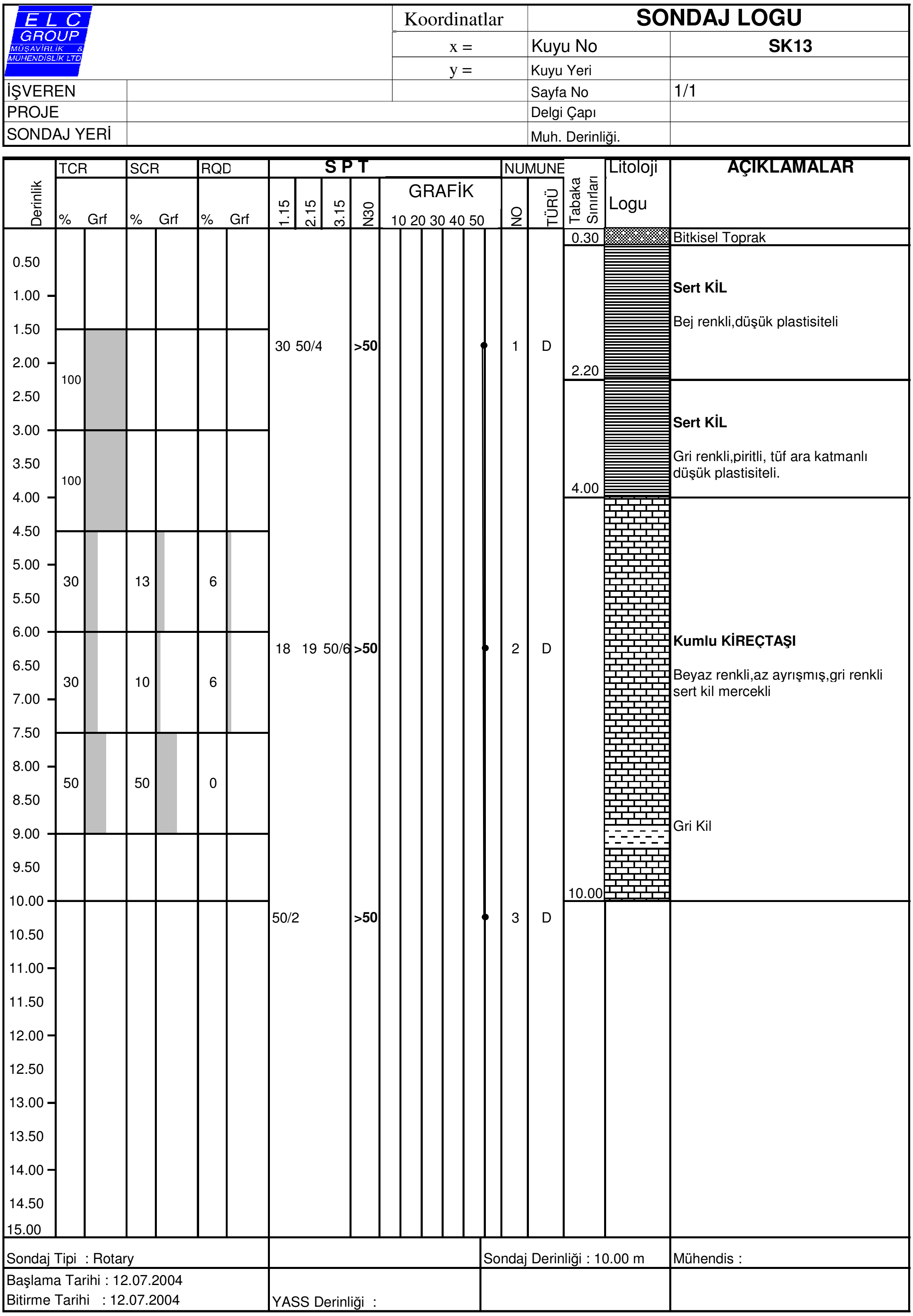


ELC GROUP MÜŞAVİRLİK & MÜHENDİSLİK LTD	Koordinatlar	SONDAJ LOGU	
	x =	Kuyu No	**SK12**
	y =	Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

