INTERNATIONAL INSTITUTE FOR AEROSPACE SURVEY AND EARTH SCIENCES

ITC

Division of Engineering Geology
Master of Science 1999/2000

CORRELATION OF STATIC CONE PENETRATION TEST RESULTS AND DYNAMIC PROBING TEST RESULTS

Research Study for the data of South Limburg- The Netherlands

(Volume -I)

By

Aroosa Hashmat

(Pakistan)

A thesis submitted in partial fulfilment of the requirements for the degree of
Master of Science in Engineering Geology

June, 2000

INTERNATIONAL INSTITUTE FOR AEROSPACE SURVEY AND EARTH SCIENCES
(ITC)

Kanaalweg 3, 2628 EB Delft, The Netherlands

Abstract

A number of different Static & Dynamic penetration tests are used today in site investigation. In Netherlands, SCPT is very common for site investigation whereas in Germany Dynamic Probing is popular.

In this research study, correlation of SCPT and Dynamic Probing test results has been done. Beside correlation, soil variability has been found statistically. On the basis of coefficient of variability, tentative soil classification has been done. It has also been tried to find the variability of soil with depth. The energy of SCPT and Dynamic Probing tests has been compared on the basis of test results. Also the energy calculated on the basis of test results has been compared with the theoretically required energy to move the cone to certain specified depth using pile formulae.

So far very few correlations have been made between SCPT and Dynamic Probing test results as compared to the correlations between SCPT and SPT(Standard penetration test). The reason may be the popularity of one type of equipment in one specific country compared to other parts of the world.

Probably the correlations made in this research for South Limburg soils can also be used in Germany and Belgium since South Limburg is the part of the Netherlands that is located between Germany and Belgium. The same type of Soil extends in a large part of Germany. Specially in those cases where only Dynamic Probing results are available, the reliable correlations made between the two tests for South Limburg soils will be very useful for the engineers of different countries like Belgium, Germany and Netherlands to get the SCPT values.

Acknowledgements

In this research study, a lot of people have been encouraging and supporting me. I would like to express my thanks to the following people.

I am very much indebted to my supervisors Ir. Wolter Zigterman from ITC and P. Michiel Maurenbrecher M. Sc. from TU, Delft for their continuous and encouraging guidance throughout the research. I am specially thankful to Mr. Wolter Zigterman for his enthusiastic guidance throughout the research and Mr. Maurenbrecher for helping in collection of data and all support to get the information needed and his valuable suggestions.

I very much appreciate the co-operation of Fugro, specially Ing. Ben Telkamp head of the office in Maastricht, for providing the data for this research study. I appreciate and pay special thanks that he arranged site visit for us and we could see the equipment in operation.

I would like to express my thanks to all ITC staff members, who have been supportive to me directly or indirectly during my study in ITC.

I express my special thanks to Dr. H.R.G.K. Hack, Director of studies ENG:2 for his kind advises and supportive guidance for research and throughout my study in ITC.

I would also take the opportunity to pay thanks to my mother, family and friends for their moral support throughout my stay in the Netherlands.

Aroosa Hashmat
Delft, The Netherlands
June, 2000

Table of contents

List of Symbols and Abbreviations

SCPT	Static Cone Penetration Test
DPT	Dynamic Probing Test
DPL	Light Dynamic probing
DPM	Medium Dynamic probing
DPH	Heavy Dynamic probing
DPSH	Super Heavy Dynamic probing
DCPT	Dynamic Cone Penetration Test
CPTU	Piezocone Test
WST	Weight Sounding Test
SPT	Standard Penetration Test
ISSMFE	International Society of Soil Mechanics and Foundation Engineering
IRTP	International Reference Test Procedure
SL	Notation used for Light Dynamic test by Fugro
SZ	Notation used for Heavy Dynamic test by Fugro
D & DKM	Notation used for SCPT
C.O.V	Coefficient of variation
I-E(SCPT)	Input Energy of SCPT
I-E(DPL)	Input Energy of DPL
I-E(DPH)	Input Energy of DPH
E-Static	Energy based on pile formula
E-Dynamic	Energy based on Dynamic pile formula
σ_V	Vertical Stress
σ_V'	Effective vertical stress
q_c	Cone resistance(The total force acting on the cone, Q_c, divided by the projected area of the cone, Ac. $q_c=Q_c/A_c$)
q_t	Corrected cone resistance(The cone resistance q_c corrected for pore water pressure effects)
q_d	Dynamic point resistance
N_{20}	Number of blows for 0.2m penetration
N_{10}	Number of blows for 0.1m penetration

1. Introduction and scope of the research

1.1 Introduction

The penetrometer evolved from the need of acquiring data on sub-surface soils that were not obtainable by any other means. Penetration testing is among the vast number of in situ tests available for soil exploration. In situ tests can often be preferable to laboratory tests because of important advantages such as

- cost - time effectiveness
- the ability to assess the soil in its natural environment

A penetrometer consists of a slender metal rod which is pushed or driven into the ground by jacks or with hammer blows. Either method is such that the resistance to penetration of the metal rods is measured at any depth. The data are plotted on a diagram representing resistance to penetration on the abscissa and depth of penetration on the ordinate (G. Sanglerat , 1972).

There are two methods of advancing the rods , namely the static method and the dynamic method , and two basic types of penetrometers:

- The static penetrometer
- The dynamic penetrometer

Penetrometers are generally used in Europe during the exploratory phase of a soil investigation to determine the soil conditions in general, such as the thickness and lateral extent of various strata, so that an evaluation of different possible foundation methods can be made .

1.2 Objective of the research

In this research study, comparison of Dynamic Probing test results has been made with Static Cone Penetration test results. The data from South Limburg area has been used for this purpose. In the South Limburg area Loess soils are present which are also found over a large area in the western part of Germany.

In Netherlands, Static Cone Penetration test is very popular due to its advantages over the traditional combination of borings, sampling and other testing whereas in Germany, Dynamic Probing test is widely used as an investigative tool for site investigation. In this research study, the correlation between the two tests results for soils at the international border has been made.

Besides correlation, attempt has also been made to use the data for tentative soil classification in the South Limburg area. The available data has also been utilised to check the variability of soil with depth. The results of Heavy Dynamic Probing and Light Dynamic Probing have been compared in terms of energy imparted by both equipments to the ground. Also energy of both equipments has been compared with the theoretical energy based on pile formulae.

It would be very useful if the Static Cone Penetration results of one country are available and for the same type of soil Dynamic Probing results can be achieved in an other country by using the correlative work of this study.

The reason for comparison is the comparable size of the cone used in both tests. Also the similarity of the results achieved from both tests along the vertical line makes it interesting to compare the two tests.

Both tests provide continuous record of readings which can be considered more advantageous as compared to Static Cone Penetration Test(SCPT) and Standard Penetration Test (SPT) comparison, where for one reading of SPT, many readings of SCPT are averaged for comparison.

1.3 Location map of the area

Location map of the study area has been attached in Appendix-F. The map indicates various cities of South Limburg area of the Netherlands, in which tests have been carried out.

1.4 Brief description of the study area and available data

The data provided by Fugro B.V the Netherlands, belongs to various projects carried out in different cities of South Limburg area of Netherlands. The data of the following cities has been used in this research study:

Maastricht, Gulpen, Beek, Nuth, Sittard, Heerlen, Landgraaf, Geleen, Kerkrade, Hoensbroek, Sittard, Geleen, Bunde. The details of the projects used in this research study have been given in Appendix-F (table F1). The overview of the data has been given in table 1.1. Besides SCPT tests, Dynamic Probing tests have been carried out according to both Heavy and Light versions of the equipment. The use of Light Dynamic Probing equipment might be due to the poor accessibility of Heavy Dynamic Probing equipment to the test site.

Table 1.1: Overview of the data used in this research study

No	Fugro project No	Location (city)	No of tests/Data sets		
			DPL	DPH	SCPT
1	P-2522	Maastricht	-	11	18
2	P-2308	Gulpen	-	14	7
3	P-2071	Maastricht	-	1	12
4	P-2411	Maastricht	-	1	4
5	P-2191	Beek	-	3	7
6	P-2614	Maastricht	-	3	3
7	P-2390	Landgraaf	2	-	5
8	P-2413	Kerkrade	3	-	5
9	P-2549	Nuth	1	-	3
10	P-2523	Heerlen	3	-	3
11	P-1491	Heerlen	2	-	3
12	P-2336	Landgraaf	2	-	2
13	P-2467	Sittard	1	-	2
14	P-2651	Geleen	1	-	1
15	P-2422	Bunde	1	-	1
16	P-2404	Hoensbroek	3	-	1

DPL Light Dynamic Probing

DPH Heavy Dynamic Probing

SCPT Static Cone Penetration Test

2. Literature review

2.1 General

Penetration testing is one of the insitu testing methods. Main applications of the penetration test in site investigation process are as follows:

- to determine sub-surface stratigraphy and identify materials present
- to estimate geotechnical parameters
- to provide results for direct geotechnical design

A number of different static and dynamic penetration tests are used today. The most common are:

- Static cone penetration test(SCPT)
- Dynamic Probing Test(DPT)
- Standard Penetration Test(SPT)
- Weight Sounding Test(WST)

2.2 Historical background of Static Cone Penetration Testing (SCPT)

A hand-operated cone penetrometer was built by Goudsche Machinefabrick of Gouda Holland, in co-operation with the Delft Laboratory of Soil Mechanics in 1936. The $10cm^2$ cone with 60 degree apex angle was pushed down by hand by one to two men. The cone was pushed down by an inner rod through tubes with an outer diameter of 36mm. The penetration resistance was read every 0.10 or 0.20m on a manometer. The maximum depth was two to three meters.

This penetrometer was used e.g. in China in the 1930's by the Whangpoo conservatory Board (WCR) to determine the bearing capacity of piles as mentioned by the Engineering Society of China (1937).

The original Dutch cone penetrometer was not provided with a conical sleeve just above the cone. This was a later improvement by the Delft Soil Mechanics Laboratory. The capacity of early cone penetrometers was very limited since they were operated by hand and the mass acting as a dead weight was formed by soil on a reaction floor. The maximum capacity in 1948 was 10 tonnes (J.De Ruiter, 1988).

In 1953, Begemann had proven from extensive tests that the total friction on the tubes was not a reliable parameter. At a certain depth, the first tube passing had a higher friction than the second, third, etc, meter of tubing (personal communications, W.Zigterman 2000).

He proposed that the local skin friction resistance should be measured every 0.2m with a separate friction sleeve located just above the cone (Begemann, 1953).

It turned out to be a significant improvement of the Dutch static cone penetration test by adding an "adhesion jacket" behind the cone. Using this new device the local skin friction could be measured in addition to the cone resistance.

Begemann was the first to propose that the ratio of the measured friction resistance along the sleeve and the cone resistance, the so-called friction ratio could be used to classify the different soil layers.

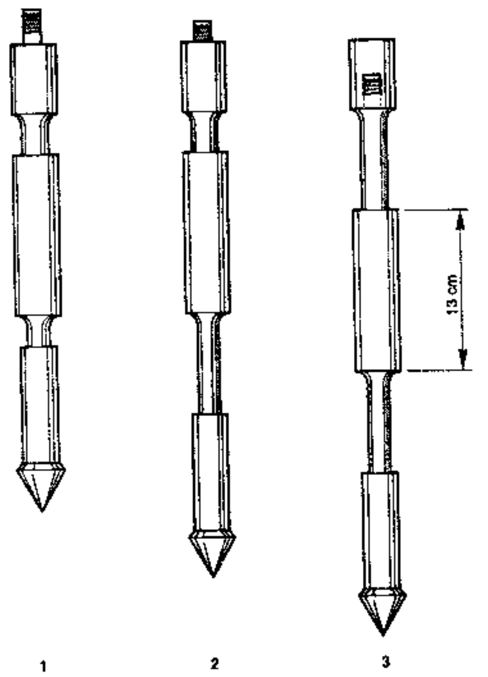


Figure 2.1: Begemann type cone with friction sleeve(Form Sanglerat, 1972)

1: Starting position

2: Cone point pushed down

3: Cone point and friction sleeve pushed down

2.3 Recent developments of Static Cone Penetration Test(SCPT)

Today machines upto 20 tonnes capacity (200kN) are common. Sufficient reaction is provided during a test by a ballasted truck. Nearly all tests are carried out with equipment that applies hydraulic oil pressure to push tubes and rods downwards.

Recent development is the truck provided with tracks to move easily on soft soils as indicated in the figure 2.2.

Existing SCPT systems can be divided into three main groups: mechanical cone penetrometers, electric cone penetrometers and piezocone penetrometers.

Figure 2.2: Truck provided with tracks(alongwith the wheels) to move easily on soft soils

2.3.1 Mechanical cone penetrometers

The standardised mechanical cone is fitted with a conical sleeve immediately above the cone.This sleeve prevents penetration of soil between the tubes and the inner rods.

As outlined by Broms and Flodin (1988) and Sanglerat (1972), several other mechanical cone penetrometers with different features were developed in other countries such as Belgium, Sweden, Germany, France and so on. Most mechanical cone penetrometers measure the force needed to press down the inner rod via a pressure cell, which can be a conventional hydraulic gauge or an electric load cell.

Usually the readings of the hydraulic pressure cells are written down every 0.20m, discontinuous testing. To get a reading the pressure of the machine is intermittent acting on the tubes and the inner rods.

The electric load cells are applied on the top of inner rods for a continuous graph of the cone resistance. For this type of measurement the tubes and inner rods are simultaneously pushed downwards.

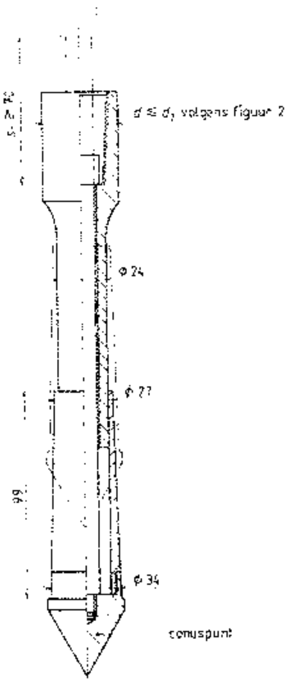


Figure 2.3: Mechanical cone penetrometer(From NEN 3680, 1982)

The Begemann friction cone can only be used for the discontinuous testing method. Every 0.20m of depth, shortly after each other two readings are taken:

1. force acting on the cone
2. force acting on the cone plus friction sleeve.

Mechanical cone penetrometers are still widely used because of their low cost, simplicity and robustness. In rather homogeneous competent soils, without sharp variations in cone resistance, mechanical cone data can be adequate, provided the equipment is properly maintained and the operator has the required experience.

Nevertheless, the quality of the data remains somewhat operator dependent. In soft soils, the accuracy of the results can sometimes be inadequate for a quantitative analysis of the soil properties. In highly stratified materials even a satisfactory qualitative interpretation may be impossible, in particular if the discontinuous method is used (Lunne et al. 1997).

2.3.2 Electric cone penetrometers

Several electrical cone penetrometers where the cone resistance is measured separately using strain gauges or vibratory wire gauges, have been developed since 1950. Delft Soil Mechanics Laboratory has worked with electric cone penetrometers since 1949 and in 1957 produced the first electrical cone penetrometer where the local side friction could be measured separately (Vlasblom, 1985). Electrical cone penetrometers were introduced on a large scale in the Netherlands by Fugro. This company used it as the standard type for SCPT's.

