

VERTICAL LOAD CAPACITY AND QUALITY ASSESSMENTS OF
BORED PILES

by

Çağdaş Arda

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ABSTRACT

VERTICAL LOAD CAPACITY AND QUALITY ASSESSMENTS OF BORED PILES

There are several methods used in the design of bored piles. To start with, mathematical modeling of the pile and soil parameters provide various formulas and theories for ultimate bearing capacity and settlement calculations of a pile, which are used in pile design. However, such model based analyses seldom give fully reliable results. Many factors, including construction methods and real site conditions also play an important role in the estimation of vertical load capacity. As a result, although economically costly and time-consuming, pile load tests are the most dependable way of measuring the load bearing capacity and displacement behavior of piles. Besides, integrity tests are the most preferable option to check the quality of pile upon installation. Various deep foundation codes have been determined to minimize the risk of foundation failures. These codes recommend higher safety factors for theoretical capacity calculation methods which have a large amount of assumptions and unknowns, whereas lower safety factors for methods which present more detailed and reliable results. However, latter methods are usually more costly and need qualified workmanship. Therefore, for every piling project a cost analysis should be performed which compares the benefit of a method with its disadvantage to decide for the most economic pile design.

The main purpose of this thesis is to compare the allowable capacity and displacement values of several load test results with each other and also with theoretical capacity calculations conducted on bored piles in three different case studies consisting of rock sockets in order to decide the most preferable and cost effective design option for each case. The effect of correlations and differences between test results to the design will be investigated, to compare the reliability of a displacement based design with a capacity based design. Besides, integrity test results will also be analysed to underline the effect of construction quality to bored pile capacity.

ÖZET

FORE KAZIKLARIN DÜŞEY YÜK TAŞIMA KAPASİTELERİNİN VE NİTELİKLERİNİN DEĞERLENDİRİLMESİ

Fore kazık dizaynı için kullanılan birçok yöntem bulunmaktadır. Başlangıç olarak, kazık ve zemin parametrelerini matematiksel modelleyerek kazıkların maksimum yük taşıma kapasitelerinin ve oturmalarının hesaplanmasına olanak sağlayan birçok formül ve teori mevcuttur. Fakat, modellemeye dayanan bu tür hesaplamalar nadiren güvenilir sonuçlar verir. İnşaatta kullanılan metodların ve saha zemin koşulların etkisi gibi birçok faktör de kazıkların düşey yük kapasitelerinin oluşmasında etkilidir. Bu nedenle, pahalı olmalarına ve süre almalarına rağmen kazık yükleme deneyleri, kazıkların yük taşıma kapasitesi ve oturma davranışlarını belirlemede en çok güvenilen metodların başında gelmektedir. Ayrıca, inşaatı tamamlanan fore kazıkların kalitesinin ve niteliklerinin belirlenmesinde en çok tercih edilen metodların başında da integrity testleri gelmektedir. Temel göçme risklerinin önlenmesi için çeşitli mühendislik kodları oluşturulmuştur. Bu kodların önerdiği güvenlik faktörleri, birçok bilinmeyeninin olduğu ve kabullere dayanan teorik dizayn yöntemleri için oldukça yüksektir, fakat daha güvenilir metodlar için ise düşük değerler sunmaktadır. Yalnız şu da göz önünde bulundurulmalıdır ki bu güvenilir metodlar hem daha pahalıdır hem de kaliteli işçiliğe ihtiyaç duyarlar. Bu nedenle, her fore kazık projesinde en ekonomik dizaynı temini için mutlaka her bir metodun yararının dezavantajıyla karşılaştırılmasına dayanan fiyat analizi gerçekleştirilmelidir.

Bu tez çalışmasının ana amacı; taşıyıcı zemine soketlenmiş fore kazık dizaynı içeren üç farklı vaka analizinde uygulanan birçok kazık yükleme deneyinin maksimum yük taşıma kapasitesi ve yer değiştirme verilerinin hem birbirleriyle hem de teorik hesap metodlarıyla karşılaştırılması ile en uygun ve ekonomik dizayn seçeneğinin belirlenmesidir. Test sonuçlarının arasındaki bağlantıların ve farklılıkların dizayna etkisi, üst yapının performans kriterine bağlı bir yer değiştirme değerine dayanan dizaynı mı yoksa kazığın yük taşıma kapasitesinin teminine dayalı bir dizaynı mı daha güvenilir olduğunun ortaya çıkarılması amaçlı araştırılmıştır. Bununla beraber, inşaat kalitesinin

kazık yük taşıma kapasitesine etkisini belirlemeye yardımcı olan kazık niteliğini tespiti yarayan test sonuçları da karşılaştırılmıştır.

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LIST OF SYMBOLS / ABBREVIATIONS

a	Acceleration
A	Pile cross-sectional area
A_b	Cross-sectional area of the base
A_p	Area of the pile tip
A_s	Cross-sectional area of the shaft
AE_p	Equivalent axial pile stiffness
B	Pile width
c	Wave speed in pile material
c'	Cohesion of the soil supporting the pile tip
D	Width or diameter of pile
e	Strain
E_b	Deformation modulus of the soil beneath the pile base
E_d	The deformation modulus of the rock mass surrounding the shaft
E_p	Elastic modulus of pile
E_s	Modulus of elasticity of soil at or below the pile point
F	Force
f	Unit friction resistance at the depth increment
f_s	Ultimate bond stress
FS_p	Factor of safety for end bearing
FS_s	Factor of safety for frictional resistance
I_p	Influence factor related to the ratio of (L/R)
I_{wp}	Influence factor $\cong 0.85$
j	Mass factor
j_c	Case damping coefficient
k	Transfer function

$K.E.$	Kinetic energy
ΔL	Pile depth along which perimeter and unit friction resistance are constant
m	Ram mass
M_r	The ratio of the elastic modulus of the intact rock to its UCR
N_c^*, N_q^*, N_ϕ^*	Pile end-bearing capacity factors
p	Perimeter of the pile
$P.E.$	Potential energy
Q	Load
q'	Effective vertical stress at pile tip level
Q_{all}	Allowable load bearing capacity
q_c	The average unconfined compressive strength related to depth
Q_d	Design load capacity
$Q_{g(u)}$	Ultimate load bearing capacity of the group pile
Q_{nsf}	Drag down force due to negative skin friction
Q_o	Pile's actual dead weight which is calculated by considering water buoyancy effects
q_p	Unit point (end-bearing) resistance
Q_p	Ultimate end-bearing (point or tip) capacity
Q_s	Ultimate friction bearing capacity
Q_u	Ultimate load bearing capacity
q_u	Unconfined compression strength
q_{wp}	Point load per unit area at the pile point
Q_{wp}	Pile tip load at working load
Q_{ws}	Total friction resistance at working load
R	Total soil resistance
s_e	Elastic compression of pile
s_p	Settlement of pile due to the end-bearing of pile into the soil

s_s	Settlement of pile due to the frictional resistance build up
v	The ram velocity at impact
V	Velocity
V_{toe}	Velocity at pile toe
W_s, W_b	Loads on the pile shaft and base
Z	Pile impedance
α	A reduction factor relating to q_u
β	A correction factor related to the discontinuity spacing in the rock mass
δ_{cr}	Allowable displacement for serviceability limit state
$\delta_{f,max}$	Maximum differential displacement
$\delta_{h,max}$	Maximum horizontal displacement at the top of the structure
δ_v	Displacement under service load
Δ_{VWSG}	Change in strain gauge readings
η	Group efficiency
ϕ	Soil friction angle
θ	Inclination angle
ν	Poisson's ratio of the soil
ξ	A factor, zero to 0.5 or zero to 0.67 based on friction resistance distribution
ASCE	American Society of Civil Engineers
ASTM	American Society of Testing Materials
BGL	Below ground level
CHA	Cross hole analyser
CPT	Cone penetrometer testing
CSL	Cross hole sonic logging
DMD	Dubai Municipality Datum
ELR	Extra length ratio
EMX	Maximum transferred energy

FMAX	Maximum measured compression force
FMX	Peak force
GWL	Ground water level
PC	Personal computer
PDA	Pile driving analyser
PIT	Pile Integrity Tester
R_d	Dynamic component of the soil resistance
R_s	The static component of the soil resistance
RMX	Maximum static resistance
SPT	Standard penetrometer testing
SPT/N	Standard penetration test resistance
UCS	Unconfined compressive strength
$VT1*Z$	Velocity times pile impedance

1. INTRODUCTION

Bored piles, like other pile types, are structural members and so should be designed as a part of the whole structure. Therefore, all the methods used in bored pile design use design working loads applied to the pile downward from the upper structure. There are various, however the most common ones used in pile design are engineering analysis including static method of analysis, dynamic formulas using wave equation analysis, and field testing. Although the main object of these methods is to evaluate the ultimate bearing capacity of the pile, more emphasis has been put on to displacement analysis in engineering design recently due to the fact that the pile response to load is primarily a movement response and growing demand for construction quality force the geotechnical engineers to design a foundation system depending on the serviceability limit state, minimizing the displacements to an allowable value.

Static method of analysis used in determining ultimate capacity and displacement of the foundation system use both various theoretical correlations and soil parameters obtained by site investigation including SPT or CPT measurements, as well as load-transfer mechanism of the pile. Dynamic analysis includes wave equation analysis, depending on wave propagation theory on pile interior layers which considered to act like springs and based on computer modelling using damping factors. Field testing of the piles depend on observations rather than theoretical formulas, and the more improved the testing system is, the better and more realistic results are obtained by interpretation of test data. The formulas and correlations of vertical load bearing capacity of bored piles in different soil or rock types are presented in Chapter 2. Displacement behavior including load-transfer mechanism of bored piles are presented in Chapter 3. Chapter 5 includes all field testing methods, describing not only the equipments and installation methods but also the interpretation methods of test results based on standards developed by American Society for Testing and Materials (ASTM).

Construction of a bored pile is complex and the quality of construction plays a key role in the formation of its bearing capacity. The construction should be in accord with the pile design, however several circumstances may cause obstructions or delays in piling

schedule, which a designer should also take into account. Various construction techniques are present for bored piles depending on soil conditions and are all presented in Chapter 4.

Allowable pile capacities should be larger than design working service loads applied to pile and displacement values should be lower than serviceability limits for an acceptable design. Total ultimate capacities are divided to a factor of safety to determine allowable working loads, however these safety factors are in a very wide range, due to the fact that they are assigned to account for uncertainties from unknown loads or loading conditions, site variations, and inaccuracies in load determination methods. As a result, various codes were developed which assign safety factors for ultimate capacities; with higher values if the capacity assessment method include more assumptions and uncertainties, to minimize the risk of foundation failures. On the other hand, the high number and quality of the methods which allows the engineer to select a low safety factor to design more economical piles might be also costly and even may exceed the cost of the overall pile schedule designed by that safety factor. Therefore, the comparison of the cost estimate of pile design and method selected for the capacity and settlement assessment should also be taken into account.

Under all the information given in the first five chapters presented, in Chapter 6, three case studies were presented consisting of three different construction sites respectively which all situated in Dubai, United Arab Emirates. For each site, soil conditions and parameters were presented, though the sites were close to each other and had similar soil characteristics. Information about the general project and applied design loads were also included. Next, total cost for both piling design and pile testing was presented to inform about the percentage of the cost applied in design. Then, for all three sites, the allowable working load and settlement values were determined by static method of analysis, using the soil parameters given. The values calculated by the method were compared first with static load testing as in Case Study-1, with Osterberg load testing as in Case Study-2 and with a combination of static and dynamic load testing as in Case Study-3 for a selected single pile. The results were investigated to see the effect of the test type to determination of allowable vertical load bearing capacity and therefore to pile design by selecting different safety factor values for different load tests, and in the end cost analysis for all case studies were conducted to compare the benefits of load tests on pile design with their

installation costs. In addition, integrity assessment was conducted in Case Study-3 to investigate that quality determination of bored piles is also important since construction quality may effect pile capacity as well.

2. VERTICAL LOAD BEARING MECHANISMS OF BORED PILES

2.1. Introduction

Piles are structural members which are made of steel, concrete or timber and are used to transmit surface loads to lower levels in the soil mass. They are more deep and costly than shallow foundations. Piles are commonly used for following purposes (Figure 2.1) (Bowles, 1996):

- To carry the superstructure loads into or through a soil stratum vertically and laterally, when one or more upper soil layers are highly compressible and too weak to support the superstructure loads.
- To control settlements and prevent ground surface damage of the structure, when spread footings or a mat is on a marginal soil or is underlain by a highly compressible stratum.
- To prevent a bearing capacity failure caused by the liquefaction of an underlying soil deposit.
- To compact loose, cohesionless deposits and to resist uplift or overturning forces.
- To prevent damages caused by swelling and shrinkage of expansive and collapsible soils.

There are several methods used in the design and construction of bored piles:

- Engineering analysis — Based on the results of subsurface exploration and laboratory testing, the bearing capacity of the deep foundation can be calculated, like as shallow foundations. Called as static pile capacity analysis, the load transfer of surface loads to lower soil levels may be by vertical distribution of the load along the pile shaft, using a friction pile and/or a direct application of load to a lower stratum through the pile point, using an end-bearing pile.
- Field load tests — Prior to the construction of the foundation, a pile or pier could be load tested in the field to determine its carrying capacity. Because of the uncertainties in the design of piles based on engineering analyses, pile load tests are

common and may result in a more economical foundation than one based solely on engineering analyses.

- Dynamic formulas — Based on using the stress wave from the hammer impact in a finite-element analysis, using a driving equipment.