	TCR		SCR		RQD		S P T					NUMUNE			Litoloji	**AÇIKLAMALAR**
Derinlik	%	Grf	%	Grf	%	Grf	1.15	2.15	3.15	N30	GRAFİK 10 20 30 40 50	NO	TÜRÜ	Tabaka Sınırları	Logu	
														0,4		
0.50																
1.00																
1.50																
							3	7	8	**15**		1	D			**Orta Katı Katı KİL**
2.00																
2.50																Kahverenkli,az çakıllı,yüzeye yakın kiremit parçalı
							Shelby Numune					1	UD			
3.00																
							2	3	5	**8**		2	D			
3.50																
														4.00		
4.00																
4.50																**Katı KİL**
							3	5	7	**12**		3	D			Bej renkli, plastik
5.00																
														5,50		
5.50																
6.00																**Çok Katı KİL**
							10	12	15	**27**		4	D			Bej renkli,marnlı - kil
6.50																
														7,00		
7.00																
7.50																
							22	22	29	**>50**		5	D			**Sert KİL**
8.00																
8.50																Bej renkli,marn veya kil
														9.00		
9.00																
9.50																**Sert Kil** Gri renkli
							24	40	10/2	**>50**		6	D			
														10.00		
10.00																
10.50																
11.00																
11.50																
12.00																
12.50																
13.00																
13.50																
14.00																
14.50																
15.00																

Sondaj Tipi : Rotary		Sondaj Derinliği : 9,82 m	Mühendis :
Başlama Tarihi : 12.07.2004 Bitirme Tarihi : 12.07.2004	YASS Derinliği : 1,50 m		

Figure C.9: Boring log of SK12



Koordinatlar		SONDAJ LOGU	
x =		Kuyu No	SK13
y =		Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

Derinlik	TCR %	SCR %	RQD %	SPT 1.15	SPT 2.15	SPT 3.15	N30	NUMUNE NO	NUMUNE TÜRÜ
1.50	100			30	50/4		>50	1	D
3.00	100								
4.50	30	13	6						
6.00	30	10	6	18	19	50/6	>50	2	D
7.50	50	50	0						
10.00				50/2			>50	3	D

Tabaka Sınırları	AÇIKLAMALAR
0.30	Bitkisel Toprak
2.20	**Sert KİL** Bej renkli,düşük plastisiteli
4.00	**Sert KİL** Gri renkli,piritli, tüf ara katmanlı düşük plastisiteli.
10.00	**Kumlu KİREÇTAŞI** Beyaz renkli,az ayrışmış,gri renkli sert kil mercekli Gri Kil