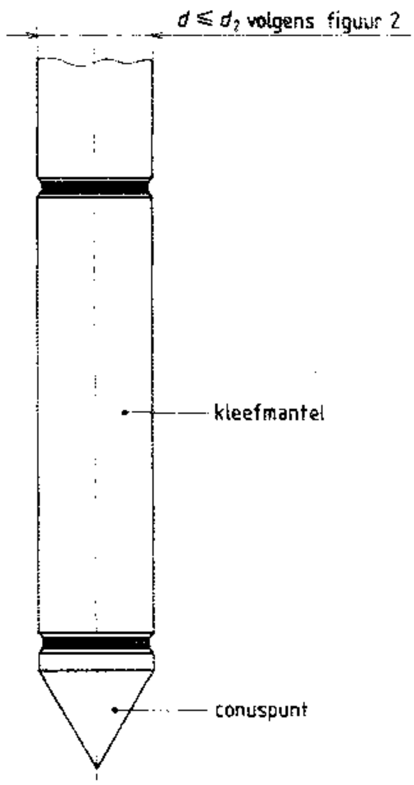


Figure 2.4: Cylindrical electrical friction sleeve cone penetrometer(From NEN 3680, 1982)

The main advantage with an electrical cone penetrometer compared to the traditional discontinuous mechanical test is that a continuous recording of the penetration resistance is obtained as a function of the depth. An electrical cone is more sensitive than a mechanical penetrometer so that it can be used also in loose sand and in soft clay and silt (J.De Ruiter, 1988).

The most commonly used electrical penetrometer is that developed by Fugro in 1965 in the Netherlands. The point and the sleeve resistance of this penetrometer are determined separately with the load cells placed just above the tip. The Fugro cone penetrometer can also be used in very soft clay due to the high sensitivity of the measuring system.

In practice the high price of an electrical cone can be the reason not to apply it in areas with many cobbles or boulders, or where weathered rock occurs at shallow depth. In these circumstances the risk of a damaged (lost) cone is too high.

Using the available wires in the cable running from the cone to the surface, several additions were introduced, e.g. the inclination measurement. This is based on the experience that often SCPT tests deviated from the vertical line. The measuring of water pressures with the piezo cone is the most important addition to the original electrical cone.

2.3.3 Piezo- cone Penetrometer

With the piezo-cone, the pore-water pressure present in the soil adjacent to the cone is measured continuously during penetration. The measured pore pressure consists of the algebraic sum of the pore pressure before penetration and the positive or negative pore pressure, which is caused by tendency for soil compression or dilation due to the penetrating cone.

The position of the filter for measurement of pore pressure is not standardised but ISSMFE International Reference Test Procedure suggests behind the cone (u_2) as the preferred location. Other locations are on the cone (u_1) or behind the friction sleeve (u_3), see figure 2.5.

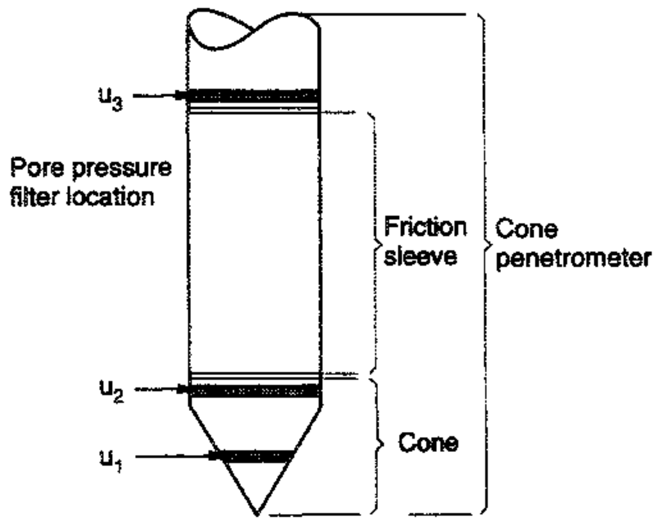


Figure 2.5: Terminology for cone penetrometers indicating pore pressure filter location (From Lunne et al, 1997)

2.4 Role of Static Cone Penetration Test (SCPT) in site investigation

The objective of any subsurface exploration programme is to determine the following:

- nature and sequence of the subsurface strata (geological regime).
- groundwater conditions (hydrogeological regime).
- physical and mechanical properties of the subsurface strata.

The above requirements are a function of the proposed project and the associated risks. There are many techniques available to meet the objectives of a site investigation and these include both field and laboratory testing. Field tests include drilling, sampling, in situ testing, full scale testing and geophysical tests (Lunne, et al. 1997).

Static cone penetration testing is one of the in situ testing methods and has following main applications in the site investigation process:

- to determine sub-surface stratigraphy and identify materials present.
- to estimate geotechnical parameters , and
- to provide results for direct geotechnical design.
- to establish the depth to firm layers.
- driveability and bearing capacity of piles.

2.5 Principle of the Static Cone Penetration Test (SCPT)

In cone penetration tests, a cylindrical cone with a cross section of 10 cm^2 and an apex angle of 60° (accepted as the reference and has been specified in the International Reference Test Procedure ISSMFE 1989) is pushed vertically into the ground at a constant rate of penetration of 20 mm/sec. During penetration, measurements are made of the cone resistance, the side friction against the cylindrical shaft just above the tip and, in piezocone tests, the pore water pressure generated at penetration by the cone (Lunne et al . 1997).

2.5.1 Standardisation

The ISSMFE has established the reference test procedure for Static Cone Penetration Test. The reference test equipment consists of a 60° cone, with 10 cm^2 base area and a 150 cm^2 friction sleeve located above the cone.

The position of the filter for measurement of pore pressure (in piezocone) is not standardised, but the International Reference Test Procedure suggests behind the cone (u_2) as the preferred location. Other locations are on the cone (u_1) or behind the friction sleeve (u_3) as already indicated in figure 2.5.

The cone resistance (q_c) is obtained by dividing the ultimate axial force acting on the cone, by the area of the base of the cone. The local unit side friction resistance (f_s) is obtained by dividing the ultimate frictional force acting on the sleeve, by its surface area.

The rate of penetration has been accepted as 20 mm/sec with a tolerance of ±5mm/sec. A continuous reading is recommended. In no case shall the interval between the readings be more than 0.2 m.

2.6 Advantages of Static Cone Penetration Test (SCPT)

The SCPT has three main advantages over the traditional combination of borings, sampling and other testing. It provides:

- continuous or near continuous data.
- repeatable and reliable penetration data.
- cost savings.

This test is very fast, several tests per day can be carried out, e.g. on one site approximately 12 tests to 12-15 m depth. This high production is possible as for the SCPT tests ballasted trucks are used, weighing 150 to 200 kN.

The SCPT provides important data in cohesionless soils, and empirical correlations are widely used to obtain estimates of effective angle of shearing resistance (Ø). The cone penetrometer has also been used to estimate for clays the undrained shear strength (Thomas, 1968 ; Jamiolkowski et al ; 1982 Aas et al 1986). In soft soils, cone penetration with electrical cones from ground level to depths in excess of 100 meters may be achieved provided verticality is maintained (Lunne et al. 1997). There is a possibility to reduce the friction on the outside of the tubes by injection of water-bentonite or a similar lubrication fluid.

2.7 Limitations of Static Cone Penetration Test (SCPT)

The SCPT has the following limitations :

Gravel layers and boulders, heavily cemented zones and dense sand layers can restrict the penetration severely and deflect and damage cones and rods especially if the overlying soils are very soft and allow rod buckling.

One main limitation of the cone penetration test is that no samples are recovered during the testing. Therefore, SCPT must normally be supplemented by borings, so that the different strata can be classified and for detailed laboratory investigations of the recovered samples (Broms and Flodin 1988). In these cases the results of the SCPT are used to select the borehole locations that may give the best information.

Top soils with coarse material like stones or debris should always be pre-bored. In some cases it may be necessary to use a casing. The pre-drilling may, in certain cases, be replaced by first pre-forming a hole through the upper problem material with a solid steel dummy probe with a diameter slightly larger than the cone penetrometer.

As the SCPT is static, a rather large reaction mass is required. Nowadays, mostly a ballasted truck of 150 kN to 200 kN is used. If the reaction mass needs to be mobilised by ground anchors, the test becomes very labour intensive and will take much more time.

2.8 Historical background of Dynamic Probing

Between the two world wars, Dynamic Probing became known besides the traditional boring methods as a means of subsoil exploration in the field of foundation engineering, especially in Europe. After 1945, the known use of Dynamic Probing equipment became widespread within and beyond Europe.

Considering the historic development of Dynamic Probing, it is not surprising that the present equipment parameters vary considerably, at least in part. The equipment parameters are rather consistent for DPL(Light Dynamic Probing) and DPH(Heavy Dynamic Probing), one of the possible reasons being that most of this equipment is being designed closely to the requirements of the German standard DIN 4094, at least for the DPL.

It has been proven that Dynamic Probing using properly designed equipment and an adequate procedure, allows measurements to be made that are as reliable as those performed with static equipment (J.De. Ruiter,1988).

2.9 Types of Dynamic Probing

Dynamic Probing has been divided into the following main types:

- DPL (Light Dynamic Probing) representing the lower end of mass range of dynamic penetrometers used worldwide, is used in quality control during construction supervision besides the regular site investigations.

- DPM (Medium Dynamic Probing) representing the medium mass range, is widely used and fills the gap of dimensions between DPL and DPSH.

- DPH (Heavy Dynamic Probing) representing the medium to very heavy mass range is widely used and also fills the gap of dimensions between the two extremes DPL and DPSH.

- DPSH (Super Heavy Dynamic Probing) representing the upper end of the mass range of dynamic penetrometers, also shows wide spread use.

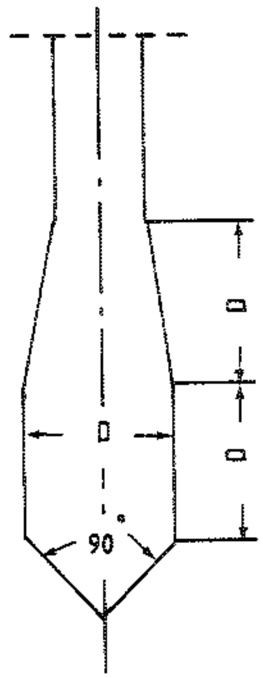


Figure 2.6: Dynamic probe as specified in IRTP(ISSMFE) (From J de Ruiter, penetration testing, 1988)

The graphical presentation of the results of dynamic presentation testing usually plots the number of blows N_{10} or N_{20} on the abscissa and the probe depth on the ordinate.

Normally probing is conducted vertically, the probing equipment being firmly supported. In some instances, pre-boring is used.

Avoiding skin friction is one of the major concerns in Dynamic Probing, because the cone resistance is the only result that allows interpretation of the test results. Four measures are being taken that are to help reduce or avoid skin friction:

- the cone diameter is larger than the rod diameter
- the rods are being turned at certain depths of penetration
- drilling mud is injected
- push rods similar to the SCPT, separate skin friction from cone resistance

2.10 Role of Dynamic Probing in site investigation

Dynamic penetrometers were originally designed in order to obtain qualitative data on the resistance to penetration of the soil and in particular to determine the compactness of cohesionless soils which are usually difficult to sample.

Dynamic penetration tests are often used to check the uniformity of the soil conditions at a particular site to estimate the location and the thickness of the different strata and to determine the depth to bedrock.

The selection of the Dynamic Probing equipment to be used for a given job normally depends on the local conditions and the purpose of the particular test. Key borings are being made next to some Dynamic Probing locations in order to enable the analysis of soil types, layer stratification with depth.

Dynamic penetrometers are mainly used in Europe during the preliminary exploration phase, to determine the thickness and the location of the different strata and during the detailed investigation phase, to estimate the shear strength and the compressibility of the various strata.

Dynamic penetrometers are also used to check the compaction of fills or the loosening of the soil at the bottom of deep excavations (J.De. Ruiter, 1988).

2.11 Principle of Dynamic Probing

A hammer of specified mass (M) and a height of fall H is used to drive a pointed probe (cone). The hammer strikes an anvil which is rigidly attached to extension rods. The penetration resistance is defined as the number of blows required to drive the penetrometer a defined distance. According to ISSMFE International Reference Test Procedure, the number of blows should be recorded every 0.1m for DPL, DPM and DPH (N_{10}) and every 0.2 m for DPSH (N_{20}).

Using cones larger in diameter than the rods, may be considered as being mandatory practically worldwide. Test equipment with cone/rod diameter ratios exceeding about 1.3 leads to results in cohesionless and in many cohesive soils that are little or not at all seriously influenced by skin friction.

Turning of the rods is one of the measures that is supposed to reduce skin friction. Skin friction is avoided practically completely if drilling mud is used during the performance of the test. This method is most effective if the drilling mud is injected through holes in the hollow rods near the cone that are directed horizontally or slightly upwards. The use of push rods (casings) similar to the method used in the SCPT is as effective as, but less used than the drilling mud method.

It is being recognised that the driving rate influences the test results. Driving rates of 15 to 30 blows per minute are commonly used. In pervious soils such as sands and gravels, the influence of the driving rate is lower; thus higher rates, e.g. 60 blows per minute, are used.

The application of driving energy is being realised as a very critical factor influencing the test results. Besides ensuring that the driving rods are being kept straight and the rod couplings are tight, the following two facts are being considered as most important:

- guarantee of free fall of the hammer
- guarantee of constant height of fall of the hammer

These facts become even more important if one considers that dynamic penetrometers are either hand or machine operated (J.De.Ruiter, 1988) .

2.11.1 Standardisation

The Dynamic Probing test has been standardised in more than twenty countries and ISSMFE has established the International Reference Test Procedure. Because of the great variety of penetrometers in use, the following classification according to the hammer masses has been chosen.

Table 2.1: ISSMFE classification according to hammer masses

Type	Abbreviation	Mass (kg)	Drop Height (m)	Investigation depth(m)
Light	DPL	10 ± 0.1	0.5±0.01	8
Medium	DPM	30 ± 0.3	0.5±0.01	20-25
Heavy	DPH	50 ± 0.5	0.5±0.01	25
Super heavy	DPSH	63.3 ± 0.5	0.75±0.02	>25

Cone Apex angle is 90 degrees. The number of blows required to drive the point, each successive 0.1m (N_{10}) is recorded for DPL, DPM and DPH so creating a record of blows/10cm with depth of penetration of the point. For DPSH number of blows required for 0.2m penetration (N_{20}) are recorded.

2.12 Advantages of Dynamic Probing

The Dynamic Probing has the advantage compared with the Static Cone Penetration Test that hard layers can be penetrated.

Van Wambeke (1982) has pointed out that the simplicity of the equipment and of the method has made the dynamic penetration test the most economical in-situ testing method and the easiest to use. In contrast to the SCPT, no large reaction mass is required.

The Dynamic Probing Test provides a continuous record of readings in contrast to the Standard Penetration Test (SPT), but less detailed than with a continuous SCPT.

Dynamic Probing can be used to detect soft layers and to locate strong layers.

2.13 Limitations of Dynamic Probing

Dynamic Probing is mainly used in cohesionless soils

In many soils, especially in soft cohesive and in organic soils, the skin friction can have substantial effect on the penetration resistance; at the same consistency, the penetration resistance increases with depth in these cases. In these soils below the phreatic level very high excess water pressures occur.

The penetration resistance, close to the borderlines of a given soil layer will be influenced by the soil types above and below that layer being penetrated. Compressibility and inclination of the layer below the penetration location will be of influence.

2.14 The Standard Penetration Test (SPT)

The Standard Penetration Test, commonly known as the " SPT", which originated in the late twenties, is one of the most widespread methods of in situ soil investigation. It is carried out in a borehole by driving a standard 'split-spoon ' sampler, using repeated blows of a 63.5 kg (140 lb.) hammer falling through 762 mm (30 in).