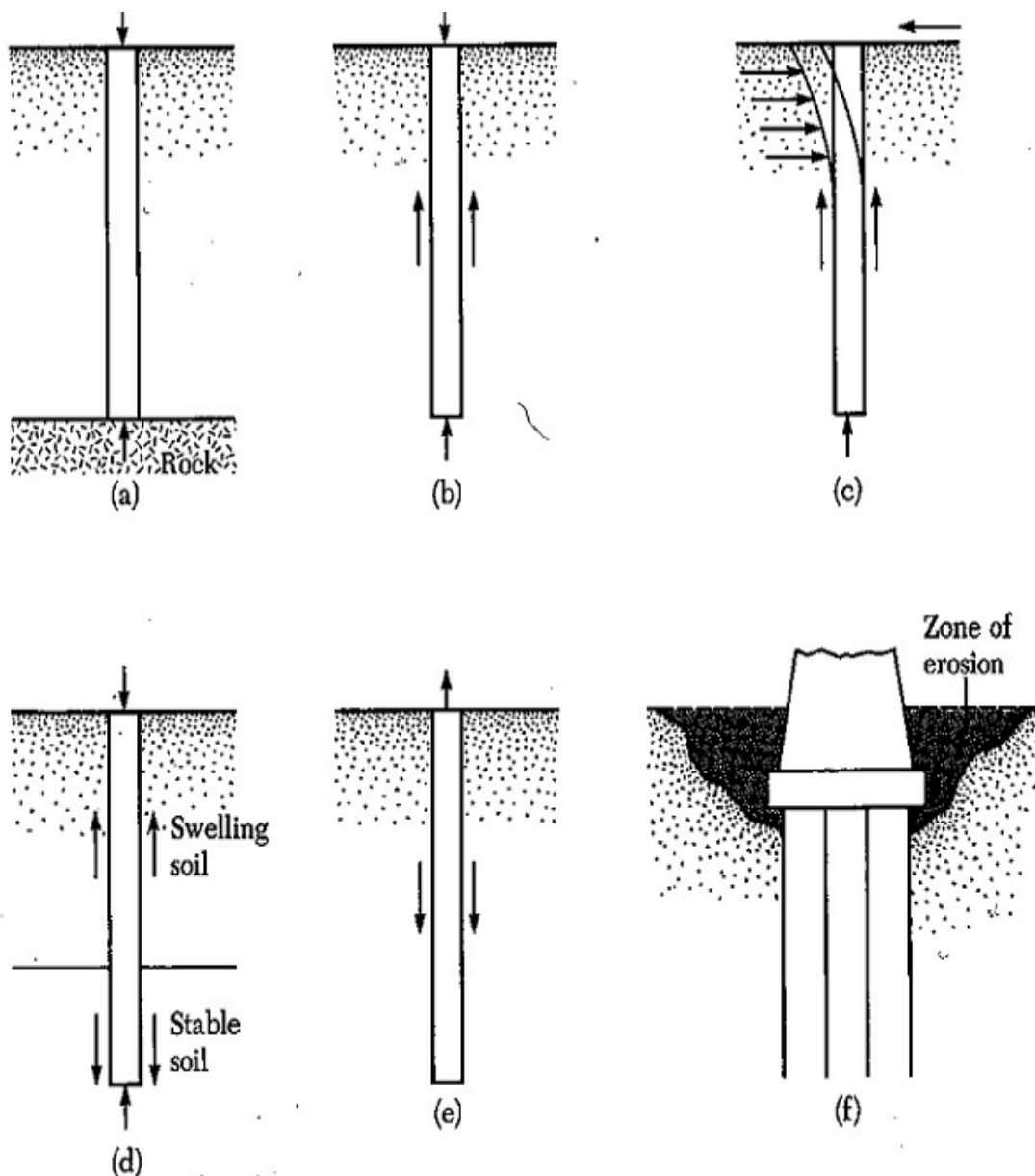


Figure 2.1. Conditions which require the use of pile foundations (Das, 2004)

Pile capacity determinations are difficult and although numerous investigations have been conducted in the past to predict the behavior and the load-bearing capacity of piles in granular and cohesive soils, the mechanisms are not yet entirely understood. A large

number of different equations are used, and seldom will any two give the same computed capacity. As a result, local experience in terms of what has worked best in the past for the local soil conditions may prove valuable in the design and construction of pile foundations.

2.2. Vertical Static Load Capacity Calculations of Bored Piles

A pile foundation can have no greater load-carrying capacity than that of the soils to which the load is transmitted, therefore the design involves two fields; first, selection and design of the piles and choice of driving equipment, and second, study of the soils to which the load is transmitted by the principles of soil mechanics. Piles do not, of themselves, basically alter the problem of foundation design, which is transmission of load from foundations to resisting subgrade; they are inserted merely to bypass inadequate strata and transmit the load to firmer material (Leonards, 1962).

Piles bear the superposed loads by undergoing deformations resulting on two distinct load bearing mechanisms as shown in Figure 2.2 (Candoğan, 2008):

- End-bearing (load carrying capacity of the pile point).
- Frictional resistance derived from the soil – pile interface.

So, load bearing equation of pile foundations can be written as (Das, 2004):

$$Q_u = Q_p + Q_s \quad (2.1)$$

where,

Q_u = Ultimate load bearing capacity

Q_p = Ultimate end-bearing (point or tip) capacity

Q_s = Ultimate friction bearing capacity.

Ultimate load bearing capacity value is mostly preferred, because usually piles carry the loads both in end-bearing and side shear.

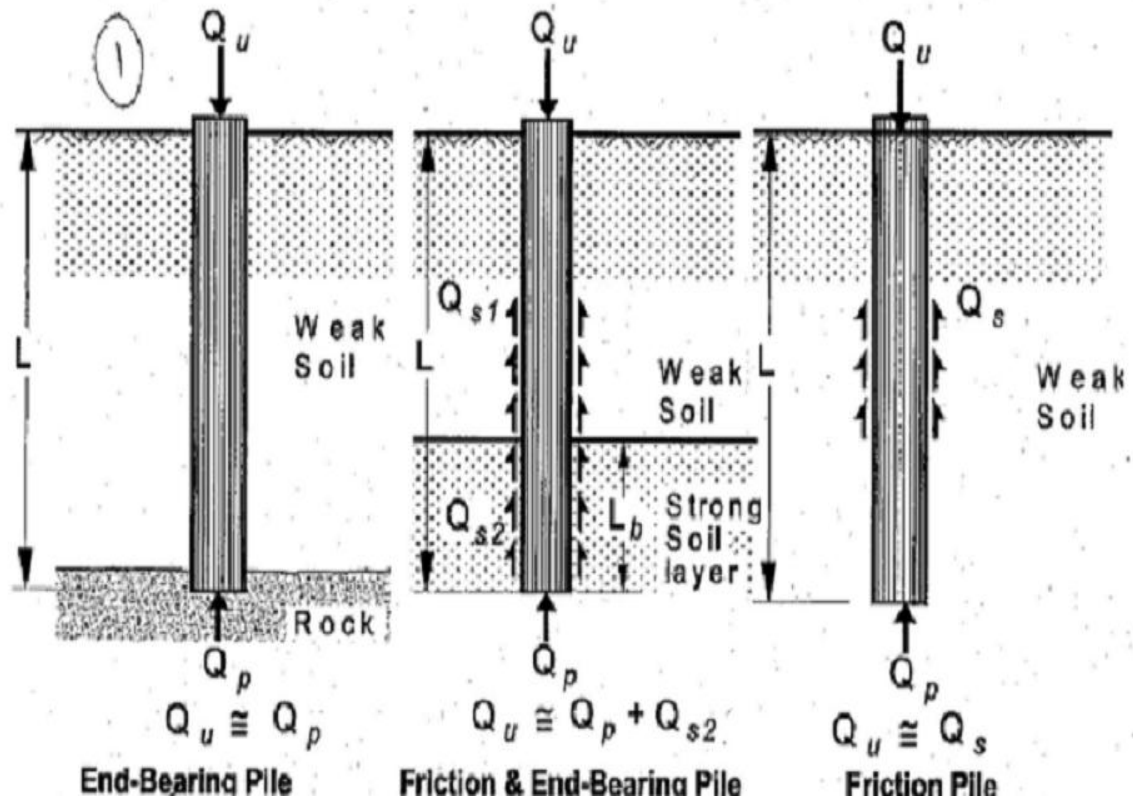


Figure 2.2. Categorization of piles on their load bearing mechanisms (Candoğan, 2008)

2.2.1. Point Bearing Piles and End Bearing Capacity

If soil-boring records establish the presence of bedrock or rock-like material at a site within a reasonable depth, piles can be extended to the rock surface as shown in Figure 2.4 (a). In this case, the ultimate capacity of the piles depends entirely on the load-bearing capacity of the underlying material; thus, the piles are called ‘point bearing piles’. In most of these cases, the necessary length of the pile can be fairly well established. The pressure distribution on the soil below the tip of a single pile may be assumed to take the form shown in Figure 2.3, as computed by means of the Boussinesq equation.

Although overlapping of bulbs of pressure from adjacent piles indicates that stress intensity is increased in that area, the only serious overlaps are those resulting in additive stresses capable of producing damaging deformations (Das, 2004 and Leonards, 1962).

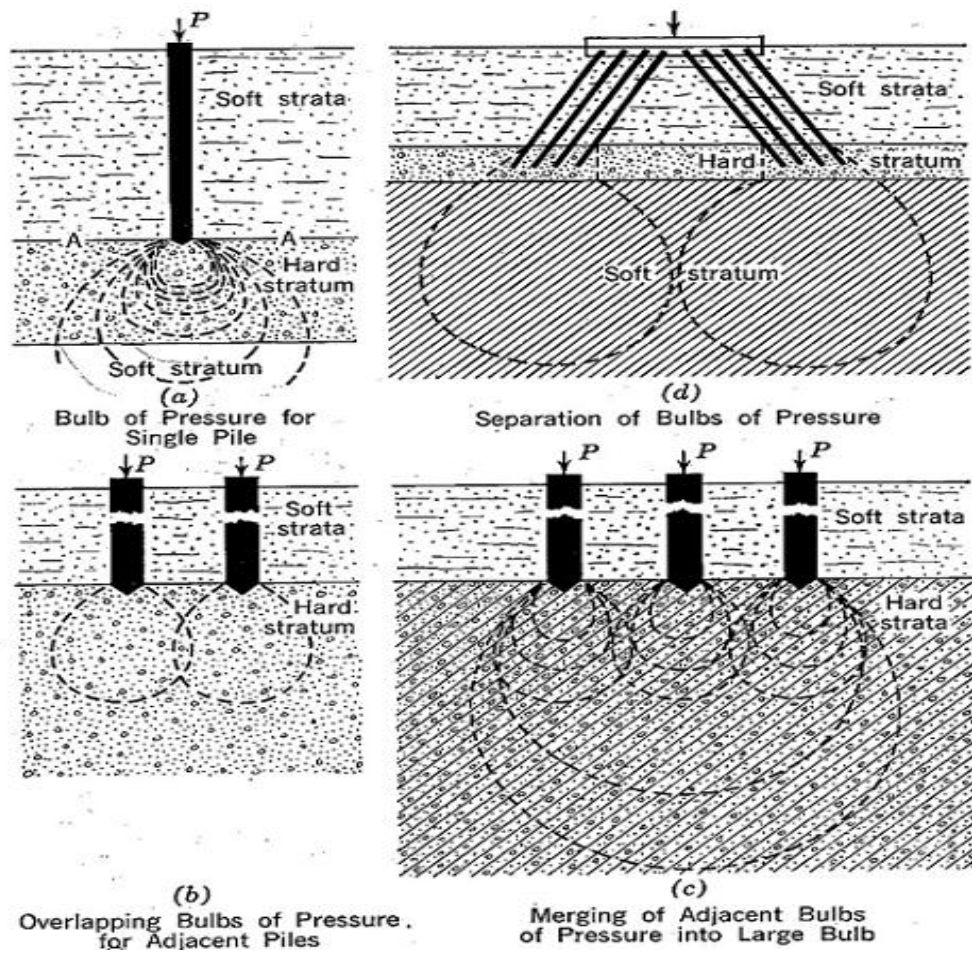


Figure 2.3. End bearing pile distributions in soil (Leonards, 1962)

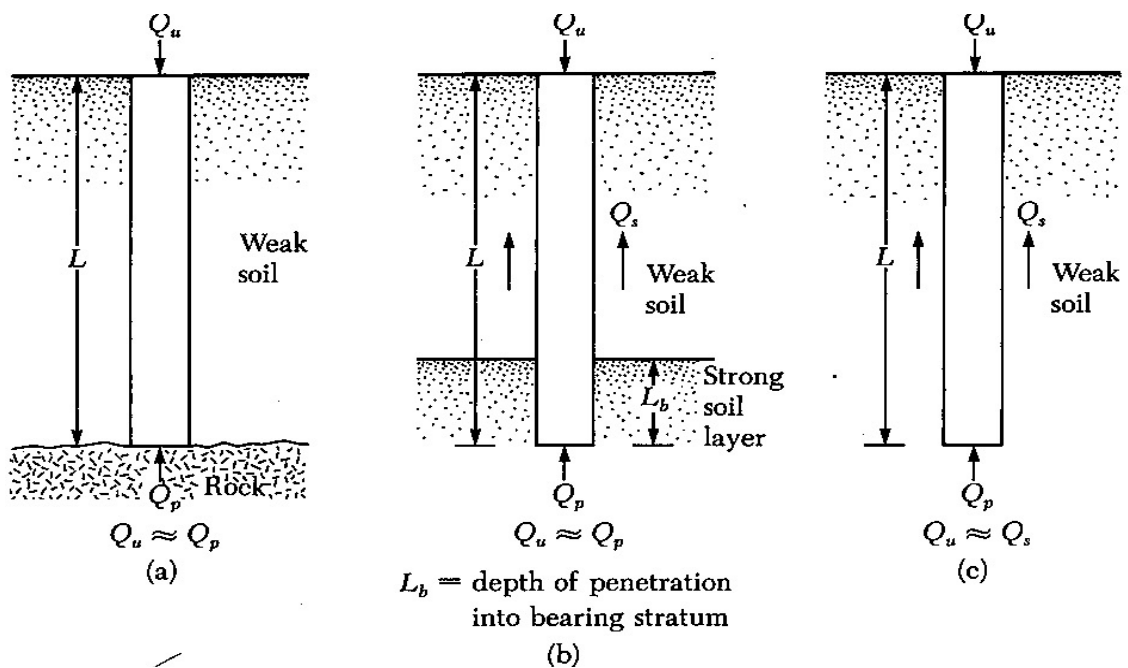


Figure 2.4(a) and (b). Point bearing piles, (c). Friction piles (Das, 2004)

The end bearing capacity is derived from Terzaghi's equation, and Meyerhof's general bearing capacity equation for vertical loading for shallow foundations is used, including a substitution of diameter instead of width, to get:

$$q_p = q'_c N_c^* + q'_v N_q^* + 0.5 \gamma B N_{\gamma}^* \quad (2.2)$$

However, as diameter is small we can neglect the third term and rewrite the equation:

$$q_p = q'_c N_c^* + q'_v N_q^* \quad (2.3)$$

This equation now gives us the end-bearing capacity of piles:

$$Q_p = A_p (q'_c N_c^* + q'_v N_q^*) \quad (2.4)$$

where,

A_p = area of the pile tip

q_p = unit point (end-bearing) resistance

c' = cohesion of the soil supporting the pile tip

q'_v = effective vertical stress at pile tip level

N_c^*, N_q^* = pile end-bearing capacity factors

The end-bearing (tip) resistance of a pile gets into full effect when tip settlements reach 1.0 – 2.5 mm. in rocks, 2.5 to 7.5 per cent of shaft diameter in granular soils, and five to 15 per cent of shaft diameter in cohesive soils. Driven piles exhibit full tip resistance at lower settlement values than bored cast in-situ piles (Candogan, 2008).