Sondaj Tipi : Rotary

Başlama Tarihi : 12.07.2004

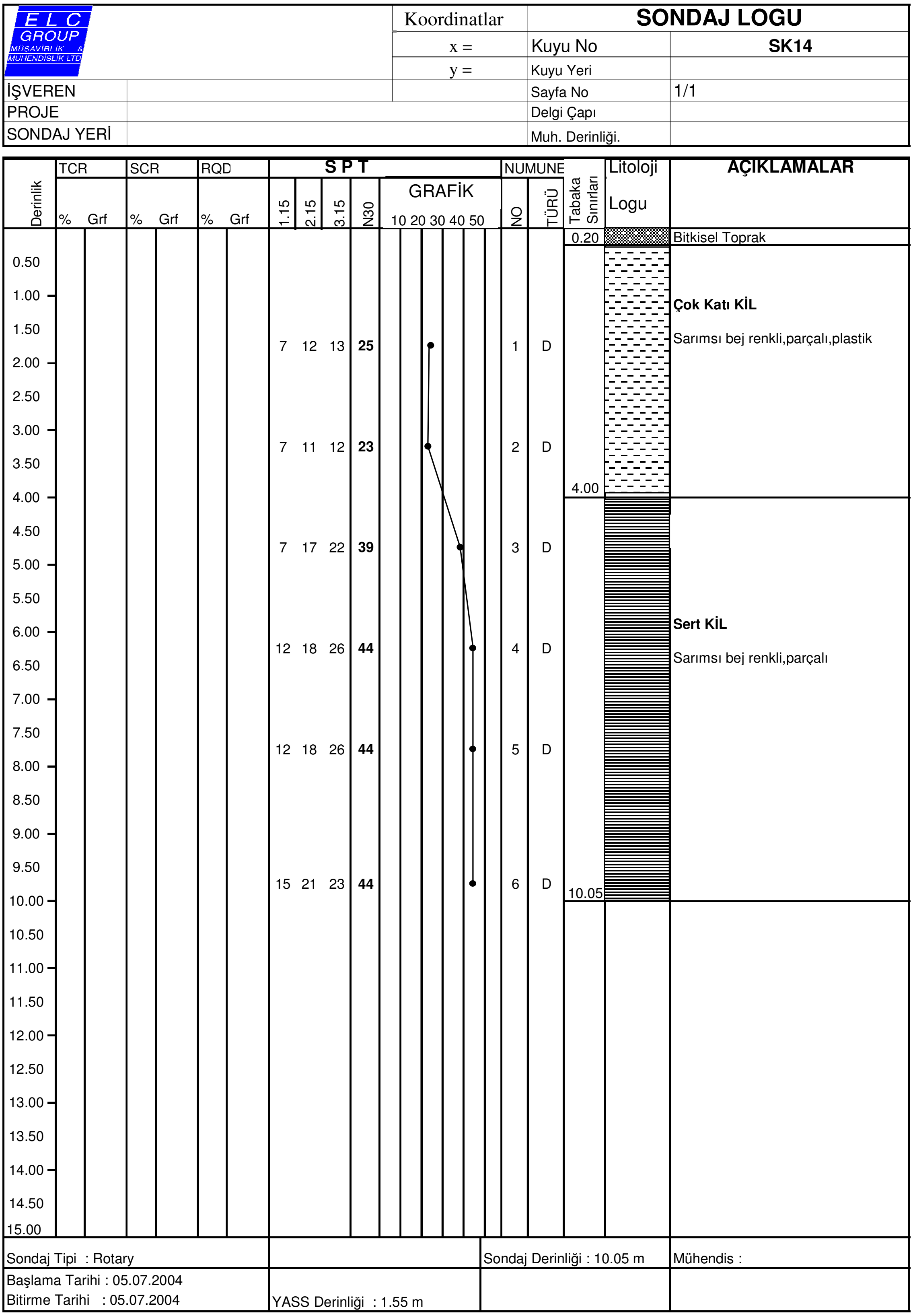
Bitirme Tarihi : 12.07.2004

YASS Derinliği :

Sondaj Derinliği : 10.00 m

Mühendis :

Figure C.10: Boring log of SK13



ELC GROUP
MÜŞAVİRLİK & MÜHENDİSLİK LTD

	Koordinatlar	SONDAJ LOGU	
	x =	Kuyu No	SK14
	y =	Kuyu Yeri	
İŞVEREN		Sayfa No	1/1
PROJE		Delgi Çapı	
SONDAJ YERİ		Muh. Derinliği.	

TCR %	TCR Grf	SCR %	SCR Grf	RQD %	RQD Grf	SPT 1.15	SPT 2.15	SPT 3.15	N30	NUMUNE NO	NUMUNE TÜRÜ
						7	12	13	**25**	1	D
						7	11	12	**23**	2	D
						7	17	22	**39**	3	D
						12	18	26	**44**	4	D
						12	18	26	**44**	5	D
						15	21	23	**44**	6	D

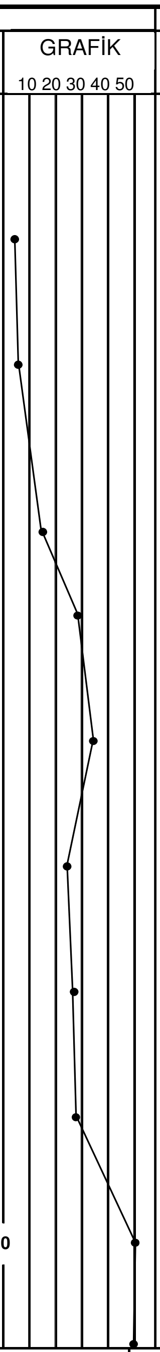
Tabaka Sınırları	AÇIKLAMALAR
0.20	Bitkisel Toprak
4.00	**Çok Katı KİL** Sarımsı bej renkli,parçalı,plastik
10.05	**Sert KİL** Sarımsı bej renkli,parçalı

Sondaj Tipi : Rotary		Sondaj Derinliği : 10.05 m	Mühendis :
Başlama Tarihi : 05.07.2004 Bitirme Tarihi : 05.07.2004	YASS Derinliği : 1.55 m		

Figure C.11: Boring log of SK14

ELC GROUP MÜŞAVİRLİK & MÜHENDİSLİK LTD		Koordinatlar	SONDAJ LOGU	
		x =	Kuyu No	SK16
		y =	Kuyu Yeri	
İŞVEREN			Sayfa No	1/1
PROJE			Delgi Çapı	
SONDAJ YERİ			Muh. Derinliği.	