The hammer is operated at the top of the borehole, and is connected to the split spoon by rods. The split spoon is lowered to the bottom of the hole, and is then driven a distance of 450 mm (18 in), and the blows are counted, normally for each 75 mm (3 in) of operation. At the end of driving the split spoon is pulled from the base of the hole, and the sample is preserved in an airtight container.

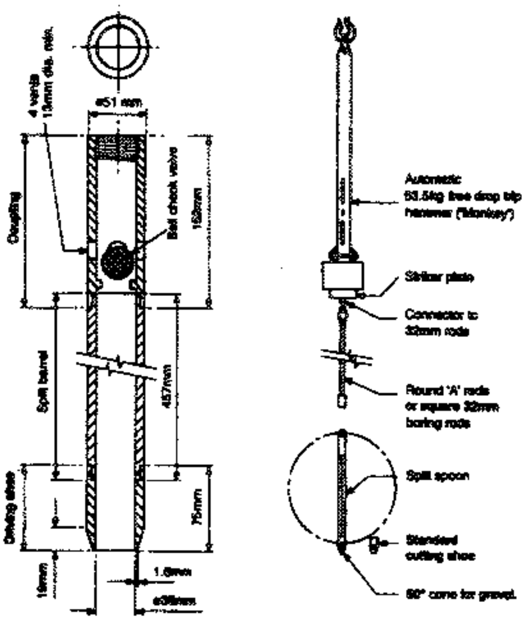


Figure 2.7: Equipment for Standard Penetration Test with an automatic trip hammer (From Clayton et al, 1995)

The penetration resistance (N) is the number of blows required to drive the spilt spoon for the last 300 mm (1 ft) of penetration. The penetration resistance during the first 150 mm (6 in) of penetration is ignored, because the soil is considered to have been disturbed by the action of boring the hole (Clayton et al. 1995).

The Standard Penetration Test (SPT) is, still one of the most commonly used in situ tests for site investigation. Empirical relations have been developed to estimate relative density, shear strength parameters. SPT is actually not properly standardised as explained in the thesis of Necla AKCA (1999). Sample facility is included (Bell, 1987).

2.15 Weight Sounding Test (WST)

The Weight- Sounding method is also known as Swedish weight sounding method, because it is used widely in Sweden. This is a simple, inexpensive and fast method.

The sounding equipment consists of a screw point, sounding rods, a handle and a number of weights. When investigating the penetration resistance of a soil, the sounding rod is gradually loaded with the weights (5 , 15 , 25 , 50 , 75 and 100 kg).

The rods and helicoidal points are pushed down until the maximum load of 100 kg is placed. The point is then rotated and the depth of penetration under the action of the load is measured. After this has been accomplished, the penetration for each 25 half-turns or the number of half-turns for a penetration of 20 cm is recorded. The penetrometer can either be operated manually or mechanically which makes its suitable for different applications(Sanglerat, 1972).

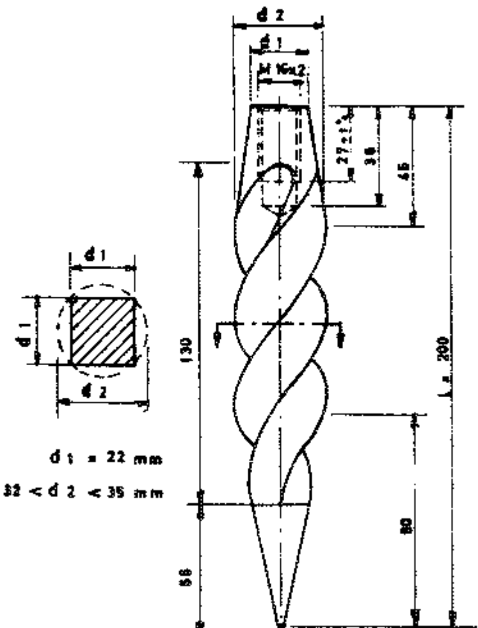


Figure 2.8: Weight sounding penetrometer(From Sanglerat, 1972)

The main application of the method is in soft to medium stiff clay and silt and in loose to dense sand. The main limitation of the method is that layers of very dense sand or gravel or layers of glacial till are very difficult to penetrate.

Weight soundings are primarily used during the exploratory phase of a soil investigation to determine the depth and the thickness of the different strata. Weight sounding can also be used to check the compaction of fills and the diggability of soil (Broms and Flodin , 1988).

2.16 Correlations of Dynamic Probing Tests with Static Cone Penetration Tests made by various authors

Butcher et al. (1996) from UK presented the correlation between Dynamic Probing data and Static Cone Penetrometer tests. Dynamic Probing was carried out at 10 clay soil test bed sites, 6 in the UK and 4 in Norway, to cover a reasonably wide range of plasticities.

Data from ten test bed sites have been used to correlate Dynamic Probing point resistance q_d with Static Cone resistance q_t. The stiff clay data are shown in the figure 2.9 which includes a line,

$q_d = q_t$

where $$q_d = \frac{M}{M+M'} * \frac{Mgh}{Ae}$$

[numbers!]

q_d are resistance values in (Pa)

M is the mass of the hammer in (kg)

g is acceleration due to gravity in (m/sec^2)

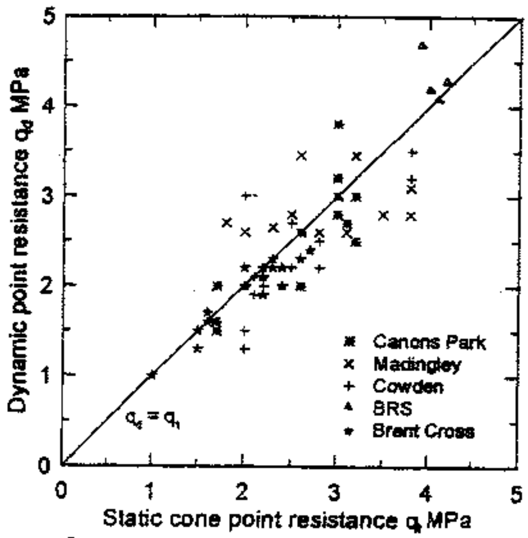
h is height of fall of hammer in (m)

A is the area at the base of the cone in (m^2)

e is the average penetration in (m) per blow

M' is the total mass of the extension rods, the anvil and the guiding rods in kg.

The line fits the data reasonably well .



what clay?

what end

Figure 2.9: Dynamic point resistance Vs static cone point resistance for stiff clays

after?

The soft clay data are shown in figure 2.10, which includes the correlation line $q_t = 0.2q_d + 0.14$

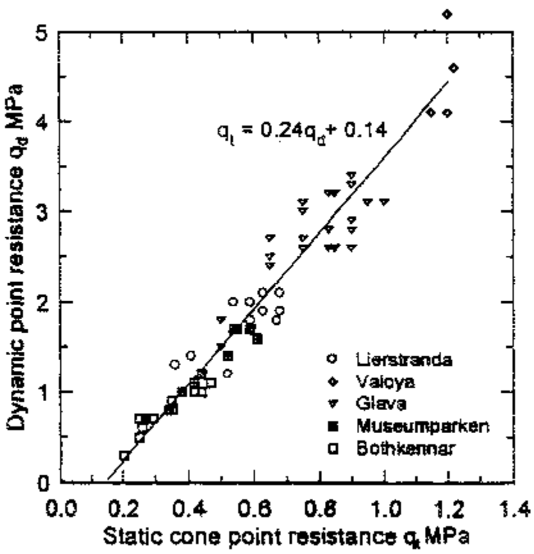


Figure 2.10: Dynamic point resistance Vs static cone point resistance for soft clays

after?

Anatolie Marcu and Cezar Culita from Romania have presented the correlation between SCPT and DPL in sands (Proceedings CPT' 95) and the following relation has been given

$q_c = 0.2\ N_{10}$

It has been mentioned that most of the correlations were drawn between the values q_c and the number of blows necessary to penetrate 20 cm (N_{20}) recorded by the DPH in sands(Marcu, 1995).

The correlations established by one of the authors are represented in figure 2.11.

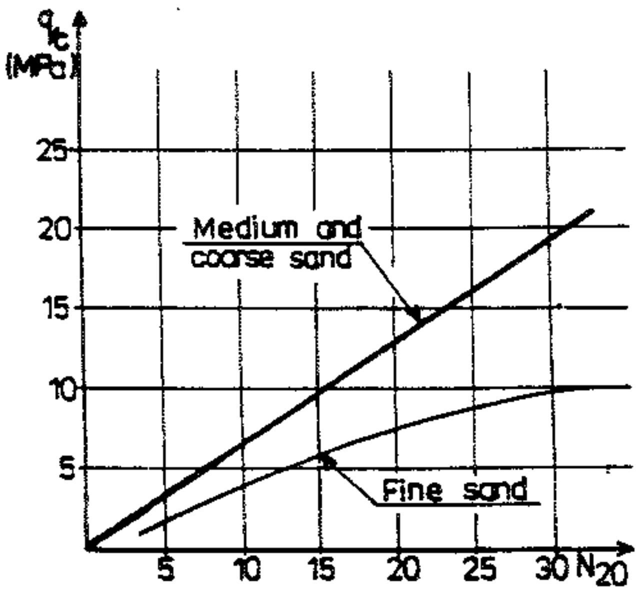


Figure 2.11: Correlations between SCPT and DPH results in sands (Proceedings CPT' 95)

In case of cohesive soils, the correlations between the SCPT and the DPT proved to be less stable. robust?

According to an internal report of Fugro, the TU(Technical University) in Aachen(Germany) found the following relation for sands using DPH in relation to q_c in kgf/cm^2:

$\log q_c = 1.854\ \log N_{20} + 1.065 \pm 0.110$

The following relations between the number of blows of Heavy Dynamic Probing and the cone resistance qc (MN/m^2) are suggested by Kralik (1984b) in Hungary.

For cohesionless soils:

$q_c = 1.095 + 0.476\ N_{20}$ (on the basis of 470 data pairs)

For silty sands:

$q_c = 0.790 + 0.515\ N_{20}$ (on the basis of 110 data pairs)

For clayey soils:

$q_c = 0.850 + 0.296\ N_{20}$ (on the basis of 50 data pairs)

The DCPT (dynamic cone penetration) test very similar to DPH reference test, is very widely used in India since 1966. Desai, (1974) made the correlation of SCPT with DCPT N_{20}, (equivalent to DPH) and reported the relation

$q_c = K \times N_c\ kg/cm^2$

This ratio varies with effective overburden pressure and is sensitive to water content.

Table 2.2: Correlation of SCPT & DCPT (N_{20}) by Desai M.D. (1974)

Soil type	Silty clay	Silty fine sand	Fine sand	Coarse sand	Gravely sand
K(kg/cm^2)	2	2.27	4	8	10
K(MN/m^2)	0.2	0.227	0.4	0.8	1.0

2.17 Interpretation of the penetration test results

Besides the factors related to test equipment and procedures, at least the following soil related parameters can additionally influence the penetration resistance:

density/consistency
soil cementation
grain size distribution , grain shape and roughness
mineral type
geological history of the soil
anisotropy
structure
stress- strain history
in situ stress state
sensitivity

2.18 Soil classification

Some of the most comprehensive work on soil classification using SCPT data was presented by Douglas and Olsen, 1981 (in Lunne, 1997).

The charts confirm early observations from Holland that sandy soils tend to produce high cone resistance and low friction ratio, whereas soft clay soils tend to produce low cone resistance and high friction ratio.

In recent years, soil classification charts have been adapted and improved based on an expanded database. Factors such as changes in stress history, insitu stresses, sensitivity, stiffness, macrofabric, minerology and void ratio will also influence the classification.

2.19 Discussions

From literature review, it has been found that only few correlations exist between SCPT and Dynamic Probing test results whereas correlation between SPT and SCPT data is very common. The available correlations have been made between the test results of SCPT and different versions of Dynamic Probing due to large variations in the type of equipment. It has been found that Dynamic Probing tests in Germany are carried out in non- cohesive soils.

Only few correlations have been made between the results of two tests. No details have been given about statistical approaches which have been used to correlate the test results but it has been mentioned that tests results which are normally subject to considerable scattering , should be analysed using statistical approaches.

The following relations between the number of blows N_{20} in Heavy Dynamic Probing and the cone resistance qc (MN/m^2) are suggested by Kralik (1984b) in Hungary.

For cohesionless soils:

$q_c = 0.476\ N_{20} + 1.095$ (on the basis of 470 data pairs)

Desai (1974) made the correlation of SCPT with DCPT N_{20}, (equivalent to DPH) and reported the relation

$q_c = K \times N_c$ (MPa)

For gravely sand, K=1 (MPa)

This ratio varies with effective overburden pressure and is sensitive to water content.

Because of the purely empirical nature of these correlations it is important to be aware of their many limitations. The correlations often partly account for complexities in natural soils.

For the application of driving energy in Dynamic Probing, two facts are important:

- guarantee of free fall of the hammer
- guarantee of constant height of fall of the hammer

The automatic latch arrangement avoids operator influence on the test which is inevitable with hand operated or manual latch type equipment.

Test equipment with cone/rod diameter ratios exceeding about 1.3 leads to results in cohesionless and in many cohesive soils that are little or not at all seriously influenced by skin friction.

Skin friction is avoided practically completely if drilling mud is used during the performance of the test. It can be concluded that Dynamic Probing tests can be carried out with a more reliable procedure than Standard Penetration Tests.

3. Brief description of the geology of the area

3.1 Brief description of geology

The area of South Limburg is situated to the north of the Hercynian Ardennes-Rhenish Massif and at the eastern margin of the Caledonian Brabant Massif. The direct vicinity of these two massifs greatly influenced sedimentation. From the upper Carboniferous onwards sedimentation was also significantly affected by the NW-SE trending faults of the Roer valley Graben (Bekendam, 1998).

During the worldwide transgression period of the Upper Cretaceous the Brabant Massif was flooded (Ziegler, 1982). Also the Ardennes-Rhenish Massif was reached by this transgression

It was already recognized in the 19th century that the Upper Cretaceous sediment succession is a transgressive - regressive sequence (Zijlstra, 1994). The regression is considered by various authors to be brought about by epi-orogenetic movements (Zijlstra, 1994). During the Tertiary the Upper Cretaceous sediments were uplifted above sea-level and the limestone became affected by dissolution and Karstification.

The period from Upper Miocene to present time is characterised by the deposition of alluvial sediments of the river Maas and its tributaries. The Maas deposits are found over much of Southern Limburg. The spread of these deposits is caused by Alpine tilting of the foreland of the Ardennes during the Pliocene. The tilting preceded the Maas river terrace deposition; material was deposited as an alluvial fan over the Limburg area. The Pliocene and Pleistocene sediments contain for that reason clay, sand and gravel partly derived from this fan.

Continuation of tectonic uplift and sea level fluctuations caused new incisions of the river Maas in the underlying limestone. Now a more defined river, called the "East-Maas" found its way in the Limburg area flowing from Maastricht in easterly direction between the Ardennes and the "Isle of Ubachsberg" to merge near Duren with the River Rhine. In its valley several matched river terraces mark several incisions in the same riverbed.

Research confirmed that nearly all Maas river terraces could be differentiated, based on height above sea level, lithological content and quartz content. The direction of flow of the present Maas river was caused by late alpine tilting in an increasingly northern-easternly direction. The break through its northern embankment near Maastricht was dated 1.7 million years ago. The older sets of gravel terraces were subjected to erosion by a new drainage system consisting of the Geul and Gulp streams, which connect to the Maas (Maurenbrecher et al, 1990).