2.2.2. Friction Piles and Friction Bearing Capacity

When no layer of rock or rock-like material is present at a reasonable depth at a site, point bearing piles become very long and uneconomical. In this type of subsoil, piles are

driven through the softer material to specified depths as shown in Figure 2.4(c). If the value of (Q_p) is relatively small, then these piles are called ‘friction piles’, because most of their resistance is derived from skin friction. However, the term ‘friction pile’, although used often in the literature is a misnomer; in clayey soils, the resistance to applied load is also caused by adhesion. Pressure distribution in the soil varies widely between single piles and groups, and also with the shape of structure and relation between its width and pile lengths. Figure 2.5 shows the type of soil action (Das, 2004 and Leonards, 1962).

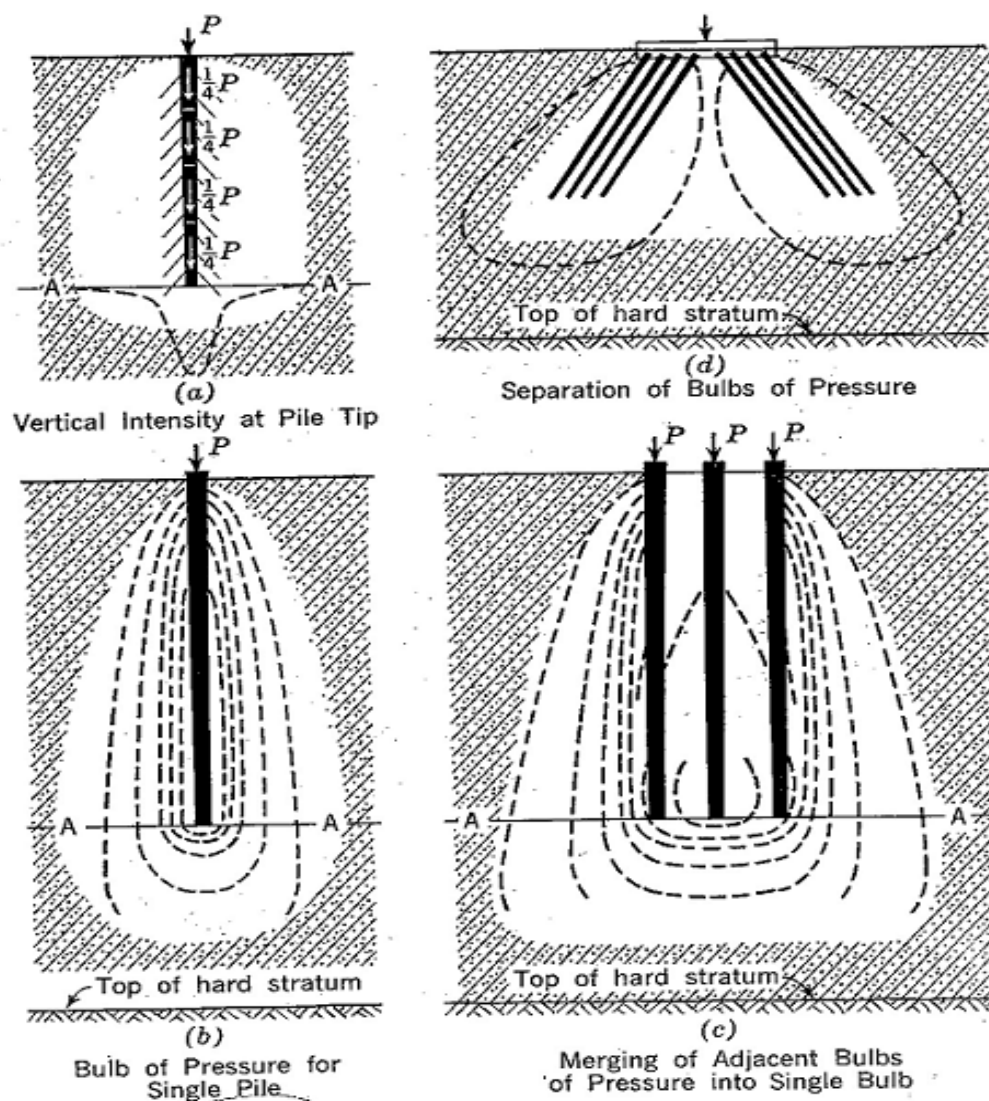


Figure 2.5. Friction-pile pressure distributions in soil (Leonards, 1962)

The frictional resistance of a pile may be written as (Candogan, 2008):

$$Q = \sum p \Delta L f \quad (2.5)$$

where,

p = Perimeter of the pile

ΔL = Pile depth along which perimeter and unit friction resistance are constant

f = Unit friction resistance at the depth increment

In the field, for full mobilization of the point resistance, the pile tip must go through a displacement of 10 to 25 per cent of the pile diameter (Das, 2004).

2.2.3. Bearing Capacity of Bored Piles in Rock Sockets

The design of piles socketed into rock is traditionally based on local knowledge derived from observation of full scale static load tests, empirical factors related to the unconfined compressive strength of intact rock, or conservative city or state ordinances. The uncertainties inherent in such approaches must inevitably lead to either a reduced level of confidence that the foundation will safely or satisfactorily perform its function of supporting the superstructure, or to overdesign, and thus additional expense to ensure that a suitable level of confidence is achieved (J. Seidel, 2000).

Where the piles are installed by drilling through soft overburden onto a strong rock, the piles can be regarded as end-bearing elements and their working load is determined by the safe working stress on the pile shaft at the point of minimum cross-section, or by code of practice requirements. Bored piles drilled down for some depth into weak or weathered rocks and terminated within these rocks act partly as friction and partly as end-bearing piles. The factors which govern the bearing capacity and settlement of the pile are summarized as (Tomlinson, 2004):

- The length to diameter ratio of the socket.
- The strength and elastic modulus of the rock around and beneath the socket.
- The condition of the side walls; roughness and the presence of drill cuttings or bentonite slurry.

- Condition of the base of the drilled hole with respect to removal of drill cuttings and other loose debris.
- Layering of the rock with seams of differing strength and moduli.
- Settlement of the pile in relation to the elastic limit of the side wall strength.
- Creep of the material at the rock/concrete interface resulting in increasing settlement with time.

The keying of the shaft concrete to the rock and hence the strength of the concrete to rock bond is dependent on the strength of the rock. The ultimate bond stress is related to the average unconfined compression strength by the equation:

$$f_s = \alpha q_u^\beta \quad (2.6)$$

where,

f_s = Ultimate bond stress

q_u = Unconfined compression strength

α = A reduction factor relating to q_u as shown in Figure 2.6.

β = A correction factor related to the discontinuity spacing in the rock mass as shown in Figure 2.7.

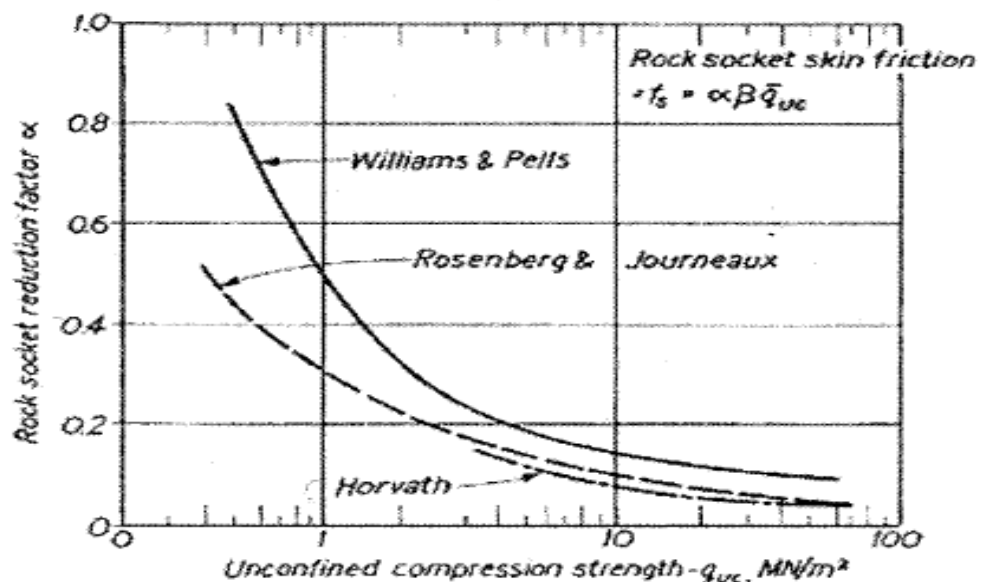


Figure 2.6. Reduction factors for rock socket skin friction (Tomlinson, 2004)

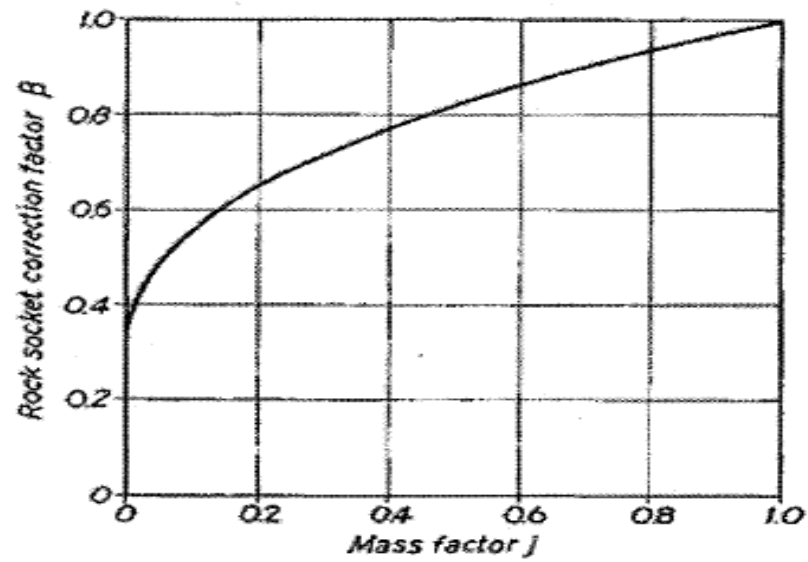


Figure 2.7. Reduction factors for discontinuities in rock mass (Tomlinson, 2004)

The β factor is related to the mass factor, j , which is the ratio of the elastic modulus of the rock mass to that of the intact rock as shown in Figure 2.8.

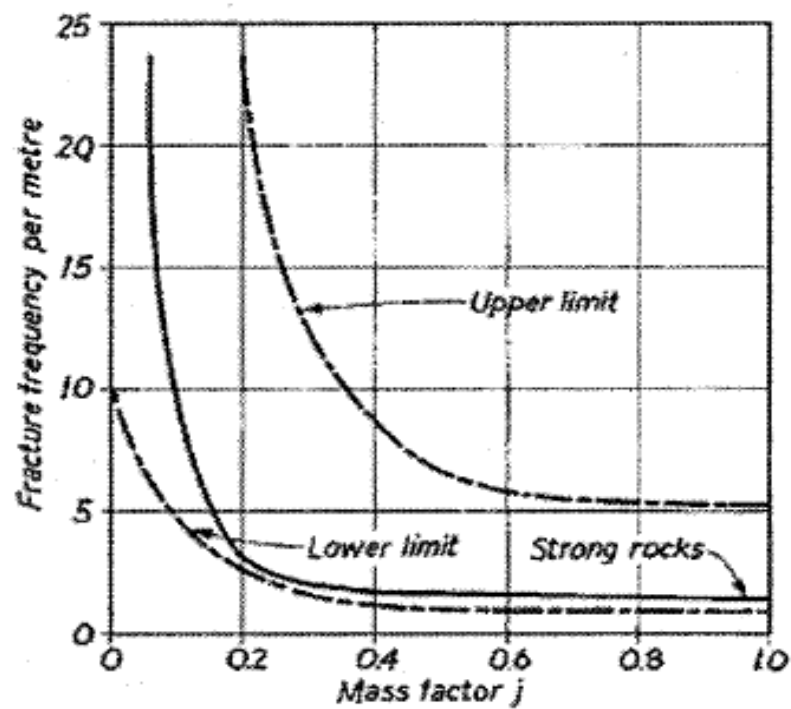


Figure 2.8. Mass factor value (Tomlinson, 2004)

If the mass factor is not known from loading tests or seismic velocity measurements, it can be obtained approximately from the relationships with the rock quality designation (RQD) or the discontinuity spacing as presented in Table 2.1 (Tomlinson, 2004):

Table 2.1. RQD – mass factor relationship

RQD (%)	Fracture frequency per metre	Mass factor j
0 - 25	15	0.2
25 – 50	15 - 8	0.2
50 – 75	8 - 5	0.2 – 0.5
75 – 90	5 - 1	0.5 – 0.8
90 - 100	1	0.8 – 1.0

Correlations between the unconfined compression strength of the rock and rock socket bond stress have been established by various formulas below at Table 2.2.

Table 2.2. Various design formulas for ultimate rock socket bond stress (Yıldırım, 2004)

$f_s = 0.21 q_L^{0.5}$	Horvath and Kenney (1979)
$f_s = 0.20 q_L^{0.5}$	Carter and Kulhawy (1988)
$f_s = 0.44 q_L^{0.3}$	Williams et al. (1980)
$f_s = 0.40 q_L^{0.5}$	Rowe and Armitage (1984)
$f_s = 0.34 q_L^{0.5}$	Rosenberg and Journeaux (1976)
$f_s = 0.30 q_L$	Reynolds and Kaderbek (1980)
$f_s = 0.20 q_L$	Gupton and Logan (1984)
$f_s = 0.15 q_L$	Reese and O'Neill (1987)
$f_s = 0.25 q_L$	Toh et al. (1989)
$f_s = 0.22 q_L^{0.6}$	Migh and Wolshi (1979)
$f_s = 0.20 q_L^{0.5}$	Horvath (1982)

The allowable end-bearing resistance of bored and cast in-situ piles in weak rocks depends on drilling techniques. The use of percussive drilling tools can result in the formation of a very soft sludge at the bottom of the drill hole which can give a wrong impression of the true character of the rock (Tomlinson, 2004).

When the joints in the rock layer are spaced widely, or where the joints are tightly closed and remain closed after pile driving, the ultimate base resistance may be calculated from the equation:

$$q_b = 2N_\phi q_u \quad (2.7)$$

where,

N_ϕ = Bearing capacity factor:

$$N_\phi = \frac{1 + \sin \phi}{1 - \sin \phi} \quad (2.8)$$

ϕ = Soil friction angle

q_u = Unconfined compressive strength

For a coarse sandstone which typically has friction angle values between 40° and 45°, the base resistance is stated by Pels and Turner to be between nine and 12 times unconfined compressive strength. Friction angles for intact rock are given in Table 2.3 which should be used only as a guide because of the wide variations which can occur due to site conditions (Tomlinson, 2004).