Derinlik	TCR		SCR		RQD		S P T				NUMUNE		Tabaka Sınırları	Litoloji Logu	AÇIKLAMALAR
	%	Grf	%	Grf	%	Grf	1.15	2.15	3.15	N30	NO	TÜRÜ			
0.50													0.50		Bitkisel Toprak
1.00															
1.50															
							1	2	2	**4**	1	D			**Yumuşak - Orta Katı KİL**
2.00															Kahverengi,yoğun fisürlü,az çakıllı,plastik.
2.50															
3.00															
							2	3	3	**6**	2	D			
3.50															
4.00													4.00		
4.50															
							Shelby Numune				1	UD			
5.00															
							5	7	8	**15**	3	D			
5.50															
6.00															
							6	12	16	**28**	4	D			**Çok Katı KİL**
6.50															
7.00															Sarımsı renkli,parçalı
7.50															7 - 8.50 arasında nispeten parçalı ve sert
							8	14	20	**34**	5	D			
8.00															
8.50															
9.00															
							8	11	14	**25**	6	D			
9.50															
10.00															
10.50															
							7	13	14	**27**	7	D			
11.00													11.00		Bu seviyede kayma izi mevcut
11.50															
12.00															
							12	13	15	**28**	8	D			
12.50															
13.00													13.00		
13.50															
							10	30	20/1	**>50**					**Sert KİL**
14.00															Gri renkli,orta plastisiteli.
14.50															
15.00							45	50		**>50**					



Sondaj Tipi : Rotary		Sondaj Derinliği : 15.30 m	Mühendis :
Başlama Tarihi : 15.07.2004 Bitirme Tarihi : 16.07.2004	YASS Derinliği : 1.20 m		

Figure C.12: Boring log of SK16

APPENDIX D
SOIL SECTIONS

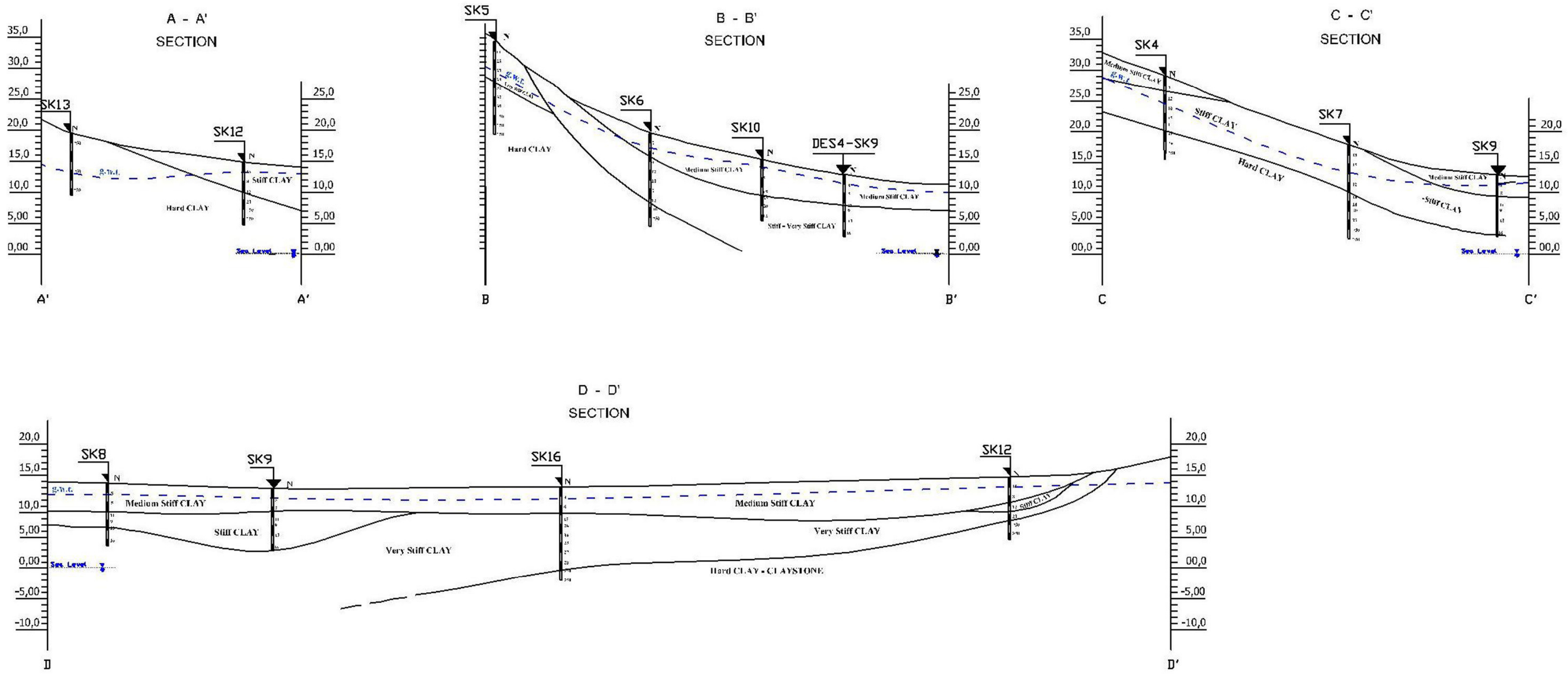


Figure D.1: Soil sections

RESUME

FUAT ÖZBAŞ
BS in Civil Engineering
Civil Engineering Faculty
Istanbul Technical University
Istanbul, Turkey