Commonly Upper Pleistocene loess deposits are to be found at the topographic surface. The major exposure of Loess in the Netherlands is restricted to South Limburg. The Loess forms part of the Loess belt, which can be traced from northern France through Belgium and the Netherlands into Germany.

3.2 Geology of South Limburg in the context of the geological map of the Netherlands

The oldest geological units on the map of the Netherlands are all only found in very small areas in South Limburg. These are:

Upper Carboniferous: Upper carboniferous is covered with younger deposits. Only one small outcrop could be seen on the map (a geological monument, the Heymans groeve).

Cretaceous: These are also covered with younger deposits. The upper Cretaceous Limestones are exploited in several quarries.

Tertiary: Lower part of tertiary consists of Limestone and the upper part contains clays, sands and some gravels.

Quaternary: It consists of deposits of East Maas; Sandy gravels, followed by deposits of West Maas i.e. several terraces of sandy gravels and Loess.

3.3 Engineering geology of Loess in South Limburg

In general, the mechanical properties of loess are a direct consequence of the particle size. The material has great mechanical stability until it is disturbed and the interparticle bonds are ruptured. Probably, the failure mechanism is a combination of the fracture of cement bonds by loading and saturation (Price, 1991).

The Loess in South Limburg shows geotechnical behaviour, which differs, from the collapsing Loess found elsewhere. High densities qualify the Loess as being almost non-collapsible. The maximum thickness of the Loess deposit is about 20 meters.

In the Loess deposits of South Limburg, the main problem is erosion. During heavy rainfall, the Loess erosion can be very high and cause many problems. The most widespread erosion is sheet erosion. On slopes steeper than 10%, gullies can easily develop if the Loess is not ploughed every year.

When a comparison is made between South Limburg Loess and Loess from China, Britain and Canada, the Loess in South Limburg seems better sorted than the other Loesses. In the Loess of South Limburg clay content and carbonate content varies.

In the carbonate rich Loess (carbonate content upto 24%), carbonate can be found as a coating or encrustation on the grains. Cryoturbation process has densified the Loess. SCPT results show higher values sometimes. Generally the cohesion of South

Limburg Loess ranges between 6-10 kPa indicating good relationship to cementation and clay content and the angle of internal friction has values between 29-32 Degree(R.R.Kronieger, 1990).

Generally unconfined compressive strength of Loess in South Limburg varies from 125-435 kPa.

In the reports of Fugro received with the data, the term Loam has been used for the Loess of South Limburg which is sandy in many cases.This is done because "Loam" is mentioned as a name for a soil type in the Netherland standard NEN 5104. However, the soil has been classified as Loess by the Geological Survey of the Netherlands.
In this thesis, the term Loess will be used . Loam also exists at the bottom of glaciers called Moraine. So to distinguish between wind blown particles and Moraine, the term Loess is more suitable.
The general soil profiles of South Limburg area have been presented in Appendix-F

3.4 The Maas river terraces

There are three main sets of gravel terraces related to the change in bedding from the Maas river from the Pleistocene onwards. Two major sets are recognised, a "high" terrace set at 110-160m above sea level and "middle" terrace set at 30-65m above sea level. The "low" terrace group reflects the Holocene bedding of the Maas.

Besides the well rounded, bleached flint pebbles the gravel contains also pebbles, which originate in the Paris basin. Main mineral components are quartz varieties, sandstone and several metamorphic rocks. The Maas deposits mainly consist of gravels as well as sand and some clay layers are also incised in between sand and gravel. The angle of internal friction varies from 35-40 for gravely sand. The Loess unit forms the cover of Pleistocene gravel terraces of Maas river.

3.5 Information of deposits tested with SCPT/DPT

SCPT and Dynamic Pobing tests have been carried out mostly in Loess and Sand & Gravel. Only in Gulpen, Limestone is also tested with SCPT. In Maastricht SCPT tests have been carried out in Loess and Sand and Gravel but stopped in Limestone formation.

In this study it has been found that testing by Light Dynamic Probing in South Limburg is not appropriate. The maximum depth reached is nearly 6m. Light Dynamic tests have been carried out in dense Sand/Gravel mixture, whereas according to DIN 4094, the equipment has limitations of application in such soils.

The maximum depth reached by SCPT is nearly 24.8m in Nuth. In Gulpen SCPT tests have been carried out in weathered Limestone. Verticality of equipment is very much disturbed and therefore readings are not reliable.

Also tests have been carried out using electrical cone. The use of electrical cone bears in it the high risk of loosing the (expensive) cone. In these places mechanical SCPT and Heavy Dynamic Probing testing is more appropriate. The Heavy Dynamic Probing tests have been carried out to a maximum depth of 20m.

4. Probing and penetration testing(procedure and equipment)

4.1 Static Cone Penetration Test

The basic principle of cone penetration test is that a rod is pushed into the ground and the resistance on the tip of the rod is measured by a mechanical, electrical or hydraulic system.

As already mentioned, in this research study data provided by the Fugro B.V, the Netherlands has been used. For SCPT tests standard equipment according to NEN 3680 has been used.

- Mechanical cone penetrometer(63 tests approximately)
- Electrical cone penetrometer
  - With friction sleeve and inclinometer(5 tests)
  - With inclinometer only(9 tests)

4.1.1 Mechanical cone penetrometer

The cone(apex angle 60°) is pushed to the required depth by extending pressure on the outer sounding tube. The load cell at the top of inner rods is applied for a continuous graph of the cone resistance. The tubes and inner rods are simultaneously pushed downwards at a rate of 20mm/sec.

4.1.2 Electrical cone penetrometer

In electrical cone penetrometer, cone is advanced at a uniform rate of penetration by pressure on the top of the sounding tube and signals from the strain gauges are carried by cable to the recording equipment in the cabin of the truck. The modern recording devices produce a graphical output for interpretation at the spot. Moreover the data are stored on magnetic tape or diskette for compilation and final graphical output with the aid of a computer and plotter (Dekker, 1991).

4.2 Dynamic Probing test procedure

The Dynamic Probing test consists of driving a point(apex angle 90°) into the ground, via an anvil and extension rods, with successive blows of a free fall hammer. The number of blows required to drive the point each successive 0.1m (N_{10}) or 0.2m (N_{20}) is recorded, so creating a record of blows/0.1m or blows/0.2m with depth of penetration of point.

The graphs received from Fugro-Maastricht indicate that tests of Dynamic Probing have been carried out according to light and heavy versions of Dynamic Probing. Figures 4.1 & 4.2 indicate Heavy and Light Dynamic Probing equipments respectively. Types and application possibilities of Dynamic Probing equipment according to DIN 4094(German standard) have been given in table 4.1.

Figure 4.1: Heavy Dynamic Probing test in Maastricht

Figure 4.2: Light Dynamic probing Test in Heerlen

Table 4.1 Types and application possibilities of Dynamic Probing equipment according to DIN(4094)

Name	Short code	Cone area $A_c(cm^2)$	Cone diameter d (mm)	Mass of hammer m (kg)	Drop height h(m)	Testing depth below starting point (m)	Restrictions for the application (soils according to DIN 4022 part 1)
Light Dynamic Probing	DPL	10	35.7 ±0.3	10 ±0.1	0.50 ±0.01	10	medium dense & dense gravels, hard clays
Medium Dynamic Probing	DPM	10	35.7 ±0.3	30 ±0.3	0.50 ±0.01	20	dense gravels
Heavy Dynamic Probing	DPH	15	43.7 ±0.3	50 ±0.5	0.50 ±0.01	25	No restrictions

4.3 Comparison of static and dynamic testing

Both tests provide continuous record of readings but Dynamic Probing is based on counting number of blows for 0.1m (N_{10}) or 0.2m (N_{20}) penetration of rods. However in SCPT, metal rods are pushed into the ground at constant rate. It has been observed that SCPT provides much more details (indicating sharp peaks where soil boundaries are encountered) as compared to Dynamic Probing test. In very soft soils Dynamic Probing can not result in appropriate blow count values.

5. Data Interpretation

5.1 Source of data

Data for this research study was received from Fugro B.V, regional office Maastricht Netherlands. All of the projects have been carried out in the South Limburg area of the Netherlands. Most recent available data has been used in this research study. Overview of the data has been given in Appendix-F(table F1).

All projects have been carried out in the year 1998-1999. Almost every project has been provided with the location map and the soil description report of the test site.

5.2 Selection of the data for correlation

5.2.1 Distance criterion

Data for the correlation work has been selected on the basis if both SCPT and Dynamic Probing tests have been carried out at the same test site i.e, both tests have been plotted on the same location map. This selection criteria is based on the accuracy of the distance measurement between two types of tests.

All selected SCPT and Dynamic Probing data sets lie in the range of maximum 30 m distance from each other. This distance range has been selected to make best possible use of available data.

5.3 Interpretation of data

In the initial stage of research, several trials were made to interpret the data. For correlation of the two test results, it is necessary that equal number of readings be obtained from both tests. Dynamic Probing test produces constant reading at 0.2m interval while SCPT shows variation within 0.2m interval. So several methods of data interpretation were tried.

Depending upon the availability of data at the initial stage, depth criterion was established for interpretation.

5.3.1 Depth criterion

The data of P-2522(Maastricht city) was interpreted in three different ways. The objective was to see the effect of averaging on the results.

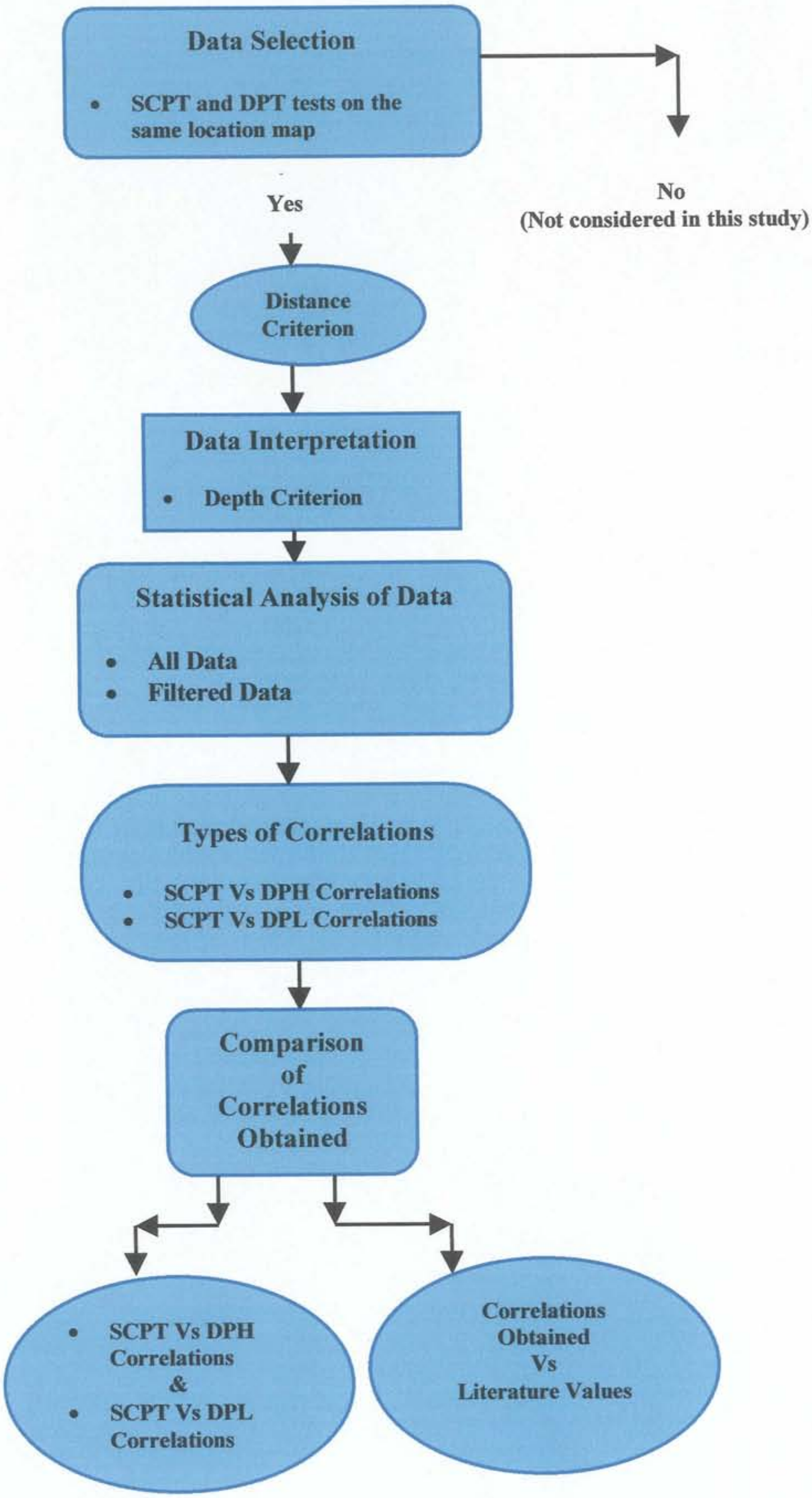
- 0.2m selected data
- 0.4m average data
- 1.0m average data

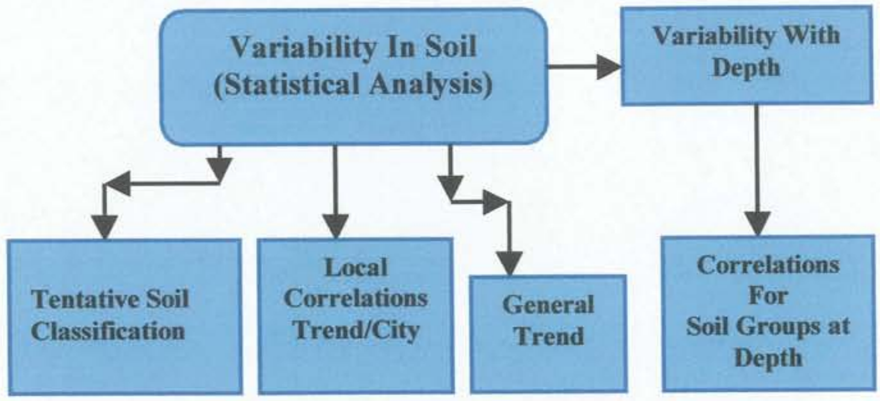
In the first trial, SCPT readings were taken at each 0.2m to have equal number of data from both tests since Dynamic Probing test readings are given at 0.2m.