The values of the bearing capacity factor obtained from the friction angle of an intact rock can be reduced substantially if the rock mass has open or clay-filled joints, or if joints which are tightly closed in-situ are subsequently opened by pile displacement and

vibrations. In the case of open joints the ultimate base resistance may be no more than the unconfined compressive strength of the intact rock.

Table 2.3. Friction angles for various rock types (Tomlinson, 2004)

Classification	Type	Friction angle (degrees)
	Schists (high mica content)	
Low friction	Shale	20 to 27
	Marl	
	Sandstone	
	Siltstone	
Medium friction	Chalk	27 to 34
	Gneiss	
	Slate	
High friction	Basalt	34 to 40
	Granite	

2.2.4. Bearing Value of Piles in Groups

In most cases, piles are used in groups, as shown in Figure 2.9, to transmit the structural load to the soil. Theory and tests have shown that the total bearing value of a group of friction piles, particularly in clay, may be less than the product of the bearing value of an individual pile multiplied by the number of piles in the group. The reduction in value per pile depends on the shear value of the soil surrounding the group and the size, spacing, and length of the piles. No reduction due to grouping occurs with end-bearing piles. For combined end-bearing and friction piles, only the load-carrying capacity of the frictional portion is reduced. Reduction of bearing value is caused by overlapping zones of pressure around each pile as like in the figure. Ideally, the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles. In practice, the minimum center to center pile spacing, is $(2.5D)$ and in ordinary situations, is actually about $(3-3.5D)$. Moreover, rather than reducing the nominal working load per pile, it is usually more economical to increase the

individual pile bearing value by increasing its length so that when the reduction factor is applied, the full nominal value will result (Leonards, 1962 and Das, 2004).

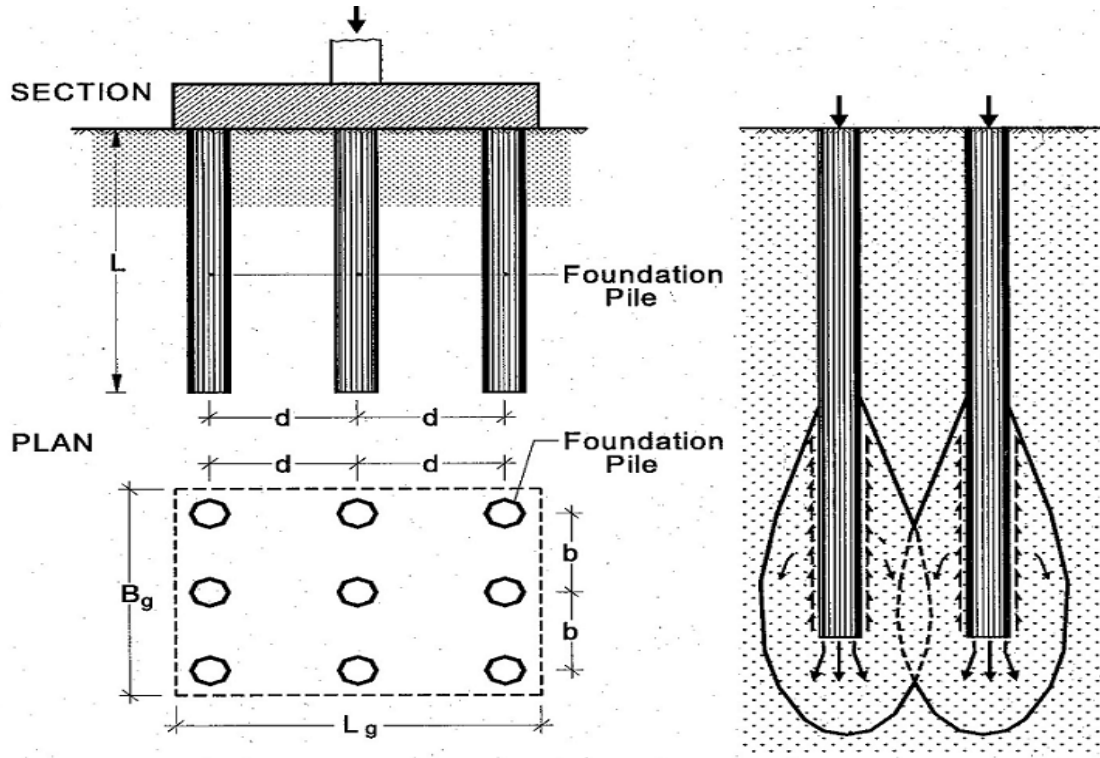


Figure 2.9. Group effect of piles (Candogan, 2008)

The efficiency of the load-bearing capacity of a group pile may be defined as:

$$\eta = \frac{Q_{g(u)}}{\sum Q_u} \quad (2.9)$$

where,

η = Group efficiency

$Q_{g(u)}$ = Ultimate load bearing capacity of the group pile

Q_u = Ultimate load bearing capacity of each pile without the group effect.

Load capacity of individual piles (when only frictional resistance is considered) in a group embedded in sand can be assigned by a method. According to this method, the ultimate capacity of a pile can be explained if Figure 2.10 is examined; which shows the

plan of a group pile. For pile type A, there are eight adjacent piles, for pile type B, there are five, and for pile type C, there are three. With this mind, Table 2.4 can be prepared (Feld, 1943).

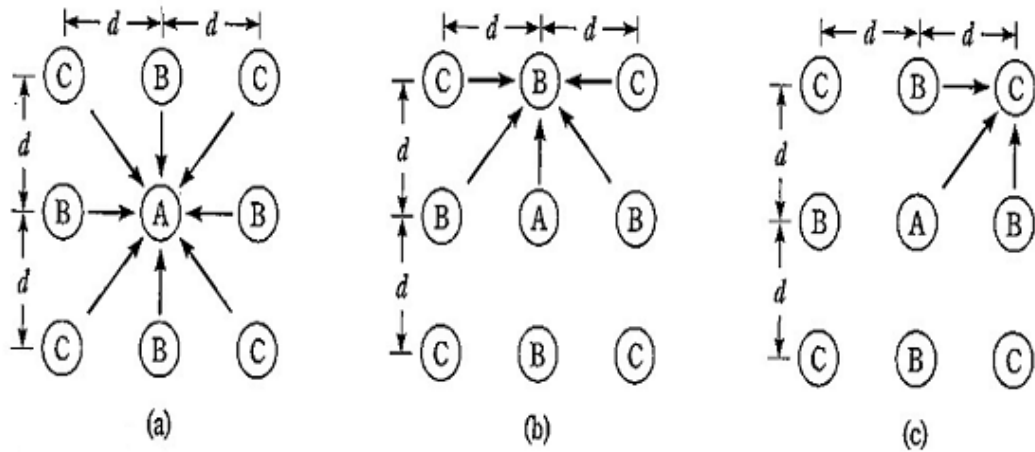


Figure 2.10. Feld's method for estimating the group capacity of friction piles

Table 2.4 Load capacity reduction calculated by Feld method

Pile type	No. of piles	No. of adjacent piles	Reduction factor for each pile	Ultimate capacity
A	1	8	$1 - \frac{8}{16}$	$0.5Q_u$
B	4	5	$1 - \frac{5}{16}$	$2.75Q_u$
C	4	3	$1 - \frac{3}{16}$	$3.25Q_u$
Q_u = ultimate capacity for an isolated pile				$\sum Q_u = Q_{g(u)}$

Hence,

$$\frac{Q_u}{Q_{g(u)}} = \frac{Q_u}{\sum Q_u} \quad (2.10)$$

2.3. Bored Pile Design

Bored piles, like all other piles, are designed as structural and deep foundation elements. The structural design ensures that the pile carries the imposed loads within allowable material stresses. The foundation design ensures that the pile bears the structural loads by inducing sufficient frictional resistance and/or end bearing reaction from the ground. Then, the design capacity and allowable bearing capacity of a bored pile can be expressed as follows (Candogan, 2008):

$$Q_d \leq Q_{all} = \frac{Q_p}{FS_p} + \frac{Q_s}{FS_s} \quad (2.11)$$

where,

Q_d = Design load capacity

Q_{all} = Allowable load bearing capacity

Q_p = Ultimate end bearing capacity

FS_p = Factor of safety for end bearing

Q_s = Ultimate friction resistance capacity

FS_s = Factor of safety for frictional resistance.

Piles' dead weights also need to be considered in pile design. Reinforced concrete piles with large diameters and H-piles have substantial weights. The unit weight of reinforced concrete ranges from 24 to 25 kN/m³, and the unit weight of steel ranges from 78 to 80 kN/m³. Considering their effective weights when under the water table, piles' actual dead weights need to be subtracted from their allowable bearing loads (Candogan, 2008).

$$Q_d \leq Q_{all} = \frac{Q_p}{FS_p} + \frac{Q_s}{FS_s} - Q_o \quad (2.12)$$

where,

Q_o = pile's actual dead weight which is calculated by considering water buoyancy effects.

After pile installation, soils around piles may undergo settlements caused by consolidation of clay or compaction of granular layers. Recent fills, alluvial and colluvial soils which have not completed their natural compaction and consolidation processes continue to settle causing drag-down forces on the piles through skin friction resistance. Also, some man or nature caused phenomena causing lowering of ground water, or the surcharge loads of floor slabs directly bore on the ground will cause settlements of the grounds and cause drag down forces on the piles. Often, drag-down forces will reach important relative magnitudes and will need to be deducted from allowable bearing capacities. So, the equation becomes (Candogan, 2008):

$$Q_d \leq Q_{all} = \frac{Q_p}{FS_p} + \frac{Q_s}{FS_s} - Q_o - Q_{nsf} \quad (2.13)$$

where,

Q_{nsf} = Drag down force due to negative skin friction

A common error in the design of deep foundations is to superimpose friction resistances and end bearing capacities. In some soils, full friction resistance is reached with a small (~0,5 cm.) displacement while significant end bearing capacity activation needs three to 15 cm. of displacement. Therefore, superposition of bearing resistance of deep foundations should be used with care, always remembering that the ultimate goal of foundation design is to stay within structurally and architecturally allowable total and differential displacements.

Structurally, a bored pile is almost always a reinforced concrete element although it can be just a concrete element. Once the frictional resistance and end-bearing reaction of the bored pile are shown to safely carry the design loads, then, the pile needs to be

designed as a structural element. Structural design consists of checks for its concrete and steel stresses under the allowable pile bearing loads. The foundation design of a bored pile consists of determining its load bearing capacity under allowable settlements. Installation techniques play an important role on the realization of the friction resistances and end-bearing reactions. The crucial parameter is to reach the bearing capacity within allowable settlements.

It is necessary to divide the calculated ultimate resistance of the pile by a safety factor to obtain the design working load on the pile. Safety factors for various deep foundation design codes are presented with detail in Section 5.6. A safety factor is required for the following reasons (Tomlinson, 2004):

- To provide for natural variations in the strength and compressibility of the soil.
- To provide for uncertainties in the calculation method used.
- To ensure that the working stresses on the material forming the pile shaft are within the safe limits.
- To ensure that the total settlement(s) of the single isolated pile or the group of piles are within tolerable limits.
- To ensure that the differential settlements between adjacent piles or within groups of piles are within tolerable limits.

However, where piles are end bearing on a strong intact rock, the concept of a safety factor against ultimate failure does not apply, since it is likely that the pile itself will fail as a structural unit before shearing failure of the rock beneath the pile toe occurs. The allowable loads are then governed by the safe working stress in compression and bending on the pile shaft and the settlement of the pile due to elastic deformation and creep in the rock beneath the base of the pile, together with the elastic compression of the pile shaft.

3. SETTLEMENT BEHAVIOR OF BORED PILES

3.1. The Behaviour of a Pile Under Vertical Load

When a pile is subjected to a progressively increasing compressive load at a rapid or moderately rapid rate of application, the resulting load – settlement curve is as shown in Figure 3.1. Initially the pile-soil system behaves elastically, considering elastic shortening of piles. There is a straight-line relationship up to some point A on the curve and if the load is released at any stage up to this point the pile head will rebound to its original level. When the load is increased beyond point A there is yielding at, or close to, the pile-soil interface and slippage occurs until point B is reached, when the maximum skin friction on the pile shaft will have been mobilized. If the load is released at this stage the pile head will rebound to point C, the amount of ‘permanent set’ being the distance OC. The movement required to mobilize the maximum skin friction is quite small and is only of the order of 0,3 to 1,0 per cent of the pile diameter. The base resistance of the pile requires a greater downward movement for its full mobilization, and the amount of movement depends on the diameter of the pile. It may be in the range of 10 to 20 per cent of the base diameter. When the stage of full mobilization of the base resistance is reached (point D) the pile plunges downwards without any further increase of load, or small increases in load produce increasingly large settlements (Tomlinson, 2004).

Friction piles may be used in the upper part of a deep, fairly uniform soil to shift the zone of maximum stress to lower portions, where less settlement will be caused. Settlement from a uniform load on cohesive soil increases in direct proportion to the width of a rectangular and diameter of a circular area. In sandy soils, the settlement under a given pressure tends to be more nearly independent of the size of the loaded area. Settlement also depends upon the ratio of depth to diameter of the loaded area, so that for equal potential settlement reductions, the depth-diameter ratios should be equal. The effect of this ratio is less in cohesionless soils. This indicates that the value of piles may be greatly affected by the relation of their lengths to the width of the loaded area. Under a narrow structure, every effort should be made to keep the piles longer than the width of the structure, so that the bulb of pressure will be lowered. This will reduce settlements.