Personal Details

Date of Birth:	31.10.1979
Place of Birth:	Istanbul

Qualifications / Degrees

2003-	**MS, Soil Mechanics and Geotechnical Engineering, Istanbul Technical University, Civil Engineering Faculty, Istanbul, Turkey**
1999-2003	**BS, Civil Engineering , Istanbul Technical University, Civil Engineering Faculty, Istanbul, Turkey**
1997-1999	**BS, Environmental Engineering, Marmara University, Engineering Faculty, Istanbul, Turkey, (Left after two years)**
1994-1997	**Arı Science High School, Ankara, Turkey**

Certificates

- ISO 9001 Internal Auditor Certificate, NMT Consulting, 16 September 2006.
- ISO 14001 and OHSAS 18001 Certificate, IEC (International European Certification), 17 December 2005.

Courses Attended

- IBM Lotus Notes v6.5.4 Administrator Course, 5-30 September 2005. (Under coverage of this course, brief administration lessons were also attended for Windows Server 2003 and Firewall on Linux).

Skills / Abilities

- **Computer Skills**

  Programs:
  - MS office (Word, Excel, PowerPoint, FrontPage, Outlook)
  - AutoCAD 2006 (Advanced)
  - SAP2000 (Advanced)
  - MS Project 2003 (Pre-intermediate)
  - IBM Lotus Notes v6.5.4 & Administrative Tools (Upper-intermediate)
  - ETABS v7.0 (Intermediate)

  Programming Languages:
  - Fortran(Upper-intermediate)
  - Visual Basic (Pre-intermediate)

- **Language Skills:**
  - **English:** Fluent in both reading, writing and speaking.
  - **Turkish:** Fluent in both reading, writing and speaking.
  - **German:** Pre-intermediate in both reading, writing and speaking

- **Management skills**

  Able to lead and motivate members in a project team. Good at problem solving even under pressure. Enthusiastic in taking responsibilities.

Publications

An article titled "The evaluation of pre-consolidation pressure results of "CH" subgrades in terms of various determination methods", Proceedings of the 16th European Young Geotechnical Engineers Conference, Vienna, Austria, (7 July 2004)

Experiences

- IBM Lotus Notes and CRM Administrator, IT Responsible, Preperation of proposals for US Government and International Companies (mostly US based), ISO 9001 Quality Management Assistant, 77 CONSTRUCTION

CONTRACTING AND TRADING CO., Istanbul, Turkey. (April 2005-Present)

- Various projects in geotechnical engineering project design office and construction sites, GEOBOS LTD ŞTİ, Istanbul, Turkey, (April-September 2004)

Various summer internship experiences through BS education,

- The project of Tarsus substructure construction site, Güriş A.Ş. Tarsus, Turkey, (August 2003)
- The project of KOÜ (Kocaeli Üniv.) Hospital construction site, Güriş A.Ş. İzmit, Turkey, (August 2002)
- Various projects in a structural design office, PROBI A.Ş., Istanbul, Turkey, (June-July 2002)
- The project of Istanbul Subway System Construction Site, Güriş A.Ş., Istanbul, Turkey, (August 2001)
- The project of Olympic Stadium Construction Site, SPOR Yapi A.Ş., Istanbul Turkey, (July 2001)
- A project including forty stared Le'tats Grant Hotel building in a construction site, DEMSAR LTD. ŞTİ, Istanbul, Turkey, (July-August 2000)

Hobbies / Interests

Sports

- I have played basketball in DSI team (1992-1994),
- I have played basketball in Engineering Faculty team of Marmara University (1998-1999),
- I have played american football in American Football Team of Istanbul Technical University (2001-2002).

Music

I like listening to blues and rock music. I have been playing guitar for 7 years.

Traveling

I deeply like traveling. I have been to various places in Europe, like Germany (twice), Austria (twice), Belgium (twice), The Netherlands (twice), Greece, Yugoslavia, Cyprus, Croatia and Bulgaria.