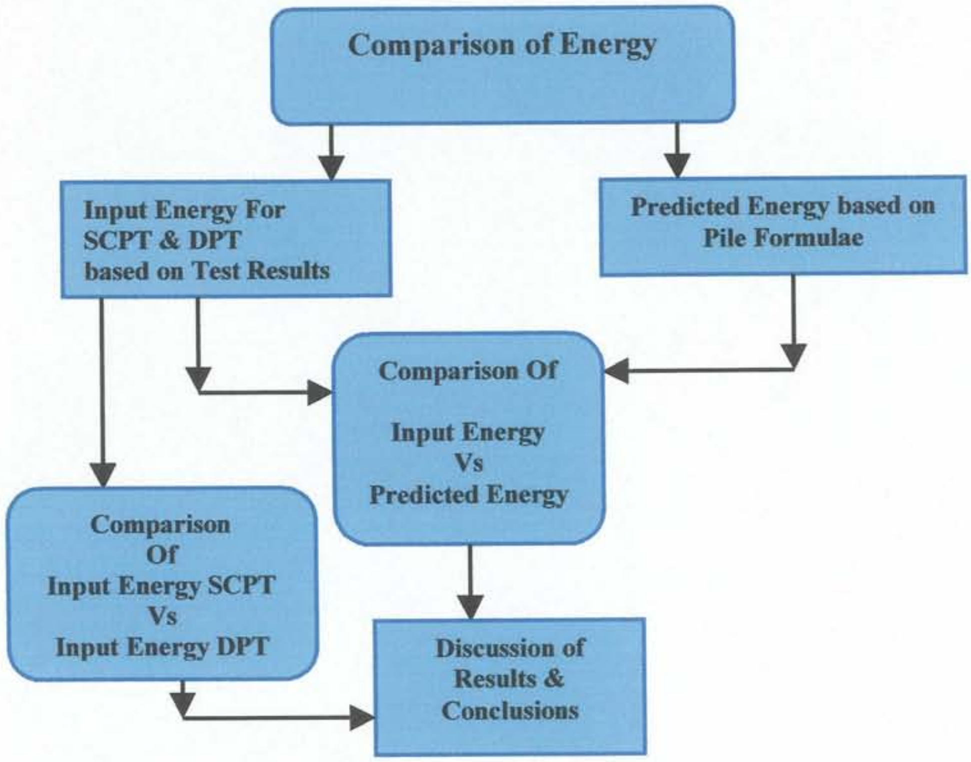
Although both tests provide continuous record of readings but it has been found from the interpretation of the graphs that SCPT gives more details than Dynamic Probing test and also the number of readings of SCPT are more than Dynamic Probing test readings in a certain range of depth. For this reason it was considered to average the readings of both tests at 0.4m and 1.0m interval for each soil type(Loess, Sand & Gravel) for comparison purposes so that all readings of SCPT can be taken into account.

In this stage of the research, the data was also analysed for all the three trials to check the effect of averaging on the results. Few trials of 0.2m average data correlations and 1.0m average data correlations have been given in Appendix-F.

Flow Chart Of Procedure




Variability In Soil
(Statistical Analysis)
Variability With Depth
Tentative Soil Classification
Local Correlations Trend/City
General Trend
Correlations For Soil Groups at Depth


Comparison of Energy
Input Energy For SCPT & DPT based on Test Results
Predicted Energy based on Pile Formulae
Comparison Of Input Energy Vs Predicted Energy
Comparison Of Input Energy SCPT Vs Input Energy DPT
Discussion of Results & Conclusions

It was realised that while considering 0.2m readings several peaks in the SCPT data were not taken into account so the data was not truly represented. When considering the 1.0m averaging, it was found that different soil types were mixed at the boundary where one soil type was changing into another type. Finally it was found that 0.4m averaging was most suitable since at the boundary of soil types where readings were averaged, the chances of mixing the two soil types were minimised and the maximum use of available data could be made.

So the 0.4m averaging was considered to be the best among all the applied methods and it was decided to adopt this method for the rest of the data.

5.4 Soil classification

According to the borehole information, Loess was subdivided. This subdivision could be recognised mainly on the basis of colour difference. Approximate boundaries based on q_c values were established in the SCPT tests near boreholes. Then these were extended by contouring to SCPT and dynamic tests far from boreholes. But it was realised that vague results were obtained and the method did not work. Therefore, the description of the subsoil given by Fugro with the data set was used.

So Loess has been considered as one single type of soil and the boundaries between Loess and Sand & Gravel have been established on the basis of SCPT results together with the help of available parts of the Fugro reports.

5.4.1 Interpretation in Sand & Gravel

Some very high and low values were noted for Sand & Gravel data from both Dynamic Probing and SCPT tests. It can be said that variability in Sand & Gravel shows low and high averages. Another possible explanation of high and low readings is the angle at which the cone of the equipment hits a gravel particle.

Since the size of the cone is comparable to the size of the gravel and if it hits the gravel at its centre then it may break the gravel and high reading will be noted. While on the other hand if it hits the gravel at a corner or at such an angle that it is pushed aside then low reading will be recorded. Also when cone penetrates the soil it pushes the soil aside and densification takes place due to which high readings can be achieved at levels where the existing density is high.

5.5 Selected number Of SCPT and Dynamic Probing tests

For this correlation research work, the selected number of SCPT and Dynamic Probing tests are as follows:

Total number of SCPT tests:	60
Total number of Heavy Dynamic Probing tests:	22
Total number of Light Dynamic probing tests:	8
The number of data pairs(q_c , N_{20}) available for correlation:	446

5.6 Types of correlations

Dynamic Probing tests have been carried out according to two versions i.e.; Heavy Dynamic Probing denoted by (SZ) and Light Dynamic Probing denoted by (SL). Two types of correlations have been made.

- SCPT Vs Heavy Dynamic Probing
- SCPT Vs Light Dynamic Probing

5.7 Statistical method used:

As the Dynamic Probing tests and SCPT tests are scattered (except very few tests) therefore, for analysis of the data, statistical method needs to be applied to take the effect of distance into account before any correlation can be made. So, inverse distance weighted averaging method has been applied to establish the weighted value, which considers the distance effect.

5.7.1 Inverse distance weighted averaging:

A point estimate calculated as a simple average of all chosen values will clearly be unsatisfactory. A simple solution to this is to weight the control point values according to the inverse of the distance to the grid point being estimated. For example, if three control points are to be used and these have values z_1, z_2 and z_3 and are at distances d_1, d_2 and d_3 from the estimated point, the estimated value z' is calculated by:

$$z' = \frac{(z1/d1) + (z2/d2) + (z3/d3)}{(1/d1) + (1/d2) + (1/d3)}$$

I can read this info! in the function!

This is simply a weighted average of z_1, z_2 and z_3, in which the z values are multiplied by the weights, summed, then divided by the sum of the weights. Weights are standardised so that they sum to one.

The use of inverse distance is fairly arbitrary and relative weight applied to near and far points can be varied by using other distance transformations, e.g. $1/d^2$, $1/d^3$. A value of 1 tends to smooth the prediction results, a value of three tends to emphasise local high and low values, 2 is the usual compromise (Swan and Sandilands, 1995).

In application of inverse distance weighted average method; power 2 has been used. All calculations of the inverse distance weighted average data sets have been given in Appendix- A .

5.8 Statistical analysis of the data

Data distribution in the form of histogram has been given in figures 5.1 to 5.3 (See pages 51 & 52). It does not fit into perfectly normal distribution but still on the basis of cumulative percentage curves it is possible to apply statistical methods. Log normal distribution was tried but it did not lead to better results.

The data was analysed with two types of statistical analysis.

- For each soil type all available data was analysed(for Heavy and Light Dynamic Probing separately).
- Data was filtered i.e. data far from general trend of q_c versus N_{20} was disregarded .

Data was filtered according to the statistical criterion (Mean±2 St Dev). The purpose of filtering the data was to disregard the data situated far away from the general trend. Even after filtering the data, 95% data was still considered in the analysis. The filtered data has been indicated by the light shade in Appendix-B. Filtering is based on SCPT values since it provides more details as compared to Dynamic Probing test.

For choosing the correlation coefficient, several trend lines were established for each type of soil. Then finally it was recognised that soils of South Limburg mostly show the best results with:

- the linear correlation, trend line with intercept obtained by method of least squares, $q_c = aN+b$
- Also power correlation, $q_c=cN^d$, trend line obtained by method of least squares has been tried.

The most suited correlation function (equation) and correlation coefficient has been determined for all data as well as for filtered data. Each type of soil has been treated separately.

Data has been analysed for Heavy Dynamic Probing versus SCPT and Light Dynamic Probing versus SCPT separately since both types of dynamic tests give different results for the same type of soil. The trend line equation and coefficient of correlation for all data and filtered data for (DPH Vs SCPT) and (DPL Vs SCPT) has been given in tables 5.1-5.4 & tables 5.6-5.9 respectively.

Heavy Dynamic Probing Versus SCPT

Table 5.1: All data analysis (Heavy Dynamic Probing versus SCPT) for linear correlation with intercept.

Soil Type	Linear Regression y=ax+b	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	q_c=0.1803N+1.4031	0.3966	0.63
Sand/Gravel	q_c=0.5318N+14.639	0.3786	0.61

Table 5.2: Filtered data analysis (Heavy Dynamic Probing versus SCPT) for linear correlation with intercept.

Soil Type	Linear Regression y=ax+b	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	q_c=0.1564N+1.5359	0.3029	0.55
Sand/Gravel	q_c=0.4924N+16.712	0.3588	0.6

Table 5.3: All data analysis (Heavy Dynamic Probing versus SCPT) for Power correlation .

Soil Type	Power Correlation $y=cx^d$	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	$q_c=0.8823\ N^{0.5421}$	0.33	0.57
Sand/Gravel	$q_c=2.3414N^{0.7299}$	0.3534	0.59

Table 5.4: Filtered data analysis (Heavy Dynamic Probing versus SCPT) for Power correlation .

Soil Type	Power Correlation $y=cx^d$	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	$q_c=0.9458N^{0.4983}$	0.2775	0.53
Sand/Gravel	$q_c=3.9563N^{0.594}$	0.3338	0.57

It can be seen from the above results that best correlation coefficients have been achieved from linear correlation with intercept and positive correlation has been found between the two tests i.e. if number of blows of dynamic test (N_{20}) increase, also the q_c values of SCPT will increase and vice versa. Tables 5.1 & 5.2 show that filtering does not improve the correlation coefficient. Sand and Gravel has very high intercept, (see table 6.1).

It can be seen that correlation coefficient decreases after removing outliers which means that some high and low values were included in the data of each soil type which were different from the general values of the particular soil.

This can be due to thin sand layers incised in Loess, giving some very high values and incision of some clayey or silty layers in Sand/Gravel, giving very low values compared to Sand/Gravel general trend of values.

which basically indicated that the correlation is very low

The correlation coefficients for linear correlation ranges from 0.61 to 0.63 for all data whereas range for filtered data is from 0.55 to 0.6. However ~~there is no~~ significant difference between the values of all data and filtered data.

The linear equations with intercept are valid for certain range of values. The lowest and highest values that were included in the correlations are listed in the table below.

Table 5.5: Lowest and Highest values included in the correlations

Soil Type	Lowest		Highest	
	q_c (MPa)	N_{20}	q_c (MPa)	N_{20}
Loess	0.423	1	6.929	28
Sand & Gravel	3.881	11	61.756	95

Light Dynamic Probing versus SCPT

Table 5.6: All data analysis (Light Dynamic Probing versus SCPT) for linear correlation with intercept.

Soil Type	Linear Regression y=ax+b	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	q_c=0.023N+2.2341	0.0593	0.24

Table 5.7: Filtered data analysis (Light Dynamic Probing versus SCPT) for linear correlation with intercept.

Soil Type	Linear Regression y=ax+b	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	q_c=0.0263N+2.1143	0.0826	0.29

Table 5.8: All data analysis (Light Dynamic Probing versus SCPT) for Power correlation .

Soil Type	Power Correlation $y=cx^d$	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	$q_c=1.0427N^{0.2826}$	0.0823	0.29

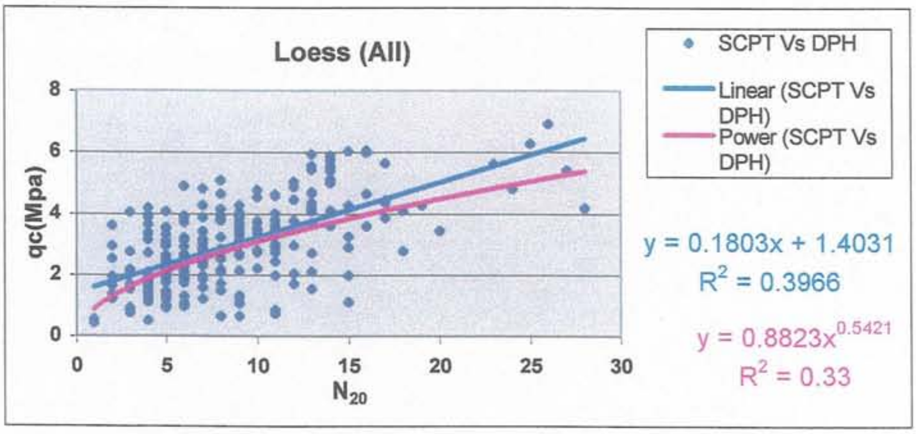
Table 5.9: Filtered data analysis (Light Dynamic Probing versus SCPT) for Power correlation .

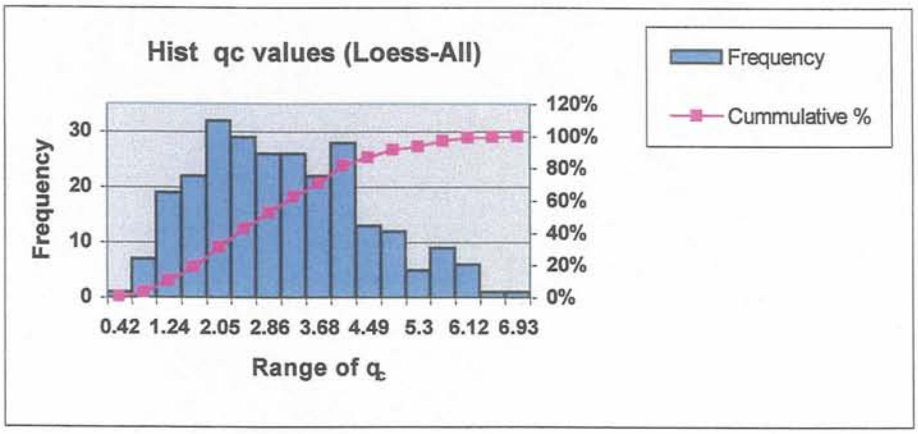
Soil Type	Power Correlation $y=cx^d$	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	$q_c=0.9339N^{0.315}$	0.1036	0.32

The correlation of data of Light Dynamic Probing tests with SCPT tests indicates positive correlation for Loess but the correlation coefficient is poor. The correlation coefficient increases slightly for the Loess with the removal of outliers. In case of Light Dynamic Probing data, power correlation shows slightly improved correlation coefficient (in Filtered data) than linear correlation coefficients but the difference is not significant.