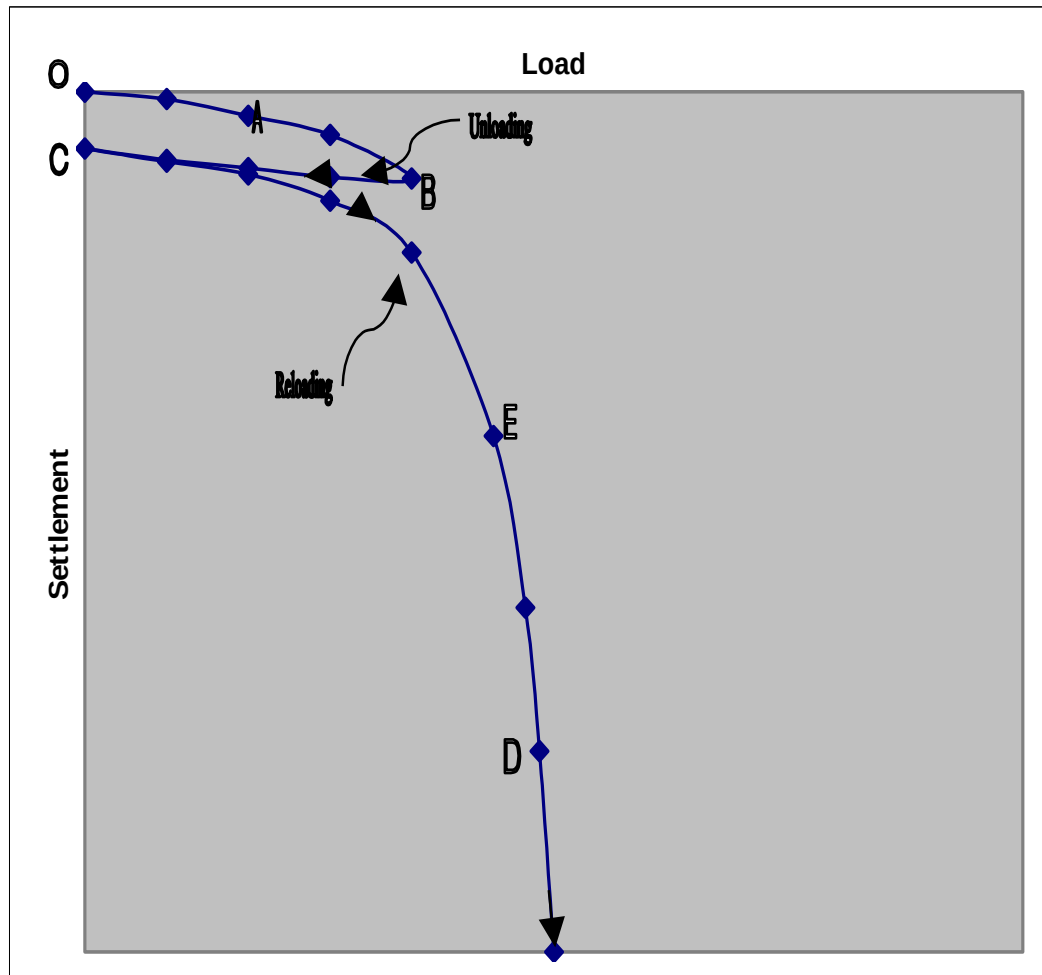


Figure 3.1. Load-settlement curve for compressive load to failure on pile (Tomlinson, 2004)

Figure 3.2 illustrates the two conditions; in narrow building A, the bulb is much lowered by piles; in wide building B, lower soil pressures are little affected and piles might be of little or no value for loads or settlement decreases. In loading A, the bulb spreads relatively more than in loading B, where little spread occurs and the effect of the pile foundation on the soil is much the same as from a raft foundation. In the latter case, the total bearing value of the foundation bears no relation to the carrying value of a single pile by itself (Leonards, 1962).

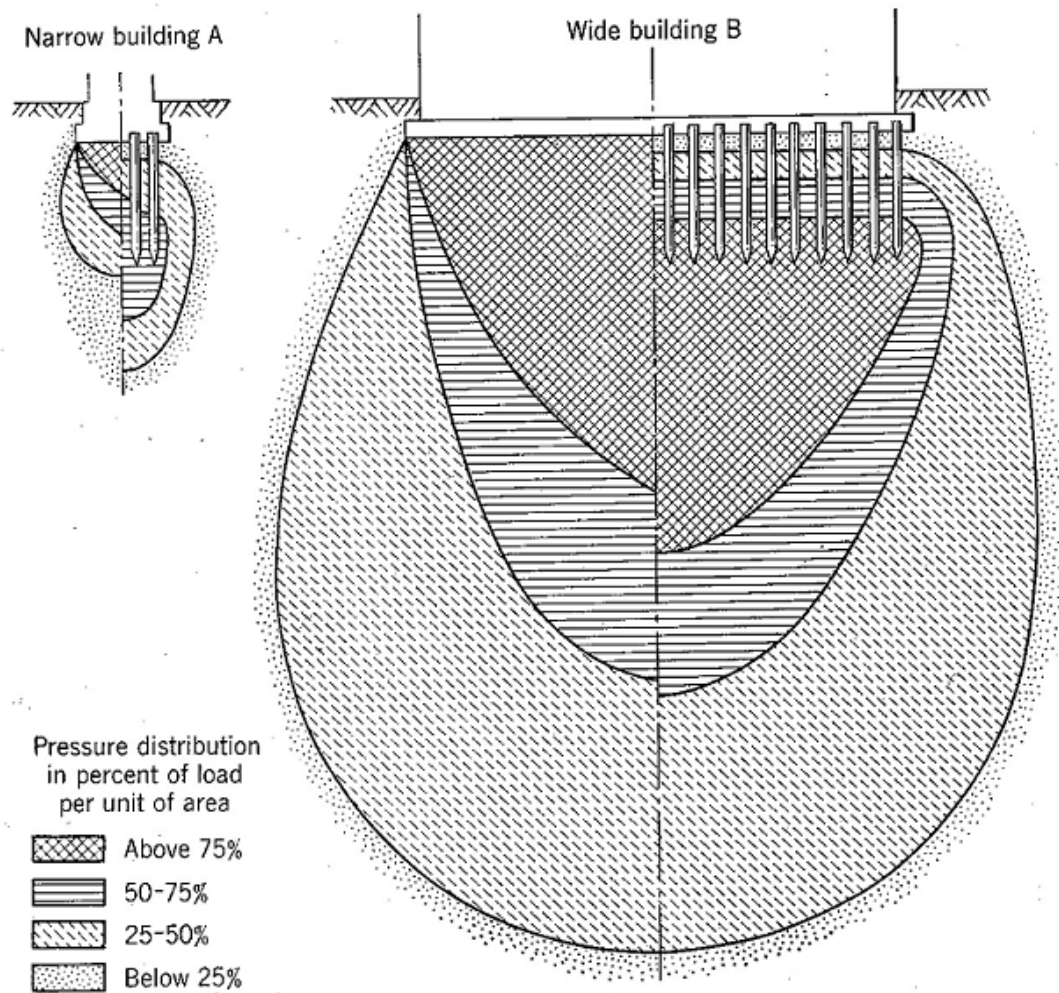


Figure 3.2. Effect of relation between foundation width and pile length on pressure distribution (Leonards, 1962)

3.1.1. Pile Load Transfer Mechanism

If strain gauges are installed at various points along the pile shaft from which the compressive load in the pile can be deduced at each level, the diagrams illustrated in Figure 3.3 are obtained, which show the transfer of load from the pile to the soil at each stage of loading shown in Figure 3.1. Thus when loaded to point A virtually the whole of the load is carried by skin friction on the pile shaft and there is little or no transfer of load to the toe of the pile as shown in Figure 3.3(a). When the load reaches point B the pile shaft is carrying its maximum skin friction and the pile toe will be carrying some load as shown in Figure 3.3(b). At point D there is no further increase in the load transferred in skin friction but the base load will have reached its maximum value as illustrated in Figure

3.3(c). The concept of the separate evaluation of shaft friction and base resistance forms the basis of all 'static' calculations of pile carrying capacity (Tomlinson, 2004).

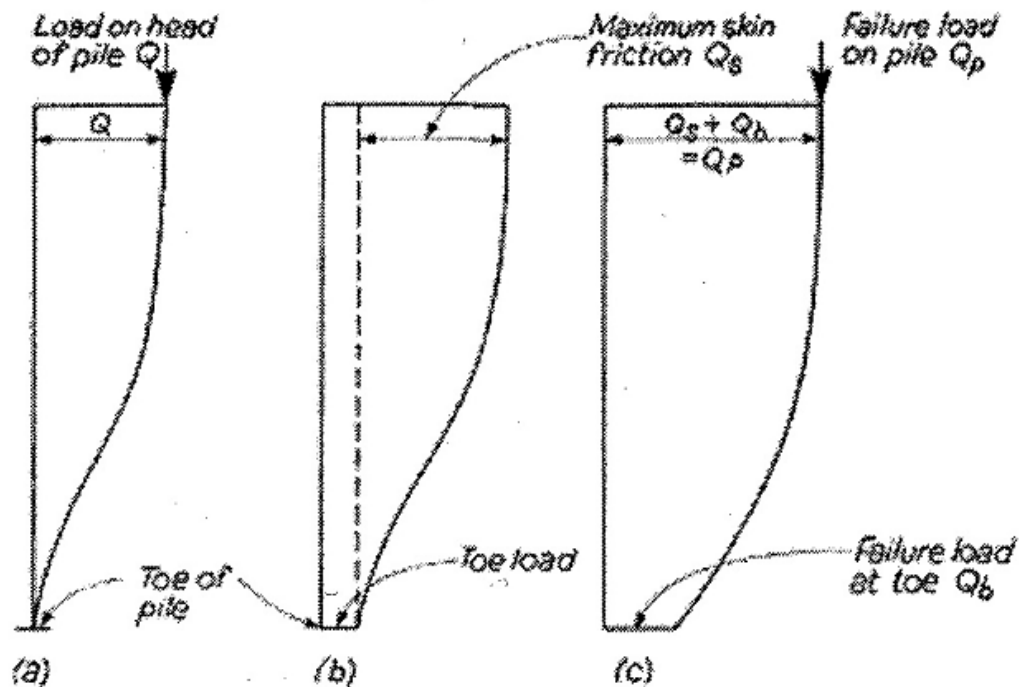


Figure 3.3. Load transfer from head of pile to shaft (Tomlinson, 2004)

- (a) At point A on load-settlement curve in Figure 3.1
- (b) At point B on load-settlement curve in Figure 3.2
- (c) At point D on load-settlement curve in Figure 3.3

The static capacity and settlement of a pile can be back-computed from load-transfer data obtained from one or more test piles that are sufficiently instrumented with strain gauges and/or telltales as shown in Figure 3.4(b) as will be more deeply examined in Chapter 5. The difference in measured load (or stress) between any two points is taken as the load transferred to the soil by skin resistance and is assumed constant in the segment length. The shear resistance is readily computed since the pile perimeter and segment length are known. The segment deformation can be computed using the average axial load in the expression (PL / AE) , and if the point displacement is known or assumed, the segment movement (termed slip) is known. A curve of slip versus shear resistance can be plotted as in Figure 3.4(c) for later use in estimating static capacity for surrounding piles. Several load increments must be applied to the pile in order to develop a load-transfer curve. A load transfer curve can be developed for each pile segment over the shaft

considered as compressible short columns of unit length. The soil is represented by a series of non-linear springs one to each element acting independently of one another. Another spring represents the compression of the soil beneath the pile base as shown in Figure 3.5. Each column is subjected to an axial force and skin friction.

By using the skin friction formula presented in section 3.1.1, the elastic compression of the column is calculated as:

$$\Delta l = l_{i+1} - l_i = \frac{Q_i}{E_p A_p} \quad (3.1)$$

where,

A_p = Cross-sectional area of the column

E_p = Elastic modulus of pile

Q = Load

L = Pile section length.

From the above, a set of simultaneous equations can be derived which, providing the axial forces are known and one boundary displacement is known or can be assumed, permit the displacements along the pile to be determined. The axial forces on the pile are calculated from the ‘transfer function’ determined analytically or experimentally which is an empirical or semi-empirical relationship of the form (Tomlinson, 2004):

$$Q_i = Q_{i+1} + k_i \Delta l_i \quad (3.2)$$

where,

k = Transfer function

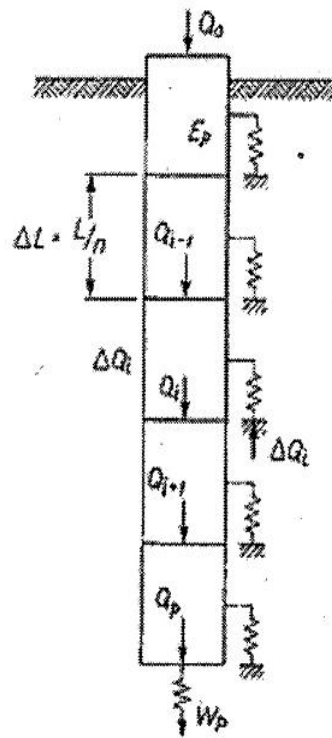


Figure 3.5. Load-transfer analysis for the transfer function approach (Tomlinson, 2004)

The principal shortcoming of the transfer function method is that the ‘springs’ act independently of one another, that is, any soil layer represented by a spring is unaffected by movements in the layers above and below. The elastic continuum method avoids this problem by assuming that each element forms part of a rigid incompressible pile surrounded by compressible soil consisting of a homogeneous, elastic, isotropic continuum and the axial load at the pile head is resisted by frictional forces on each element. Slip at the pile/soil interface can be allowed for by changing the magnitude of the friction force on the element.

The equations used in the transfer function and elastic continuum methods can be written in matrix form and evaluated by digital computer. Both methods have the disadvantage that they over-simplify the deformations in the soil mass and ignore changes in the deformation characteristics of the soil caused by the pile installation process and the subsequent loading of the pile. Finite element techniques have been used to simulate these effects and to determine the resulting load/deformation behaviour of the pile. The load-deformation behaviour can be presented in the form of t - z curves as shown in Figure 3.6

relating the pile head settlement or uplift to a range of loading up to the stage of failure. Similar curves can be presented for the compression of the pile base ($q - z$ curve). It was noted at the beginning of this section that the adoption of nominal safety factors in conjunction with conventional methods of calculating pile bearing capacity can obviate the necessity of calculating working load settlements of small-diameter piles. Hence it is necessary to check that the design pile capacity does not endanger the serviceability limit state of the supported structure (Tomlinson, 2004).