Table 5.10: Lowest and Highest values included in the correlations

Soil Type	Lowest		Highest	
	q_c (MPa)	N_{20}	q_c (MPa)	N_{20}
Loess	0.417	3	5.668	74





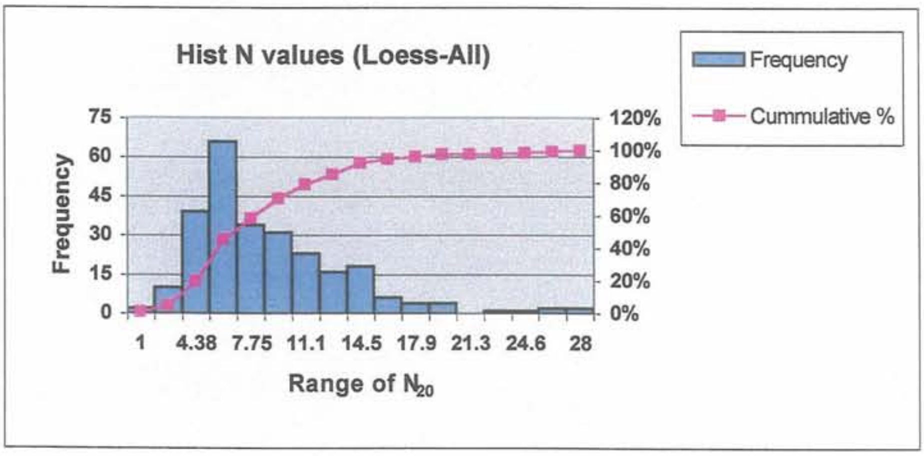


Figure 5.1: Statistical analysis of all data of Heavy Dynamic Probe & SCPT for Loess

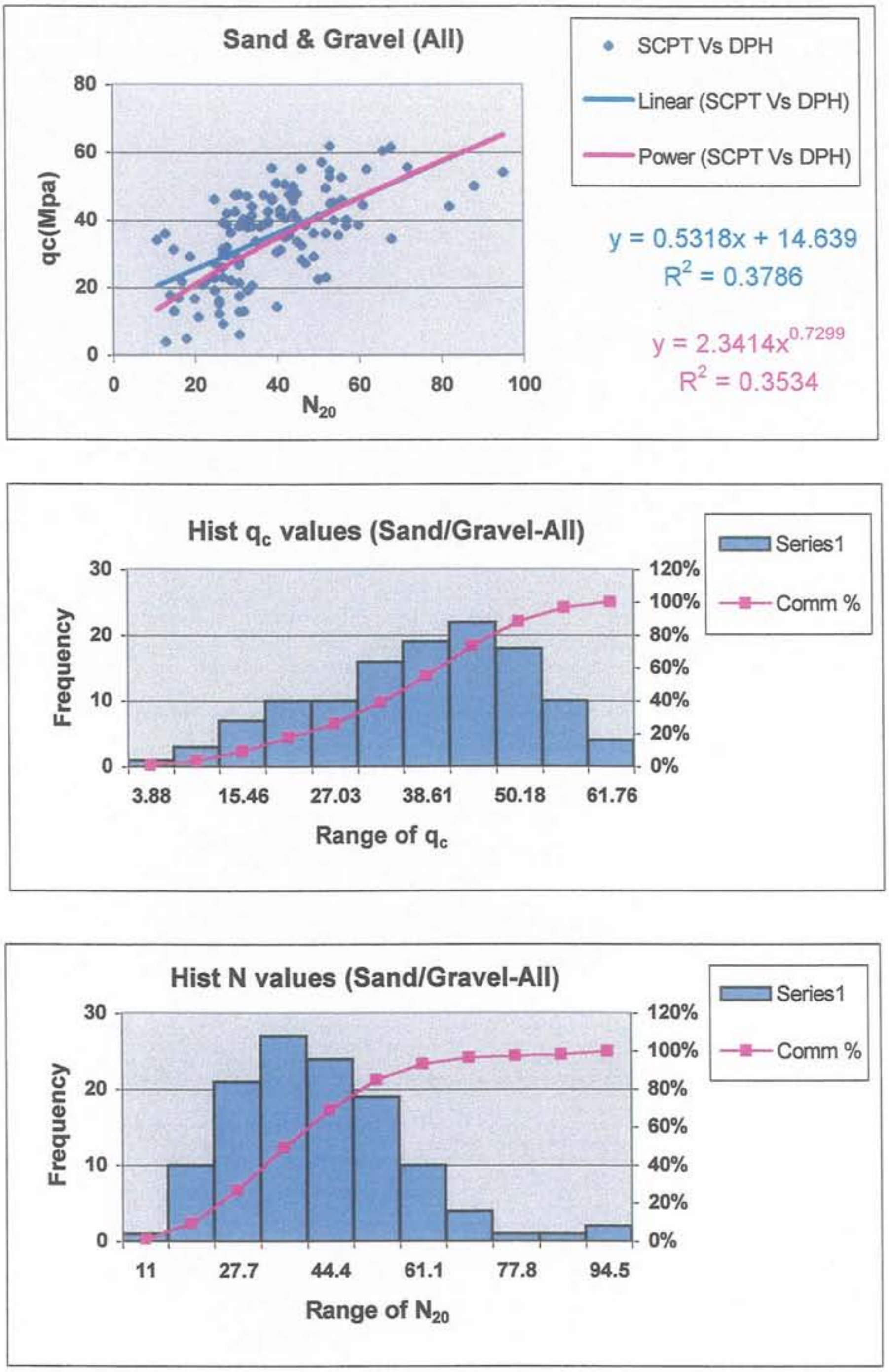
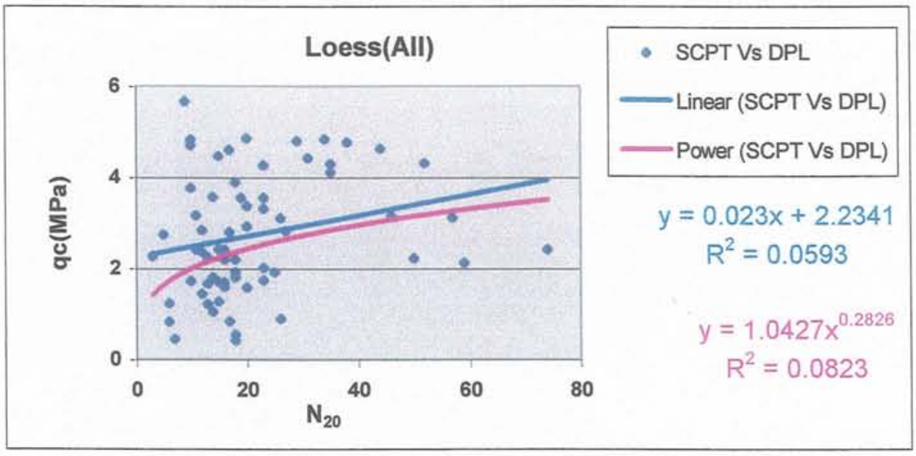
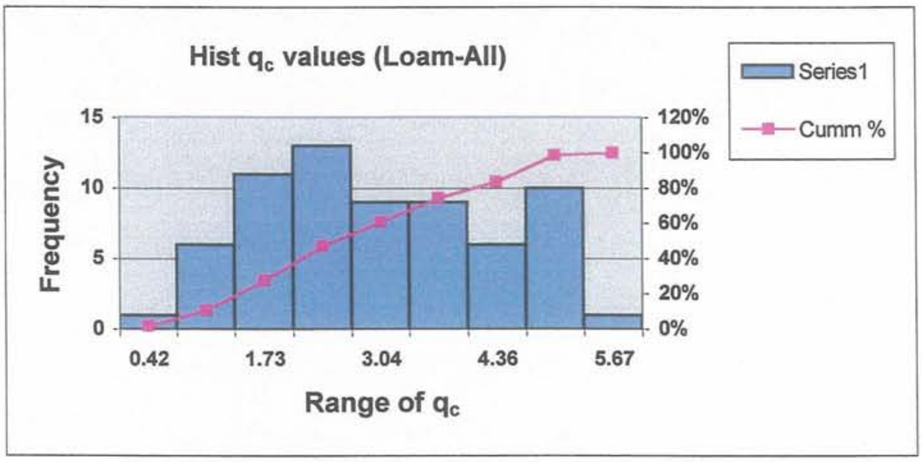


Figure 5.2: Statistical analysis of all data of Heavy dynamic probe & SCPT for Sand/Gravel





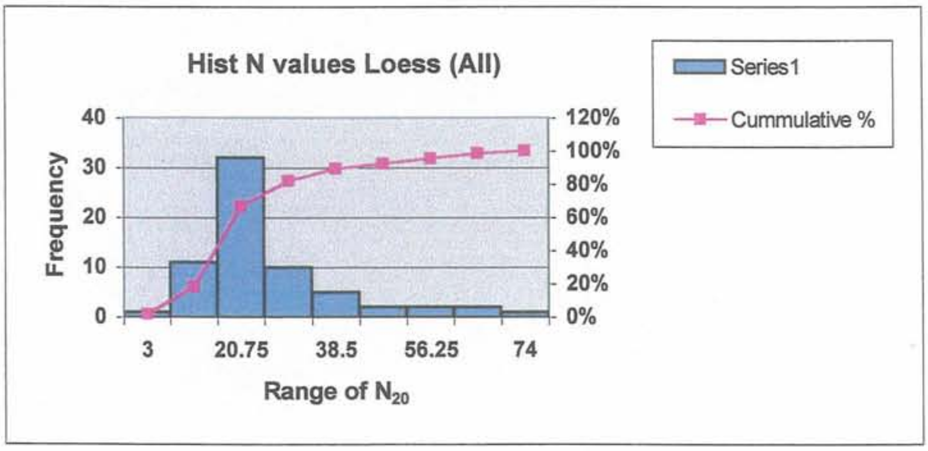


Figure 5.3: Statistical analysis of all data of Light dynamic probe & SCPT for Loess

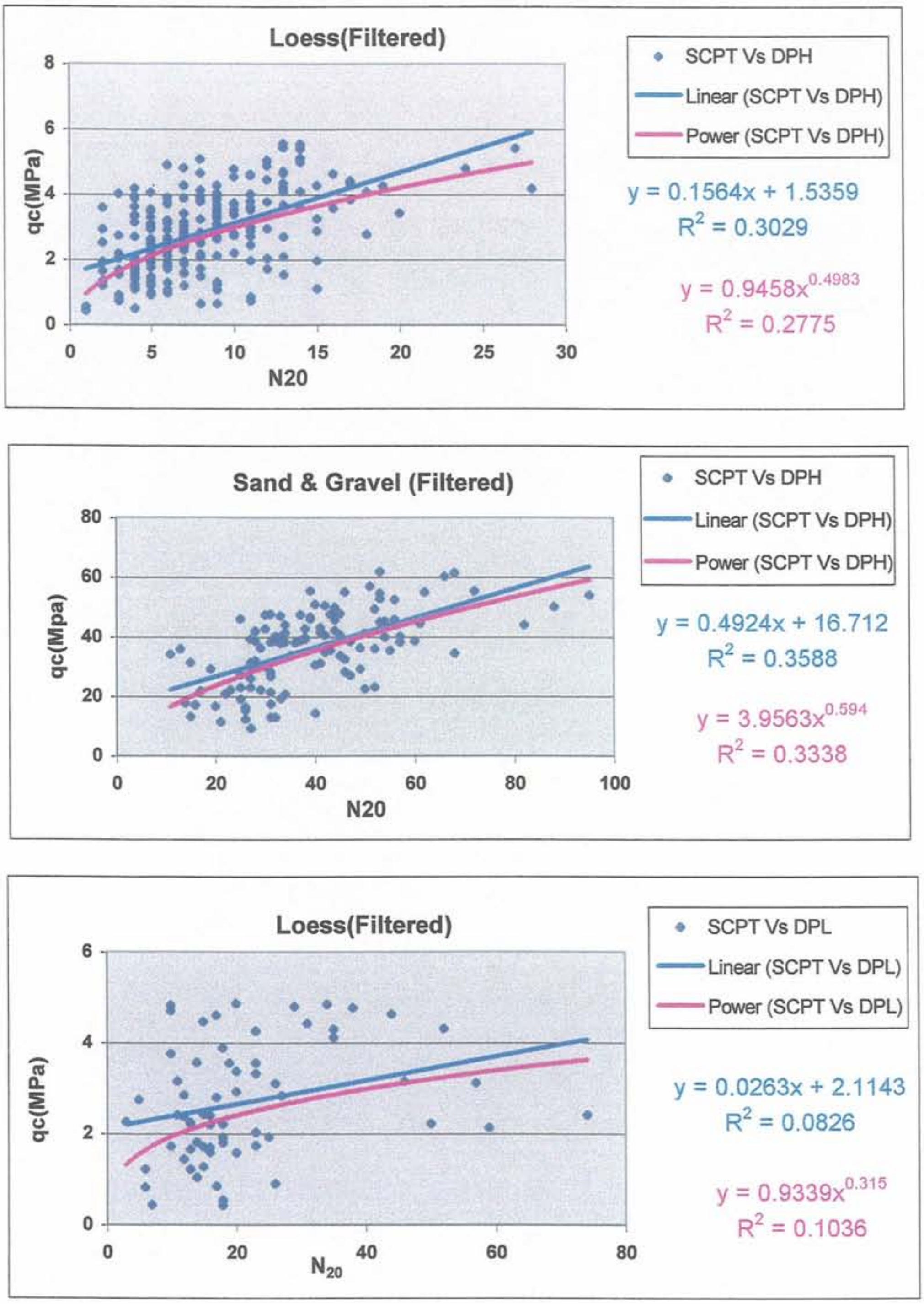


Figure 5.4: Correlations for filtered data

So considering the results of correlations achieved, it can be said that soils of South Limburg area indicate the general trend of linear correlation with intercept for both types of tests. In the figures 5.3 and 5.4, it can be clearly seen that the reliabilities of the correlations near the ultimate values are lower than in the "central" part of the graphs.

5.9 Comparison of correlation results for Heavy and Light Dynamic Probing

hy -1 ? how?

For this comparison only linear correlation with intercept has been considered which ~~has~~ been accepted as the representative of the general behaviour of South Limburg area ~~soils for the~~ dynamic and SCPT tests results.

Table 5.11: Comparison of the Heavy and Light Dynamic Probing test Correlations

Soil Type	All data		Filtered data	
	Correlation Coefficient (R)		Correlation Coefficient (R)	
	Heavy Dynamic	Light Dynamic	Heavy Dynamic	Light Dynamic
Loess	0.63	0.24	0.55	0.29

From this comparison we find that the data obtained by Heavy Dynamic Probing produces better correlation results as compared to the data of Light Dynamic Probing.

So it can be said in view of the results achieved that for the soils of South Limburg area, Heavy equipment of Dynamic Probing is more suitable than the Light equipment.

It is not known if the Light Dynamic Probing is more often carried out in poorly accessible places, e.g. behind existing buildings. These places may have a higher likelyhood of disturbed soil layers due to human activities.

6. Data analysis and discussion of the results

6.1 Comparison of the correlation results with literature values

The literature review showed, only very few correlations have been made in the past between SCPT and Dynamic Probing test results. It can be seen that data of South Limburg indicates linear trend and the linear correlation with intercept gives better coefficient of correlation than power correlation.

No correlations have been found from literature for Loess, for comparison with the results of this study and only one example presented by Bela Kralik (1984) in Hungary for cohesionless soils can be considered for comparison with Sand and Gravel correlation results of South Limburg (table 6.1).

The following relation between the number of blows N_{20} in Heavy Dynamic Probing and the cone resistance qc (MN/m^2) is suggested by Bela Kralik (1984b).

For cohesionless soils:

$$q_c = 0.476\ N_{20} + 1.095$$

Table 6.1 Comparison of the results of Sand and Gravel with Kralik results from literature.

Bela Kralik(1984) Hungary	q_c=0.476N_{20}+1.095
South Limburg research study(All Sand/Gravel)	q_c=0.5318N_{20}+14.639
South Limburg research study(Filtered Sand/Gravel)	q_c=0.4924 N_{20}+16.712

The relation found by Kralik between number of blows (N_{20}) in Heavy Dynamic Probing and the SCPT cone resistance (q_c) is compared in figure 6.1 with the relation found in this research study for Sand & Gravel. It can be seen that higher values will be obtained for (q_c) for the same values of (N_{20}) if we consider the results obtained by South Limburg data as compared to the relation proposed by Kralik.

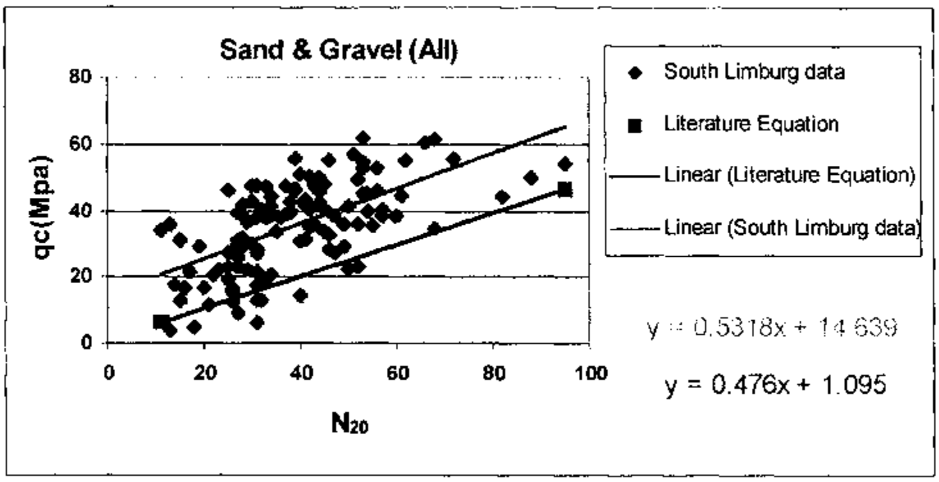


Figure 6.1: Comparison of the results of Sand and Gravel(All) with Kralik results from literature.

One of the main reasons can be that soils of South Limburg are densely packed, giving higher values of resistance. Another reason may be that we have compared the results of the mixture of Sand & Gravel with the results of cohesionless soils, which might be pure sand. As described earlier, very high q_c values may occur in Gravels due to the large sizes of particles compared to the size of the cone. There is also the possibility that the results of Kralik have been obtained from another version of Heavy Dynamic Probing (not according to German standard equipment).

6.2 Variability in soil

In this part of research, soil of each city has been analysed separately by obtaining mean and standard deviation of the data of all SCPT tests (Appendix-C). SCPT test results show a large difference between the soils as compared to Dynamic Probing test results. The main objective of this analysis was to see variability of the soil of South Limburg statistically, although it belongs to same geological unit. Besides it is considered to be useful if the q_c results of a certain type of soil can be reduced to two characteristic values: Mean and Standard deviation. These might be applied as criteria for automatic processing in a GIS. Comparison of the results for each soil type has been given in the tables 6.2 & 6.3.

Table 6.2 Comparison of Mean and Standard Deviation of SCPT for Loess in South Limburg.

City Name	Maastricht	Geleen	Nuth	Beek	Landgraaf	Heerlen	Kerkrade	Gulpen
Mean[MPa]	2.2	2.86	2.17	3.42	2.14	3.48	3.16	3.29
STDEV[MPa]	1.64	1.16	0.92	1.60	1.33	2.62	1.53	1.82
C.O.V	0.74	0.40	0.42	0.47	0.62	0.75	0.48	0.55

Table 6.3 Comparison of Mean and Standard Deviation of SCPT for Sand & Gravel in South Limburg

City Name	Maastricht	Nuth	Landgraaf	Heerlen	Kerkrade
Mean[MPa]	37.01	14.07	34.24	26.10	24.13
STDEV[MPa]	15.03	8.80	13.53	8.90	16.55
C.O.V	0.40	0.62	0.39	0.34	0.68

Coefficient of variation(C.O.V)= STDEV/Mean

Considering the coefficient of variation for Loess & Sand and Gravel, the following analyses have been done:

- Tentative soil classification
- Local correlations(trend/city)
- General trend

6.2.1 Tentative soil classification

It can be seen from table 6.2 that Loess of Maastricht, Heerlen, Gulpen and Landgraaf show high value of C.O.V. Geleen, Nuth, Beek and Kerkrade results indicate low value of C.O.V. So we can say that although Loess analysed in this research lies in the same geological unit we have to expect variability. Loess is aeolian deposit; therefore grain size should be nearly homogeneous. Variability in density caused by human activities and erosion is possible. Therefore, it can be that reworked Loess shows high C.O.V as compared to parent Loess. C.O.V has also been calculated for Maastricht &

Beek data on the basis of N_{20}(DPH) but low values have been obtained (Appendix-C) as compared to results obtained by SCPT data.

Considering the values of coefficient of variation from table 6.2, we can tentatively classify Loess as follows:

Table 6.4 Tentative Soil Classification for Loess on the basis of Mean and Standard Deviation

City Name	Range of C.O.V	Soil Classification
Maastricht, Landgraaf, Heerlen,Gulpen	0.55-0.75	Reworked sandy Loess, densely packed
Geleen, Beek, Kerkrade, Nuth	0.4-0.48	Parent Loess,Very densely packed

Similarly it can be seen from the results of table 6.3 for Sand & Gravel that Maastricht, Heerlen and Landgraaf show low C.O.V while the results of Nuth and Kerkrade indicate high values of C. O.V. So we can say that variability in density and grain size of sand and gravel is possible.

Considering the coefficient of variation in Sand and Gravel from table 6.3, we can also tentatively classify Sand and Gravel as follows:

Table 6.5: Tentative Soil Classification for Sand and Gravel on the basis of Mean and Standard Deviation

City Name	Range of C.O.V	Soil Classification
Maastricht, Heerlen, Landgraaf	0.34-0.40	Sand less Gravely or Gravel less sandy, very densely packed
Nuth, Kerkrade	0.62-0.68	Sand more Gravely or Gravel more sandy, Medium dense

6.2.2 Local correlations (trend/city)

After finding the variability of soil from one place to another, the South Limburg soils were also analysed to see if they show local trend (trend/city) for correlations. Results have been presented in (Appendix-D) and have also been summarised in the table 6.6 and 6.7. As already mentioned in chapter 5 that linear correlations with intercept gave the best results, so linear correlation with intercept has been chosen here. It can be seen from table 6.7 that there is no correlation between q_c and N_{20} in the data from Landgraaf.

Table 6.6 Correlation results of SCPT and DPH (Trend/city)

City Name	Soil Type	Linear correlation equation	Correlation Coefficient(R)
Maastricht	Loess	q_c=0.1262N_{20}+1.6228	0.36
Maastricht	Sand/Gravel	q_c=0.4965N_{20}+16.467	0.6
Gulpen	Loess	q_c=0.1585N_{20}+1.2906	0.66
Beek	Loess	q_c=0.3102N_{20}+0.7066	0.84

Table 6.7 Correlations results of SCPT and DPL (Trend/city)

City Name	Soil Type	Linear correlation equation	Correlation Coefficient(R)
Landgraaf	Loess	q_c=0.028N_{20}+2.0079	0.09
Heerlen	Loess	q_c=0.0766N_{20}+1.6472	0.5
Kerkrade	Loess	q_c=0.0655N_{20}+1.1417	0.76
Nuth	Loess	q_c=0.0249N_{20}+0.9652	0.68

6.2.3 General trend

From Maastricht to NNE (Beek) and then to SE (Gulpen) a trend has been found. The trend from Sittard to the NE at (Kerkrade) has also been examined but did not give good results. These correlation trends have been found in certain specified direction in view of the availability of the data for specific cities. Results have been presented in (Appendix-D) and have also been summarised in the table 6.8 and 6.9

Table 6.8 Correlation results of SCPT and DPH (General trend)

City Name	Soil Type	Linear correlation equation	Correlation Coefficient(R)
Maastricht-Beek(NNE)	Loess	q_c=0.2383N_{20}+1.104	0.66
Maastricht-Gulpen(SE)	Loess	q_c=0.1437N_{20}+1.4986	0.57

Table 6.9 Correlations results of SCPT and DPL (General trend)

City Name	Soil Type	Linear correlation equation	Correlation Coefficient(R)
Sittard-Kerkrade(SE)	Loess	$q_c=0.023N_{20}+2.2341$	0.24

6.3 Variability with depth

Loess is classified as one soil type usually on the basis of grain size. After finding variability from one location to another, the variability in Loess and Sand & Gravel layer is examined with depth. The data at each 0.8m depth interval (starting upwards from the boundary of Loess and Sand & Gravel) has been plotted with different colour (Appendix-D, figure 12). The whole data is clustered in two groups shown in figure 6.2 below.

So we can recognise GroupI Loess with low values of q_c and N_{20} in the depth range of 0-5.6m. Low values may be due to less carbonate content or probably Loess is more organic in this range. Group II Loess shows increasing trend of values of q_c and N_{20} in the depth range of 5.6m-9.6m. There may be possibility of increasing carbonate content in this zone or probably high values are due to the fact that it is more sandy. Possibly this Loess may be older.

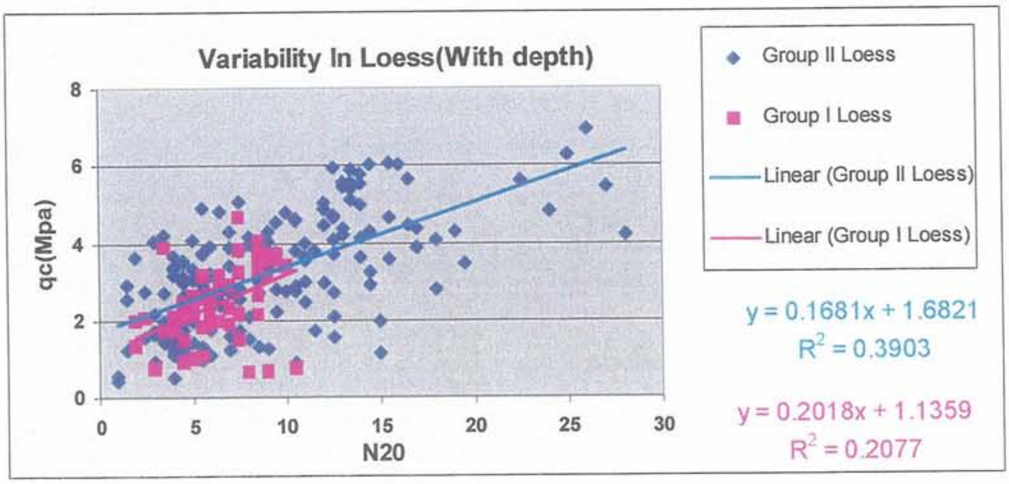


Figure 6.2 : Variability in Loess with depth(SCPT Vs DPH)

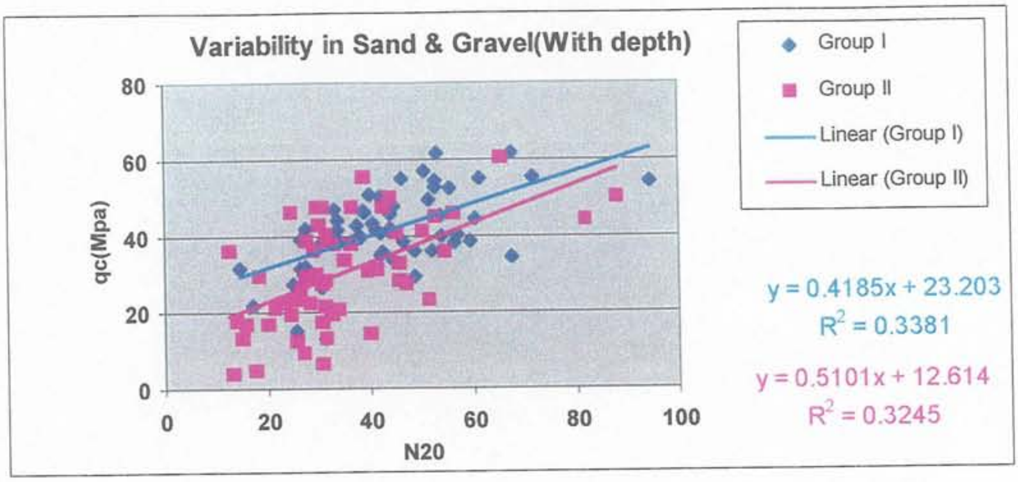


Figure 6.3 : Variability in Sand & Gravel with depth (SCPT Vs DPH)

Similarly Sand and gravel data at 0.8m interval has been plotted with different colours (Appendix-D, figure 13). Two groups can be recognised figure 6.3 above. Within Sand and gravel at depth of 1.6m-3.2m GroupI shows high values of q_c and N_{20}. The high values may probably indicate more gravely sand. At first 0.8m and from 3.2m-5.6m, Group II data indicates low values. There may be possibility that within this depth range the soil is more sandy with a smaller amount of gravel.

7. Comparison of Input Energy and theoretically Predicted Energy

The empirical correlations based on simple statistics suggest there must be a physical relationship between the two tests. The input energies of SCPT and Dynamic Probing calculated on the basis of test results have been compared with the theoretically predicted energy based on pile Bearing capacity formulae (ultimately both tests are used for pile foundation design).

The energy form.

For Dynamic Probing, driving work is defined as (Ruiter, 1988):

$$R_d = \frac{Mgh}{A} \text{ x } \frac{N}{D' \text{x} 1000} \quad \text{(kPa)} \quad \ldots\ldots\ldots\ldots(7.1)$$

where

M	is the mass of the hammer (kg)
h	is the height of fall of hammer (m)
A	is the base area of the cone (m^2)
g	is the acceleration of gravity (10 m/sec^2)
D'	is the defined depth interval of penetration (m)
N	is the number of blows per defined depth interval of penetration

same unit?

Input energy to move the Dynamic Probe cone to the depth interval of 0.4m can be calculated as follows (personal communications, W.Zigterman 2000):

Input energy (I-E) = R_d x A x 0.4 (kN-m)........................(7.2)

For SCPT, input energy to move the cone to the depth interval of 0.4m can be calculated as follows:

Input energy = I-E(SCPT) = q_c x A_c x 0.4 x 1000 (kN-m)........................(7.3)

why x1000?

A_c is the base area of the cone (m^2)

q_c is cone resistance (MPa)

Two approaches have been made for comparison of input energy and theoretical energy based on pile formulae.

- Approach for static loading
- Approach for dynamic loading

Calculations and results of input and theoretical energy have been given in Appendix-E. Calculations have been done for Loess and Sand & Gravel separately. For energy calculations, tests from projects No P-2522, P-2413 and P-2411 have been used. Summary of calculations has been presented in table E(A), Appendix-E.

7.1 Approach for static loading

In this part of the research, the input energy of SCPT has been compared with the theoretical energy based on pile formula using cohesion and friction angle of soil.

Point resistance of the cone can be found as a variation of the formula for base plate foundation = $Q_p = A_c[\, cN_c + \gamma B/2 \text{ x } N_\gamma + \gamma d N_q]$ (kN).........................(7.4)

The term ($\gamma B/2$ x N_γ) is very small as compared to ($\gamma d N_q$) for deep foundations(so also for a cone), so above equation can be simplified as follows:

$Q_p = A_c[\, cN_c + \gamma d N_q]$ (kN)...........................(7.5)

A_c is the base area of the cone (m^2)

c is cohesion of the soil (kPa)

γd is vertical stress (kPa)

N_c,N_q are the bearing capacity factors (for piles their values differ from those for a base plate foundation).

$N_c = 9$ (for cohesive soils)

Thus we can find theoretically, the energy required to move the point to a certain depth interval, 0.4m in this case (personal communications, W. Zigterman 2000).

Energy according to pile formula will be

$$E_{.static} = Q_p \times 0.4 \quad (kN\text{-}m) \ldots\ldots\ldots\ldots\ldots\ldots (7.6)$$

?

The location of failure surface for deep foundations is less well known and depending on the failure surface assumed, various authors have proposed different values of N_q as a function of friction angle.

Considering the wide range of values for N_q, two of the N_q curves have been used. The low values have been proposed by Berezantsev and high values by Meyerhof (Lambe & Whitman, 1979 figure 33.4). Both curves give N_q values that are very sensitive to variation of the friction angle.