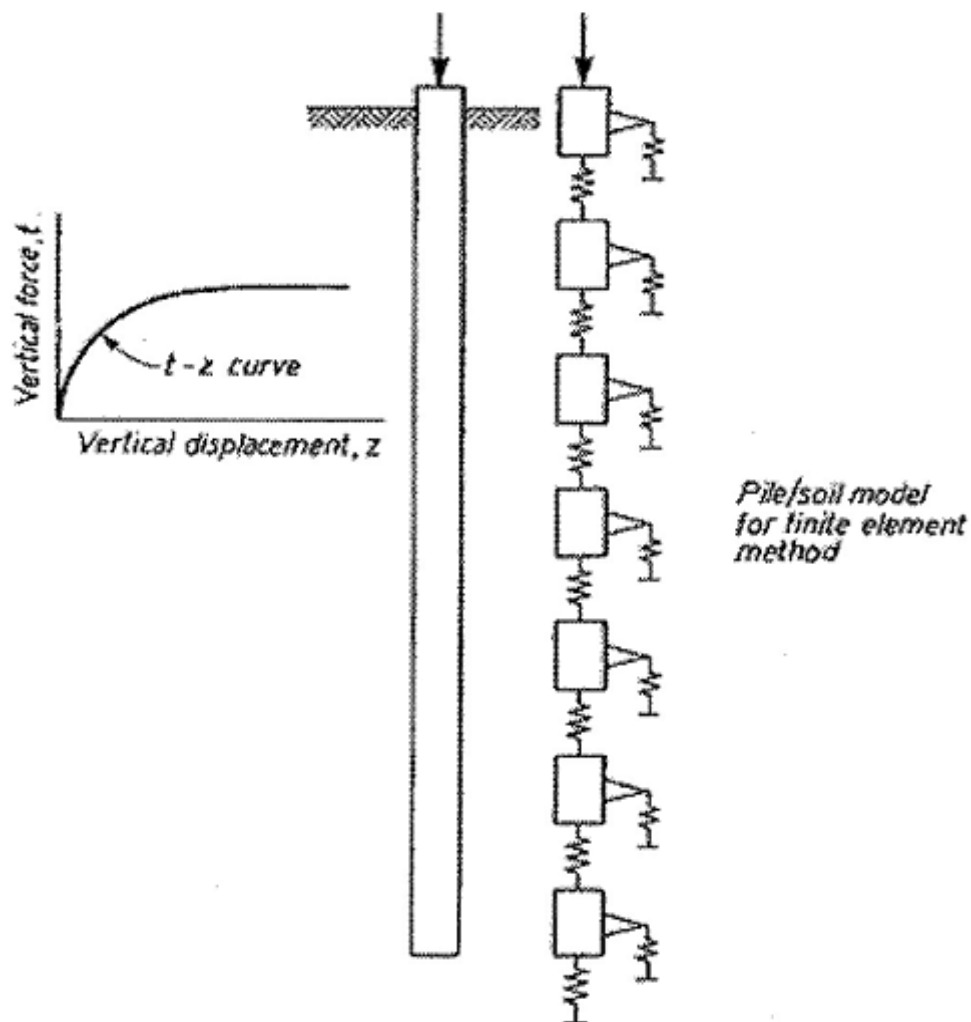


Figure 3.6. The $t - z$ curve for deformation of a pile under vertical axial loading
(Tomlinson, 2004)

At ultimate load, the failure surface in the soil at the pile tip is like that shown in Figure 3.7(c). Pile foundations are deep foundations and the soil fails mostly in a punching

mode. That is, a triangular zone is developed at the pile tip, which is pushed downward without producing any other visible slip surface.

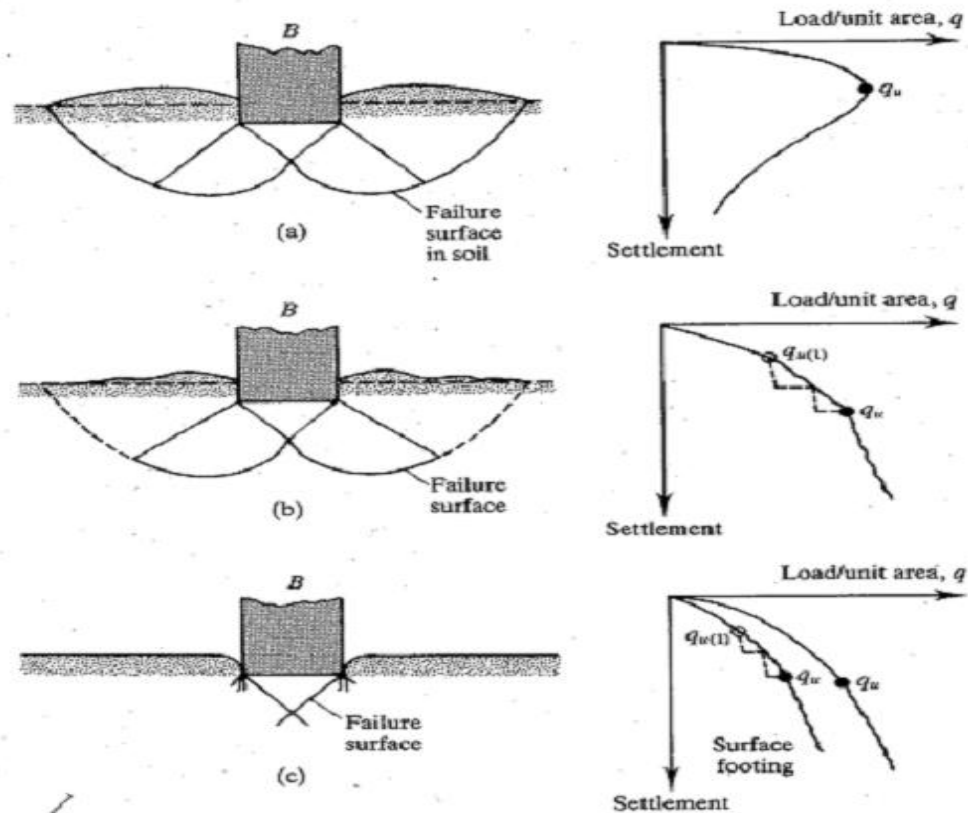


Figure 3.7. Nature of bearing capacity failure in soil: (a) general shear failure, (b) local shear failure, (c) punching shear failure (Das, 2004)

3.2. Elastic Settlement of Piles

Pile settlement calculations are difficult to perform, and the values obtained after tedious calculations are not very accurate. However, shallow and deep foundations are utilized to control settlements, and the importance of settlement calculations in foundation design can not be over-emphasized. When conducted by tenured geotechnical engineers who take into account the installation techniques together with the other relevant data, and especially when calibrated with field load tests, the theoretical calculations will converge to quite accurate values.

Total elastic settlement of a pile (head) is (Das, 2004):

$$s = s_e + s_p + s_s \quad (3.3)$$

where,

s_e = elastic compression of pile

s_p = settlement of pile due to the end-bearing of pile into the soil

s_s = settlement of pile due to the frictional resistance build up.

When the pile material remains within elastic limits, (Das,2004):

$$s_e = \frac{(Q_{wp} + \xi Q_{ws})L}{AE} \quad (3.4)$$

where,

Q_{wp} = pile tip load at working load,

ξ = 0 to 0.5 or 0 to 0.67 based on friction resistance distribution in Figure 3.8

Q_{ws} = total friction resistance at working load

L = pile length

A = pile x-sectional area

E = pile elastic modulus (21 – 210 Gpa).

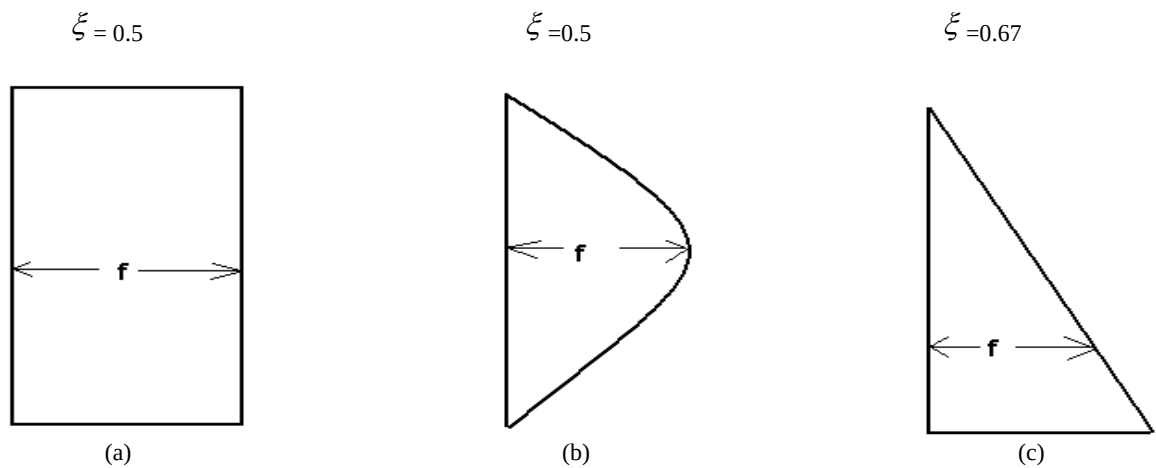


Figure 3.8. Various types of distribution of skin friction resistance along the pile shaft
(Das, 2004)

Elastic modulus of steel (piles) is 200 – 210 Gpa. As concrete's elastic modulus is 21-32 Gpa., reinforced concrete piles have modulus ranges of 23 to 35 Gpa. The elastic compression of short piles is a small value and can be neglected.

Pile settlements due to their tip end bearing into the ground can be written as (Das, 2004):

$$s_p = \frac{q_p D}{E_s} (1 - \mu_s) I_{wp} \quad (3.5)$$

where,

D = Width or diameter of pile

q_{wp} = Point load per unit area at the pile point

E_s = Modulus of elasticity of soil at or below the pile point

μ_s = Poisson's ratio of soil (typically, 0.15 to 0.40)

I_{wp} = Influence factor $\cong 0.85$

Settlement of piles due to their shaft's picking up the frictional resistance is:

$$s_{ws} = \frac{q_s D L}{E_s} (1 - \mu_s) I_{ws} \quad (3.6)$$

where,

p = pile perimeter,

L = pile length,

I_{ws} = Influence factor

If the settlement of piles is excessive, it should be reduced by one or more of the following measures (Tomlinson, 2004):

- Reduce the working load on the pile
- Reduce the load on the base by increasing the shaft resistance, i.e., by increasing the shaft diameter
- Increase the length of the shaft to mobilize greater shaft friction, and to take the base down to deeper and less-compressible soil.

Generally the pile settlements can be calculated if the load carried by skin friction and the load transferred to the base at the working load can be reliably estimated. The pile head settlement is then given by the sum of the elastic shortening of the shaft and the compression of the soil beneath the base as follows (Tomlinson, 2004):

$$\Delta = \frac{W_s}{\pi \phi E_p} + \frac{W_b}{4A_b E_b} \quad (3.7)$$

where,

W_s and W_b = Loads on the pile shaft and base

L = Shaft length

A_s and A_b = Cross-sectional area of the shaft and base

E_p = Elastic modulus of the pile material

B = Pile width

ν = Poisson's ratio of the soil

I_p = Influence factor related to the ratio of (L/R)

E_b = Deformation modulus of the soil beneath the pile base

The pile shaft settlements given by the first term in this equation assume a transfer load to the soil uniformly down the pile shaft. However, the distribution is not uniform where a high proportion of the total load is carried in skin friction on the shaft of deeply penetrating piles.

In rock sockets, the settlement of a bored pile where the load is carried by rock socket skin friction only is calculated using the equation (Tomlinson, 2004):

$$settlement = \frac{Q}{B_d} \quad (3.8)$$

where,

Q = The total load carried by the pile head

l_p = Influence factor

B = The diameter of the socket

E_d = The deformation modulus of the rock mass surrounding the shaft.

Tomlinson (2004) gives a relationship according to BS8004 which estimates the deformation modulus of a jointed rock mass from empirical relationships with the unconfined compressive strength of the intact rock:

$$E_d = j \times M_r \times q_c \quad (3.9)$$

where,

j = Mass factor (Figure 2.8, Table 2.1)

M_r = The ratio of the elastic modulus of the intact rock to its UCR (Table 3.1).

Table 3.1 Values for (M_r) depending on rock type (Tomlinson, 2004)

		Values for M_r
Group 1	Pure limestones and dolomites	600
	Carbonate sandstones of low porosity	
Group 2	Igneous	300
	Oolitic and marly limestones	
	Well-cemented sandstones	
	Indurated carbonate mudstones	
Group 3	Metamorphic rocks including slates and schists	150
	Very marly limestones	
	Poorly cemented sandstones	
	Cemented mudstones and shales	
Group 4	Slates and schists (steep cleavage/foliation)	75
	Uncemented mudstones and shales	

The influences factors of Pells and Turner are shown in Figure 3.9. Where the rock sockets are recessed below the ground surface or where a layer of soil or very weak rock overlies competent rock, a reduction factor is applied to Equation 3.9. Values of the reduction factor are shown in Figure 3.10.

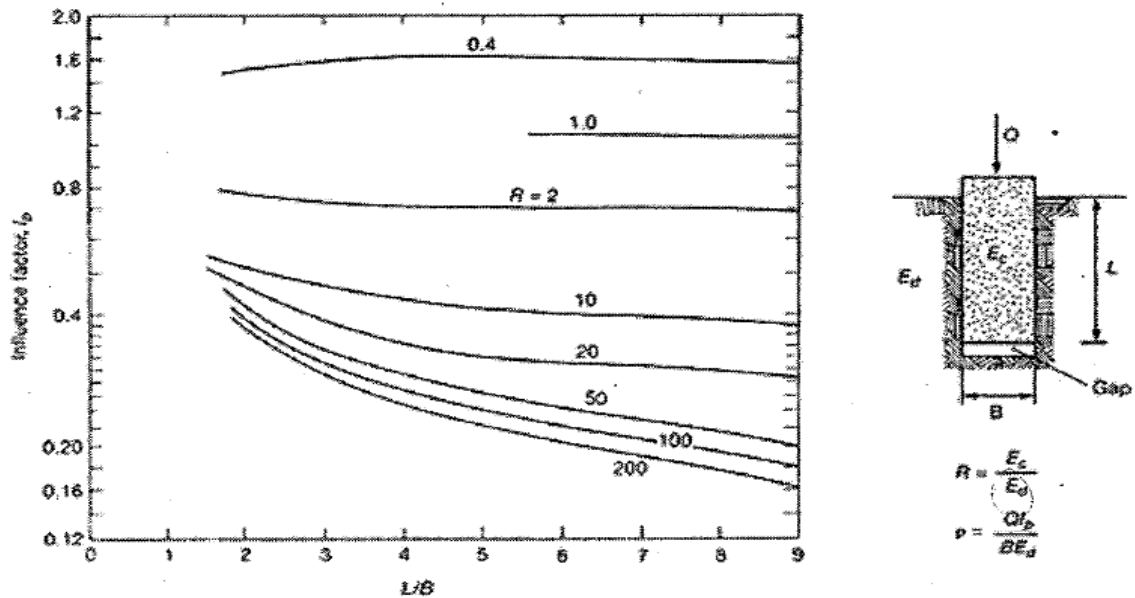


Figure 3.9. Elastic settlement influence factors for rock-socket skin friction on piles (Tomlinson, 2004)

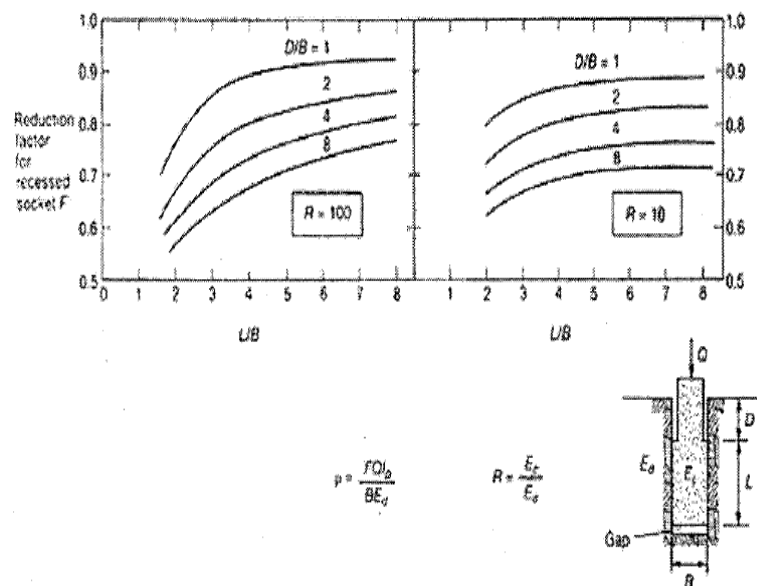


Figure 3.10. Reduction factors for calculation of settlement of recessed sockets (Tomlinson, 2004)

4. INSTALLATION OF BORED PILES

4.1. Introduction

Bored piles are installed by first excavating (boring) the ground to obtain a shaft or hole and then, placing rebar and concrete. While driven piles can be steel, reinforced concrete, composite or timber, bored and cast-in situ piles are reinforced concrete elements. Bored piles have started to be used at a much later date than driven piles. When there were higher load bearing grounds at a particular depth, the surface layers were excavated to that depth and were filled with stone and sand, or lean concrete. Bored piling methods were gradually developed as the ground was excavated by mechanical devices and reinforced concrete was used to make these deep foundations an integral part of the shallow foundations.

In the first half of the 20th century, typical bored pile diameters ranged between 30 to 100 cm and the depths seldom exceeded 30 meters. Today, using elaborate and powerful drilling rigs, piling depths of 100 or more meters, and piling diameters of three to five meters can be achieved. Figure 4.1 shows bored pile installation using a temporary casing oscillator and a rotary drill rig.

Bored piles always incorporate an excavation and therefore, the soils around the periphery need to be kept stable during excavation, rebar cage placement and concreting. Accordingly, various techniques of bored piling are categorized with regard to their excavation support systems. The simplest piling procedure is the installation of bored piles in stiff clays or weak rocks that would require no wall support for stability. The pile is constructed by simply boring the shaft, cleaning the base, placing the rebar cage and tremie concreting the pile (Figure 4.2). Note that, with or without wall support, all cast in-situ piles, even if they are excavated in dry conditions, should be concreted with tremie technique or a comparable technique. The reasons are, first, the prevention of concrete segregation and, second, forcing up the boring debris contaminated first concrete to the top of the pile by a continuous feed of fresh concrete from below.



Figure 4.1. Hydraulic rotary drill machine



Figure 4.2. Steel cage preparation

Very often, either a temporary or permanent casing or a stabilizing liquid is needed in order to stabilize the pile wall during boring, rebar placement, and concreting. Temporary casings are used when pile design foresees a strong concrete soil frictional resistance for economy and of course when it is not viable to use stabilizing fluids due to site restrictions or extreme ground conditions. The permanent casings are used when negative friction (drag down force) is to be minimized, or to protect the pile from hazardous chemicals, or to use the casing as a part of the pile's reinforcement, or if bulging or necking risks are high, or some combination of these reasons. Stabilizing fluids such as bentonite suspensions will often be used for piles with large diameters (greater than one meter) and going down to substantial depths, e.g. over 30 meters.

We can compare the advantages and disadvantages of bored piles as below (Candogan, 2008):

Advantages of bored piles:

- Lengths can be easily adjusted according to ground conditions.
- Pile element is manufactured in-situ under relatively controlled conditions, and risk of damage to the pile is low.
- Can be installed in stiff consolidated clays without causing upheaval.
- Can be installed to great depths and with large diameters enabling the design to large lateral and vertical loads.
- Can be installed with tolerable vibrations and noise.
- There is no risk of ground heave.
- Can be installed in any type of ground conditions as drilling tools can penetrate all pile required grounds.
- Pile base can be belled to increase end-bearing capacity and decrease length.
- Pile base can be inspected and pre or post improved with grouting.
- Rebar cages can be instrumented for testing and quality control.

Disadvantages of bored piles:

- May suffer necking in if the active pressure of the soft soil is higher than the passive pressure of the fresh concrete.
- May suffer bulging out if the active pressure of the fresh concrete is greater than the passive pressure of the soft soil.
- Have low end-bearing strengths due to the disturbance of shafts` base, and it is expensive to remedy this mechanism.
- The shaft stability needs to be ensured for most types of soils.
- The excavated material needs to be disposed off the site.
- Inspection and quality control of the underwater reinforced concrete pile shaft is difficult.

Depending on the installation procedure a number of possible hazards arise, which can hamper the pile integrity and quality. With bored piles these dangers are (Klingmüller and Kirsch, 2004):

- Interruption of the concrete flow due to inadequate synchronisation of the pullout velocity of the casing and the level of the fresh concrete
- Segregation of the concrete during the free fall phase at the end of the concreting pipe
- Hydraulic fracture at the bottom of the bore hole due to insufficient counter water pressure
- Lack of sufficient penetration of the casing below the bottom of the bore hole.

In order to avoid faulty pile installation, specifications and codes of pile installation stipulate measures of quality assurance, which monitor the decisive parameters continuously in a way that deviation from standard values can be noticed. Recording of the boring depth and the penetration depth respectively, the compilation of a meaningful pile installation record and concrete quality checks are the most important quality measures during pile installation. Also after installation is completed, piles can suffer certain damages. Cracks in unreinforced piles can arise from horizontal loads during the construction processes on site. Horizontal deformation can occur in soft layers, where the concrete lacks lateral support with subsequent bulging of the pile shaft (Klingmüller and Kirsch, 2004).

4.2. Bored Pile Installation

Bored piles are installed by boring or excavating the ground, next placing the rebar cage into the excavated borehole, then filling the hole with concrete using the tremie technique. Large rigs, also called as carriers, which are specially manufactured crawler cranes, are used to carry the mechanical or hydraulic rotary drilling machines. Sometimes the hydraulic crawler cranes and machines are manufactured as special purpose piling rigs (Figures 4.3 and 4.4). In both cases, the augers or buckets attached to the telescopic mechanical or hydraulic kelly bars of the rotary machines are used for boring the ground. Service cranes are used for rebar cage and concrete placing operations. Similar to the driven piling practice a daily pile production report needs to be filled in for every bored pile (Candoğan, 2008).



Figure 4.3. Piling rig machine (IMT AF220) Figure 4.4. Rig machine (Bauer BG28)

4.2.1. Borehole Drilling

Every drilling technique has a level of difficulty related to four basic parameters (Candogan, 2008):

4.2.1.1. Ground Conditions. The homogeneity or heterogeneity of the ground conditions may necessitate different drilling techniques and tools. Also, extreme hardness or softness of clays, the looseness of granular soils, abrasive hardness of calcareous sandstones with quartzite veins, and overburden layers all impose difficulties to the task of drilling. The difficulty is not only with the process of drilling but is also related to flushing. If the formation is karstic, highly faulted or cavernous, the flushing fluid frequently lost as it is not able to recirculate to surface, making drilling very difficult if not impossible. Similarly, in the presence of artesian water pressures, drilling can be a very difficult task.

4.2.1.2. Length. The pile length is always an important factor of difficulty in foundation engineering. The drilling rods` torsional and flexural rigidities need to be increased with depth which means that their weights also increase. The maneuvering times also increase as additional segments of kellys would need to be added or extracted. All these factors result in the usage of heavier and more expansive drilling equipments which have higher winching, extracting and thrusting capacities. Also, the flushing of the excavated debris becomes more and more difficult as the depth increases. Thus, the capacity of the air compressors and water pumps must be increased. All, these increases in capacity mean more expensive and heavier accessory equipment. Heavier drilling and accessory equipments are difficult to mobilize and the costs add up to make deep drilling a very expensive operation.

4.2.1.3. Diameter. The diameter is also an important factor of difficulty in drilling. Needed torques, and therefore torsional rigidities, and thus weights of the rods increase. This would mean more powerful, heavier and more expensive drilling machines. Flushing volumes and pressures must be increased which requires more powerful, heavier and more expensive accessory equipment. Mobilizations would be more difficult and costly for the heavier machines.

4.2.1.4. Drilling location conditions. The site conditions at the drilling location can also be very important difficulty parameters. The drilling can be done above ground or underground. The above ground or surface drilling does not mean that the drilling conditions would be easier. There could be a congested rural site where working, maneuvering and logistics are extremely difficult. Also, it may be necessary to work

hanging on a cliff which is not a favorable working condition for sure. Perhaps, the most difficult drilling location condition is a grouting gallery or tunnel under a large dam when the operations are continued after impounding. Not only the access, logistics, geometry and maneuvering are restricted, but high artesian water pressures make drilling extremely difficult.

4.2.2. No or Partial Casing Supported Installation

In very stiff and hard clays and in weak rocks, pile excavation can be realized without any shaft wall support. Care must be shown to clean the pile base properly, and keep it clean during rebar cage and concrete placement, because possible collapse in the borehole bottom might prevent rebar cage to be installed properly. Unsupported excavated holes should be concreted without delay to ensure a clean pile tip. As the ground near the surface is erratic and can be affected by the surface activity around the pile, often a temporary casing is placed for the top three-four meter of the pile hole. This casing, which is removed after concreting, also serves as a guide for the verticality of the pile and is called a 'guide casing'.

4.2.3. Temporary or Permanent Casing Supported Installation

In most projects, temporary and permanent casings are used to support the periphery of the pile shaft during excavation and concrete placing. These casings are generally placed and extracted by vibrohammers, casing oscillators or duplex hydraulic pile drilling rigs. The illustrative drawings given in Figure 4.5 show vibrohammer driven/extracted, temporary casing supported bored piling method which has been successfully implemented in cohesionless soils such as fine and coarse sands and small gravels. The vibro driving of the casing improves the granular soils and friction bearing capacities increase substantially. Also, the vibro extraction process vibrates the concrete so that the concrete is well placed in and around the cage (Candoğan, 2008).

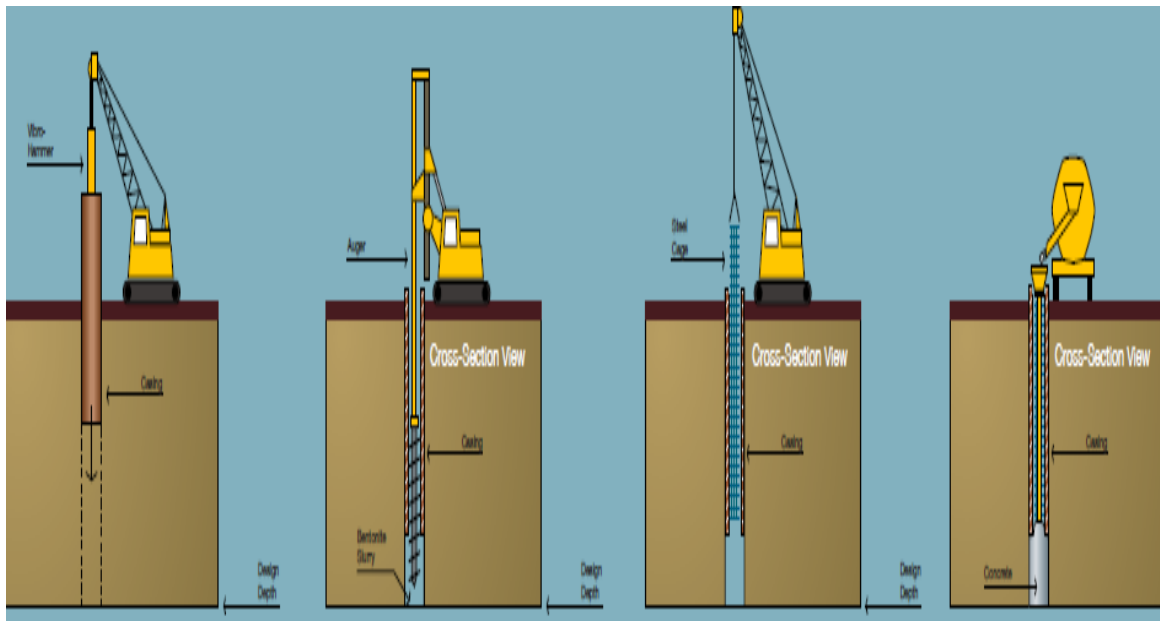


Figure 4.5. Bored pile construction with temporary casing supported method (Land Transport and Authority, <http://www.lta.gov.sg/>, 2004)

As the diameter and depth of the piles get larger, the capacity and, therefore, the weight of the vibro hammer need to be increased. At the same time, the boom of the carrier crane needs to be taller and, accordingly, the crane capacities become impractically high. Therefore, this technique is normally feasible for pile diameters under 120 cm and pile depths less than 30 meters. Pile production speeds are very high as the technique is suitable for industrial line production when multi casings are used, and there is no bentonite suspension circulation, desanding, and disposal problems. Figure 4.6 shows casing oscillator driven and extracted segmental casing method. The casing oscillator exerts vertical thrust together with oscillations to drive the casings. After each casing is placed the soil inside is excavated by rotary augers/buckets (Figures 4.7 and 4.8) or piling grabs. The sturdy casings are double walled and are connected with the aid of specially designed locking wedges which are embedded at the joints. After the excavation is completed, the rebar cage is placed and concrete is placed with the help of a tremie pipe. Care has to be shown not to float up the rebar cage. Also, the level of concrete has to be carefully monitored as the casings are extracted one by one. During the extraction the oscillator applies an upward lifting force (Candoğan, 2008).



Figure 4.6. Casing oscillator used to advance casings down to the limestone (H. Beck Foundation Co., Inc.)