The calculations of theoretical energy based on pile formula (equation 7.6) for SCPT test from Project (P-2411) in Loess using Berezantsev and Meyerhof Nq values have been given in tables E1 & E2 respectively (Appendix-E). Calculations of input energy for the same SCPT test based on (equation 7.3) is also given in tables E1 & E2. Results have been represented in figure E1.

Similarly calculations of input energy & theoretical energy for SCPT test from project (P-2413) in Loess have been given in tables E3 & E4 respectively. Results have been represented in figure E2.

For Sand and Gravel, calculations based on two different friction angles i.e., Ø=35° and Ø=40° have been done. Calculations of theoretical energy based on (equation 7.6) for SCPT test from project (P-2522) in Sand and Gravel using Berezantsev and

Meyerhof N_q values for Ø=35° have been given in tables E5 & E6 respectively. Calculations of input energy for the same SCPT test based on (equation 7.3) is also given in tables E5 & E6. Results have been represented in figure E3.

Similarly calculations for Ø=40° using Berezantsev and Meyerhof N_q values have been given in tables E7 & E8 respectively. Results have been represented in figure-E4.

7.2 Approach for dynamic loading

In the second approach, comparison of input energy of Dynamic Probing (based on test results) has been made with the theoretical energy based on dynamic pile formula. Most commonly used dynamic pile formula, known as the Engineering News formula has been used. Dynamic pile formulae are widely used to determine the static capacity of a pile, although they produce a large variation in the results.

Engineering News formula (Lambe & Whitman 1979):

$$R = \frac{166.64E}{s + 2.54} \quad \text{..........................(7.7a)}$$

E mgh x 1/1000 ....(7.7b)

E is the energy per blow in (kN-m)

m mass of hammer (kg)

g acceleration due to gravity (m/s^2)

h height of fall of hammer (m)

R is the allowable pile load (kN)

[High values of factor of safety (upto 5) have been considered in equation (7.7a)]

s is the average penetration in (mm) per blow

So energy required for penetration (failure state) according to dynamic pile formula will be

E-Dynamic = R * 0.4 * F (kN-m)...........................(7.8)

F = Safety factor between allowable load and failure state, value unknown(F=1 has been used in the calculations).

The calculations of input energy based on (equation 7.2) and theoretical energy according to dynamic pile formula (equation 7.8) for Heavy and Light Dynamic Probing in Loess have been given in tables E9 and E10 respectively. The tests from projects (P-2411 & P-2413) for Heavy and Light Dynamic Probing respectively, have been used in calculation. The results have been presented in figures E5 and E6 respectively.

Similarly, calculations of input & theoretical energy for Heavy Dynamic Probing in Sand and Gravel based on test from project (P-2522) have been given in table E11. The results have been presented in figure E7.

Comparison of input energy of (SCPT Vs DPH) and (SCPT Vs DPL) in Loess, based on equations 7.2 and 7.3 has been given in figures E8 and E9 respectively. Summary of calculations has been given in the table E(A) (Appendix-E).

7.3 Discussion of results

It can be seen from figures E1 to E4 that energy based on static pile formula, increases linearly with depth. Whereas the input energy to push the cone, based on test results shows non linear behaviour. There seems no relationship between the two types of energies.

Reason is that theory is based on elastic half space which is homogeneous while nature is never homogeneous. There is large variation in friction, depending upon whether soil is more dense or not.

It can be seen from figures E1 and E2 that somewhere input energy is less than the theoretically required energy to push the cone to the required depth interval. Figure E3 indicates that input energy curve of SCPT based on (equation 7.3) shows larger amount of energy than the energy calculated on the basis of pile formula (equation 7.6).

Similarly figure E4 shows the same SCPT input energy curve as in Figure E3 but the theoretical energy based on pile formula has been calculated for Ø=40° in Sand and Gravel. It can be seen that amount of energy based on pile formula increases considerably with change in friction angle. Reason is that bearing capacity factor N_q=900, has been used according to Meyerhof. Due to very high values of N_q proposed by Meyerhof, input energy in figure E4 is less than energy required theoretically to push the cone.

So we can say that difference in calculation results based on idealised logspiral curve and actual input energy (based on test results) is due to the fact that theory is based on assumed failure surfaces while in reality soil can fail through weakest layers and not according to elastic half space theory.

why are these figures not here?

It can be seen from figure E5 that energy curve based on dynamic pile formula (equation 7.8) is very similar in shape to the input energy curve of Heavy Dynamic Probing based on (equation 7.2), in Loess. The difference is only in the amount of energy. The low values of energy according to Engineering News formula can be explained on the basis of high Factor of safety (2-5) that should be considered in the (equation 7.8).

From figure E6, it is clear that no specific relation can be established between the energy curve based on Engineering News formula (equation 7.8) and Light Dynamic Probing input energy curve. It seems that results of Light Dynamic Probing are rather erratic.
Similarly it can be seen from figure E7 that theoretical energy curve based on (equation 7.8) shows somewhat similarity with the Heavy Dynamic Probing input energy curve based on (equation 7.2) in Sand and Gravel. This can be due to sensitivity of "s" in the formula.

It can be seen from figure E8 that input energy curve of SCPT based on (equation 7.3) and Heavy Dynamic Probing energy curve based on (equation 7.2), in Loess shows nearly same shape but amount of dynamic energy is high. Perhaps last reading of Dynamic Probing is in next layer (Sand & Gravel) when SCPT is stopped to prevent damage.

Figure E9 shows totally different curves for input energy of SCPT and Light Dynamic Probing test in the Loess. Light Dynamic Probing results are more variable and less reliable than Heavy Dynamic Probing.

So we can say that similarity of input energy curves of Heavy Dynamic Probing with input energy curves of SCPT and with energy curve based on Engineering News formula, also confirms the reliability of the results of Heavy Dynamic Probing & better correlation obtained between SCPT and DPH.

Generally it has been found that Heavy Dynamic Probing results are more reliable than Light Dynamic Probing results in the soil of South Limburg. This can also be confirmed from the energy curves of both equipments. The input energy curve of DPL does not show any clear relation with the input energy curve of SCPT (Figure E9) and also with the theoretical energy curve based on Engineering News formula (Figure E6). This may also confirm the poor correlations between SCPT and Light Dynamic Probing.

8. Conclusions and recommendations

8.1 Conclusions

The main conclusions that can be drawn from this research have been summarised as follows:

- For South Limburg soils, which are mainly densely packed Loess and Sand & Gravel, the use of Light Dynamic Probing is not a good choice. Maximum depth explored by this equipment is nearly 6m.

- The use of mechanical cone penetrometer for SCPT tests and Heavy Dynamic Probing is appropriate for most foundation investigations.

- In this research study a procedural method has been developed for the comparison between SCPT & Dynamic Probing test results based on South Limburg soils.

- It has been found that both tests indicate linear correlation and best coefficients of correlation have been achieved from linear correlation with intercept for South Limburg soils. Generally it has been found that results obtained by Light Dynamic Probing are rather erratic. Low coefficient of correlation has been obtained for the results of Light Dynamic Probing as compared to the Heavy Dynamic Probing. The results can be summarised as follows:

All data analysis (Heavy Dynamic Probing versus SCPT) for linear correlation with intercept.

Soil Type	Linear Regression y=ax+b	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	q_c=0.1803N+1.4031	0.3966	0.63
Sand/Gravel	q_c=0.5318N+14.639	0.3786	0.61

All data analysis (Light Dynamic Probing versus SCPT) for linear correlation with intercept

Soil Type	Linear Regression y=ax+b	Correlation Coefficient (R^2)	Correlation Coefficient (R)
Loess	q_c=0.023N+2.2341	0.0593	0.24

- In this research study, the statistical correlations have been verified, on the basis of energy relationships with limited results.

  - The input energy of SCPT & Dynamic Probing has been compared with the theoretical energy based on pile formulae.

  - The input energy of SCPT shows almost no relation with the theoretical energy. In some cases input energy is less than the theoretically required energy to push the cone to the required depth interval. The possible explanation of this deviation can be that the theory is based on the elastic half space which is homogeneous. In reality there is large variation in friction, depending upon whether soil is dense or not.

  - Also theory is based on assumed failure surfaces while in reality soil can fail easily through weakest layers and not according to elastic half space theory.

  - It has been found that bearing capacity factors are very sensitive to friction angle. With little increase in friction angle, theoretical energy increases considerably.

  - The input energy curve of Heavy Dynamic Probing shows almost same shape as the theoretical energy curve based on Engineering News formula. Whereas the input energy curve of Light Dynamic Probing does not indicate specific relation with theoretical energy.

  - For Loess, however, the input energy curve of DPH is very similar to the input energy curve of SCPT, while there is no similarity in the input energy curves of DPL & SCPT.

- On the basis of comparison of the energy of the test equipment one can conclude that DPL energy curve shows deviation from theoretical energy as well as from SCPT energy curve. This suggests that the results obtained by Light Dynamic Probing are not as reliable as the results of Heavy Dynamic Probing. This may also be the reason of poor correlation between the results of SCPT and Light Dynamic Probing.

- In this research study, the soil of South Limburg has been classified statistically, on the basis of coefficient of variation, by taking the Mean & Standard deviation of SCPT test results.

Tentative Soil Classification for Loess on the basis of Mean and Standard Deviation

Range of C.O.V	Soil Classification
0.55-0.75	Reworked sandy Loess, densely packed
0.4-0.48	Parent Loess, Very densely packed

Tentative Soil Classification for Sand and Gravel on the basis of Mean and Standard Deviation

Range of C.O.V	Soil Classification
0.34-0.40	Sand less Gravely or Gravel less sandy, very densely packed
0.62-0.68	Sand more Gravely or Gravel more sandy, Medium dense

- It has been found that soils of South Limburg also show local trend. Correlations/city have been made (see tables 6.6 & 6.7).

- Also general trend based on the available data has been found (see tables 6.8 & 6.9).

- Variability with depth has also been examined. Loess has been classified into two groups. Similarly two groups can be recognised in Sand & Gravel with depth. Correlations for each group with depth have also been made.

8.2 Recommendations

The present study did not have within its scope an examination of the differences of foundation design based on SCPT and Dynamic Probing as ultimately these tests are used for the design of foundations including pile bearing design. A comparative study should be carried out to see if the methods developed for the SCPT (i.e. Koppejan method) and that for the Dynamic Probing produce similar results.

References

1. Aas, G., Lunne, T. and Hoeg K. 1986. Use of In-Situ Tests for Foundation Design in Clay. Proc. of In-Situ ' 86., Geotechnical Special Publication No 6, ASCE, Blacksburg, pp 1-30.

2. Akca, Necla, July 1999. Correlation of SPT-CPT Data From The United Arab Emirates. M.Sc Thesis, Engineering Geology ITC Delft. The Netherlands.

3. Begemann, H.K.S., 1953. Improved method of determining resistance to adhesion by sounding through a loose sleeve placed behind the cone. Proceedings of the third International Conference on Soil Mechanics and Foundation Engineering, Zurich.

4. Bell, F.G. 1987. Ground Engineer's Reference Book. Butterworths, 59 chapter.

5. Bekendam, R.F.,1998. Pillar Stability and Large-Scale Collapse of Abandoned Room and Pillar Limestone Mines in South Limburg, The Netherlands.

6. BS 5930: 1981. Code of practice for Site Investigations. British Standards Institution.

7. Broms, B.B. and Flodin, N. (1988) " History of soil penetration testing". Proceedings of the International Symposium on Penetration Testing, ISOPT-1, Orlando, 1, 157-220, Balkema Pub., Rotterdam.

8. BUTCHER, A.P., McELMEEL, K. and POWELL, J.J.M. (1996). Building Research Establishment, Watford, Herts, UK. Advances in site investigation practice. Thomas Telford, London.

9. Clayton, C.R.I., Matthews, M.C., and Simons, N.E. 1995. Site Investigation. (2nd edition), Blackwell Science, Cambridge, USA.

10. Douwes Dekker, ir. D.M. July 1991. Soil Mechanics II, M.Sc course in Engineering Geology ITC Delft. Delft University of Technology, Faculty of Mining and Petroleum Engineering.

11. Desai, M.D., et al, (1974). Importance of adequate field inv. tests. proc. Symposium on earth and rockfill dams, INSSMFE, Kurukchhetra, 1974, vol. II, 427.

12. Interpretation of Results of Dynamic Penetration Tests. Group discussions. Proceedings of the European Symposium On Penetration Testing In Stockholm , June 5-7, 1974. Volume 2.

13. Jamiolkowski, M., Lancellotta, R., Tordella, L. and Battaglio, M., 1982. Undrained Strength from CPT. Proc. 2nd European Symp. on Penetration Testing. Amsterdam, Vol 2, p. 599-606.

14. Kralik, B. (1984b). Neuere Erkenntnisse bei Sondierungen, Proc. 9th Budapest Conference on Soil Mech. and Found. Eng. 167-174.

15. Kronieger, R.R. (1990). " Geotechnical uniformity of the Weichselien Loess sequence in South Limburg, the Netherlands". 6th international IAEG Congress, 1990 Balkema, Rotterdam.

16. Lambe, T.william, Robert V. Whitman,1979. Soil Mechanics, SI Version. Massachusetts Institute of Technology, With the assistance of University of Sydney. John Wiley & Sons, New York, USA.

17. Lunne, T., P.K. Robertson and J.J.M. Powell, 1997, Cone Penetration Testing in Geotechnical Practice, Blackie Academic and professional, London.

18. Maurenbrecher, P.Michiel, Rene Kronieger & Prof David Price, August 1990. Excursion Guide, Geology of the Netherlands. 6th Congress of the

International Association of Engineering Geology, Amsterdam-The Netherlands.

19. Marcu, A. and Culita, C. (1995). State-Of-Practice on CPT in Romania (National Report). Proceedings CPT' 1995.

20. Price, Prof D.G. September 1991. Properties of Some " Problem" Soils. Site Investigation. Faculty of Applied Earth Sciences. Section Engineering Geology. Delft University of Technology. The Netherlands.

21. Plasman, S.J. and Kronieger, R.R. (1990). "An engineering geological classification of the Loess deposits in the Netherlands". 6th international IAEG Congress, 1990 Balkema, Rotterdam.

22. Ruiter, J.De 1988: Dynamic Probing (DP): International reference test procedure. Proceedings of the First International Symposium on Peneration Testing/ ISOPT-1/ ORLANDO/20-24 March 1988, A.A.Balkema/ Rotterdam/Brookfield/1988.

23. Swan, A.R.H. and Sandilands, m., 1995. Introduction to Geological Data analysis, Blackwell Science.

24. Sanglerat, G. 1972. The Penetrometer and Soil Exploration; Interpretation of penetration diagrams - Theory and practice. Elsvier Publishing Company, Amsterdam.

25. Thomas, D. 1968. Deep-Sounding Test Results and the Settlement of Spread Footings on Normally Consolidated Sand. Geotechnique, Vol 8, No.4 p. 472-488.

26. Van Wambeke, A. 1982. The Dynamic Penetration Test. Amici et Alumni, Em. Prof. Dr Ir E.E. De Beer.

27. Vlasblom, A. (1985) " The electrical penetrometer; a historical account of its development" LGM Mededelingen, Part XXII, 92 pp.

28. Ziegler, P.A. (1982) Geological atlas of Western and central Europe, Shell Internationale Petroleum Maatschappij B.V., 130 pp.

29. Zijlstra, J.J.P. (1994) Sedimentology of the Late Cretaceous and Early Tertiary (Tuffaceous) Chalk of Northwest Europe, Geologica Ultrajectina, 119, University of Utrecht (PhD-thesis), 192pp.