Figure 4.7. 24 inch rock auger and double row auger (<http://www.jpself.com>)

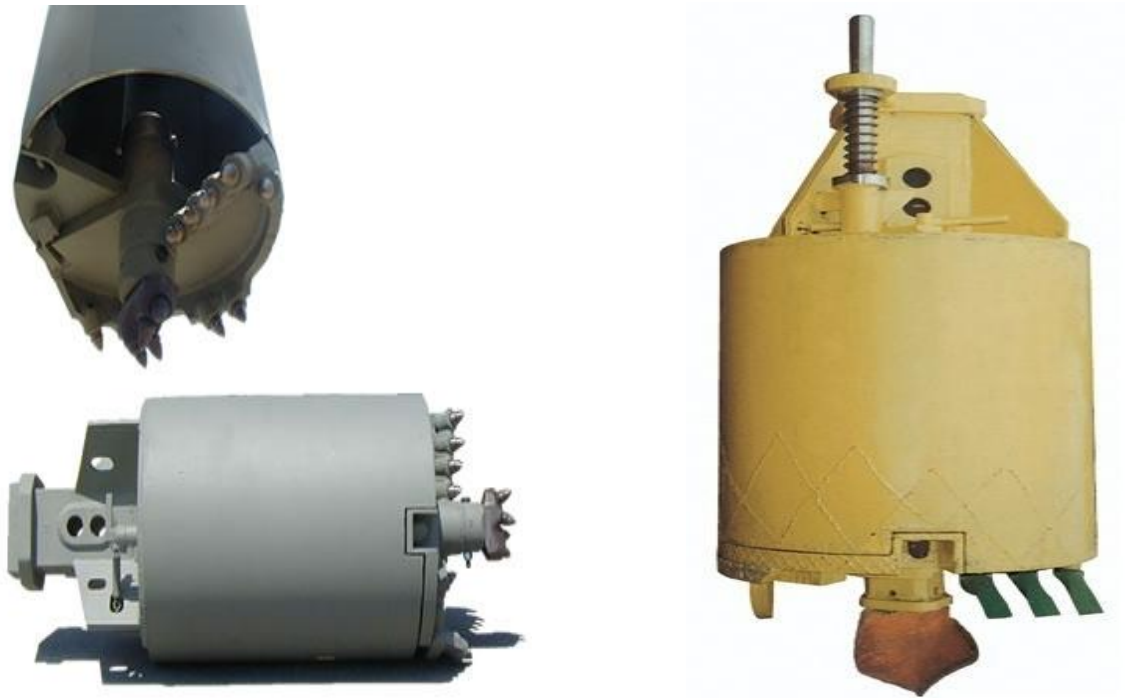


Figure 4.8. Various types of drilling buckets (<http://www.jeffreymachine.com> and <http://www.akelmakina.com.tr>)

4.2.4. Stabilizing Fluid (Bentonite Suspension) Supported Excavation

As the piling diameters get bigger and lengths get deeper, driving or extracting full depth casings can be very cumbersome if not impossible. Also, certain clays would have high cohesion values which cause stickiness on casings and it would be difficult to penetrate or extract casings. Site and environmental conditions permitting, stabilizing fluids e.g. bentonite suspensions can be used to stabilize the walls of the pile excavation. While plain water may be sufficient to stabilize cohesive soils; thixotropic fluids, obtained by mixing bentonite powder or similar clays with water using agitators and compressors after which the suspension attains its properties in resting tanks, are often used in cohesionless soils. Also, high speed colloidal mixers promptly prepare such mixes with sufficient specific gravity (G_s) and thixotropy (Figure 4.9). Usually, the density, sand content, viscosity and pH of bentonite is tested on site. The viscosity value should be between 32 sec. – 50 sec. ; the density should be between 1.0 – 1.50; and the sand content should be maximum four per cent, all based on the plasticity of the concrete, the diameter and the reinforcement cage configuration of the pile.



Figure 4.9. Bentonite slurry mixers (IECSA Co.,
<http://www.iecsacv.com/graphics/eb/DSC00112.jpg>)

Pile boring using stabilizing fluids is often the most economical bored piling technique, especially if the suspension can be recycled with the help of de-silters and de-sanders, plus, discarding of the debris and the over contaminated fluid is viably possible. Most urban and many rural areas will have stringent rules on discarding of the contaminated fluid. Piles installed with the help of stabilizing fluids are bored with a three to five meters long ‘guide casing’(Figure 4.10). Guide casings prevent shaft stability problems under live surface loads, or when suspension levels may fall/rise abruptly during excavation/concreting. Often, the rebar cages are temporary welded to the guide casing by two or four hooks, to ensure rebar cage stability and maintain the project top of steel level requirement. Sometimes, pile base plates are used to hold down the rebar cage. Continuous concreting and maintaining the tremie pipe at least two meters below concrete level is important for quality of pile (Figure 4.11).



Figure 4.10. Guide casing installed bored pile excavation

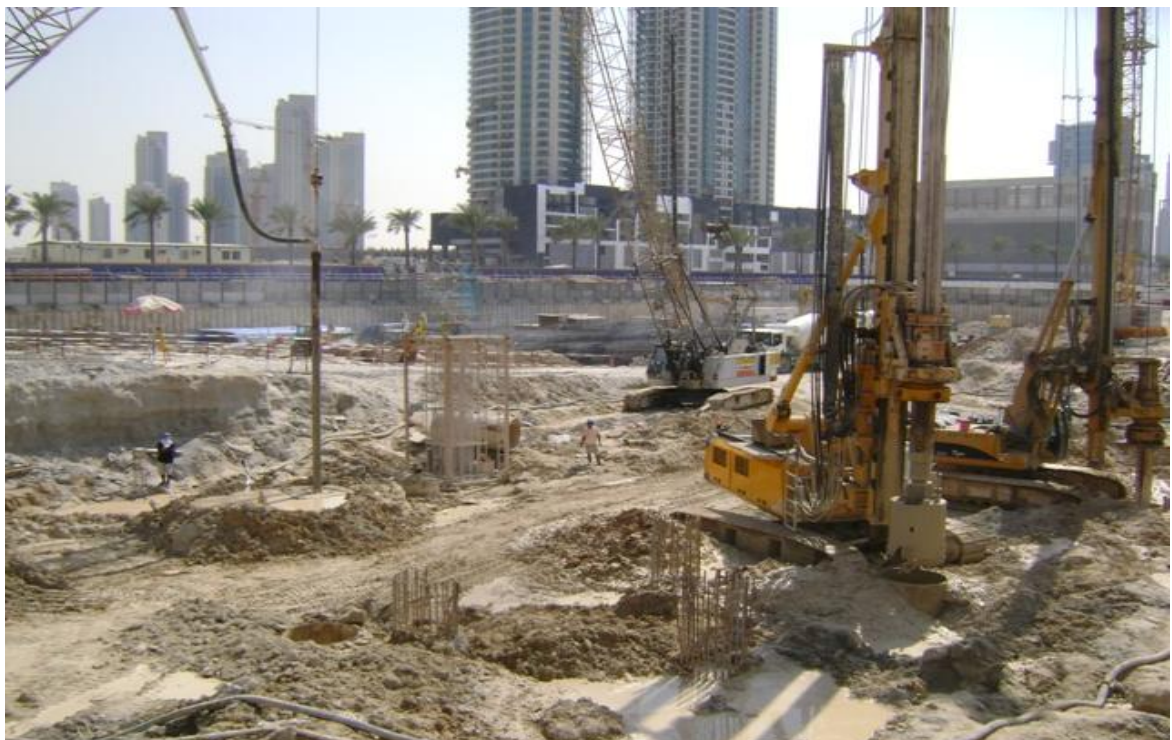


Figure 4.11. Concreting and drilling process

Figures 4.12, 4.13, 4.14 and 4.15, respectively show the bentonite suspension supported production method, which were used in all Case Studies presented in Chapter 6. First, the stabilizing fluid, which is most commonly a bentonite suspension, is prepared. Then, as excavation progresses, the prepared fluid is placed into the pile shaft and after boring finish, steel and tremie installation is maintained. The contaminated fluid is collected back, desanded, desilted and reused by sending again to the pile by pumps, named as 'flushing'. As concrete is placed, the stabilizing fluid is placed in storage tanks for recycling. This technique is often faster and cheaper than the full depth casing methods.

Pile defects may occur if casting is not maintained properly. First, casting should be maintained continuously and a delay in casting between two concrete trucks, wherever the concrete level on the pile is, may decrease the possibility of good pile chemistry and bonding and may cause the closure of tremie pipe connections, the weakest point of tremie pipes. Next, lifting up the tremie pipes above concrete level by the crane during casting may cause pile defects and decrease strength of the pile as then not only liquid will enter the tremie and decrease concrete strength but also avoid the cleaning of pile body from dirty concrete consisting of concrete-mud mix coming from toe of the pile.



Figure 4.12. Stabilizing fluid, bentonite preparation and storage



Figure 4.13. Steel installation



Figure 4.14. Flushing of borehole by desanders



Figure 4.15. Pile casting

5. FIELD TESTING OF BORED PILES

5.1. Introduction

Ultimate bearing capacity and settlement calculations of a pile are conducted on mathematical modeling of the pile and soil conditions. As it is improbable to correctly simulate the real conditions that correlate to the actual installed state of piles in the field, such model based analyses seldom give fully reliable results, and should always be used with caution. Full scale field tests are the most dependable way of measuring the load bearing and settlement behavior of piles. Testing measures the response of a pile-soil system to loads as in load testing or electronic impulses as in integrity testing and so may provide data for research and development, engineering design, quality assurance, or acceptance or rejection in accordance with the specifications and contract documents. Thus, load and integrity testing of piles are crucially important for the success of a piling program.

However, pile load testing is an expensive task as it demands long hours of engineering time, test set-up costs, and most important of all, significant time losses in schedule. The late evaluation of the measurement taken for a quality test of the pile is not ideal, since a possibility of finding a problem that requires remediation on the pile may cause both delays in construction time and extra costs. Nevertheless, no matter how costly, time consuming and demanding pile testing can be, any significant piling project cannot be conducted without the verification of a comprehensive pile testing schedule. Recent developments in equipment and methods aid in real-time monitoring of various pile installations. Electronics have progressed in the last decade where real-time measurements for deep foundation solutions are made routinely. Datas are acquired digitally and results are transferred electronically and stored in computer files. Besides, introduction of cellphones make site engineering much more practical since immediate reporting of problems and observations is routine. Deep foundation installations require inspection, and electronic methods now commonly can provide significant detailed inspections automatically.

Due to the crucial importance of pile testing in geotechnical engineering practice, several codes and specifications give information and standards on pile testing, as well as various case studies, manuals and textbooks are devoted to this subject.

5.2. Conventional Top Down Static Loading Test

5.2.1. General Comments

The objectives of a static load test is usually (Tomlinson, 2004):

- To determine the ultimate bearing capacity of the piles or shafts, relating this to the design parameters
- To separate the total resistance contributed by the side shear and end bearing capacity
- To determine the stiffness of the soil/rock and shaft system at design load. A back analysis of the tests data will enable the soil/rock modulus to be evaluated, and hence the deformation of shaft groups may be predicted with greatly increased confidence.

Two principal types of test are used for compressive loading on piles. The first of these is the constant rate of penetration (CRP) test, in which the compressive force is progressively increased to cause the pile to penetrate the soil at a constant rate until failure occurs. The second type of test is the maintained load (ML) test in which the load is increased in stages to some multiple, say 1.5 times or twice the working load with the time-settlement curve recorded at each stage of loading and unloading.

The CRP method is essentially a test to determine the ultimate load on a pile and is therefore applied only to preliminary test piles or research-type investigations. The method has the advantage of speed in execution, less than one working day, and because there is no time for consolidation or creep settlement of the ground the load-settlement curve is easy to interpret. The CRP test is not suitable for checking the compliance with the specification requirements for the maximum settlement at given stages of loading. There is also the

difficulty of pricing tenders for this form of load testing since the failure load on the pile is not known with any certainty until the test is made.

The ML test is best suited for contract work, particularly for proof loading tests on working piles. It is also suitable for use where empirical methods are employed to predict the ultimate load from measurement of residual deflections after returning the test load to zero at four or five stages up to the maximum. The test consists of loading the pile incrementally in stages and returning the load to zero after each increment of 100 per cent, 150 per cent and 200 per cent. This test is very time consuming, requiring from 30 to 70 or even more hours to complete. This form of test is necessary if the net settlement curve is used as the basis of defining the failure load. It is essential to maintain a constant rate of load application. It has been found that the slower the rate the smaller is the ultimate failure load. It is a high possibility that ultimate load might not be estimated by test results as maximum load increment of 200 per cent of the working load on the pile is smaller than the failure load. Therefore, methods described in Section 5.2.7 can be used for interpretation of test data.

There is also the quick maintained load test method, in which the load is applied in increments of about 10 percent of the proposed design load and maintained for a constant time interval, usually about 2 to 15 minutes. The duration of this test will generally be about 45 minutes to 2 hours, again depending on the variation selected. The advantage of this test, like the CRP test, is that it can be completed in less than one working day. Unlike the CRP test, however, no special loading equipment is required. Dynamic testing is also accepted to be defined as a quick maintained load test due to its pace (Tomlinson, 2004).

5.2.2. Testing Equipment

The apparatus for applying compressive loads to the test pile or pile group shall be constructed so that the loads are applied to the central longitudinal axis of the pile or pile group to minimize eccentric loading. Load reaction system, test girder, reference beams, jacking system including the hydraulic jacks, hydraulic pump, and pressure gage, load cell are the main equipments for the test.

5.2.3. Methods for Applying Loads

The following methods can be used to apply the compressive load on the test shaft (Zhang, 2004):

- A platform is constructed on the head of the shaft and a mass of heavy material, termed 'kentledge' is placed on the platform.
- A bridge, carried on temporary supports, is constructed over the test shaft and loaded with enough dead weight. The ram of a hydraulic jack, placed on the shaft head, bears on a cross-head beneath the bridge beams, so that a total reaction equal to the weight of the bridge and its load may be obtained.
- Reaction shafts capable of withstanding an upward force are constructed on each side of the test shaft, with a beam tied down to the head of the reaction shafts and spanning the test shaft. A hydraulic jack on the head of the test shaft applies the load and obtains a reaction against the underside of the beam (Figure 5.1).
- Ground or rock anchors are constructed to transfer the reaction to stiff strata below the level of the shaft base.



Figure 5.1. A typical static test arrangement with the third method

For methods (i) and (ii), more or less any material available in sufficient quantity can be used as the dead weight. Specially-cast concrete blocks or pigs of cast iron may be hired and transported to the site. The cost of transport to and from the site is a significant factor. Regular-shaped blocks have the advantage that they may be stacked securely, and are unlikely to topple unexpectedly. Sheet steel piling, steel rail, bricks, or tanks full of sand or water have been used as the dead weight from time to time. It is important that the mass of material be stable at all times during and after the test. The reaction beams for methods (ii) and (iii) are subjected to high bending and buckling stresses and they should be designed to carry the maximum load safely. The maximum safe load should be clearly marked on the beam so that it is not inadvertently exceeded during a test. The third method was used in the static tests conducted in Case Studies studied in Chapter 6. A typical load test arrangement is shown in Figure 5.2.

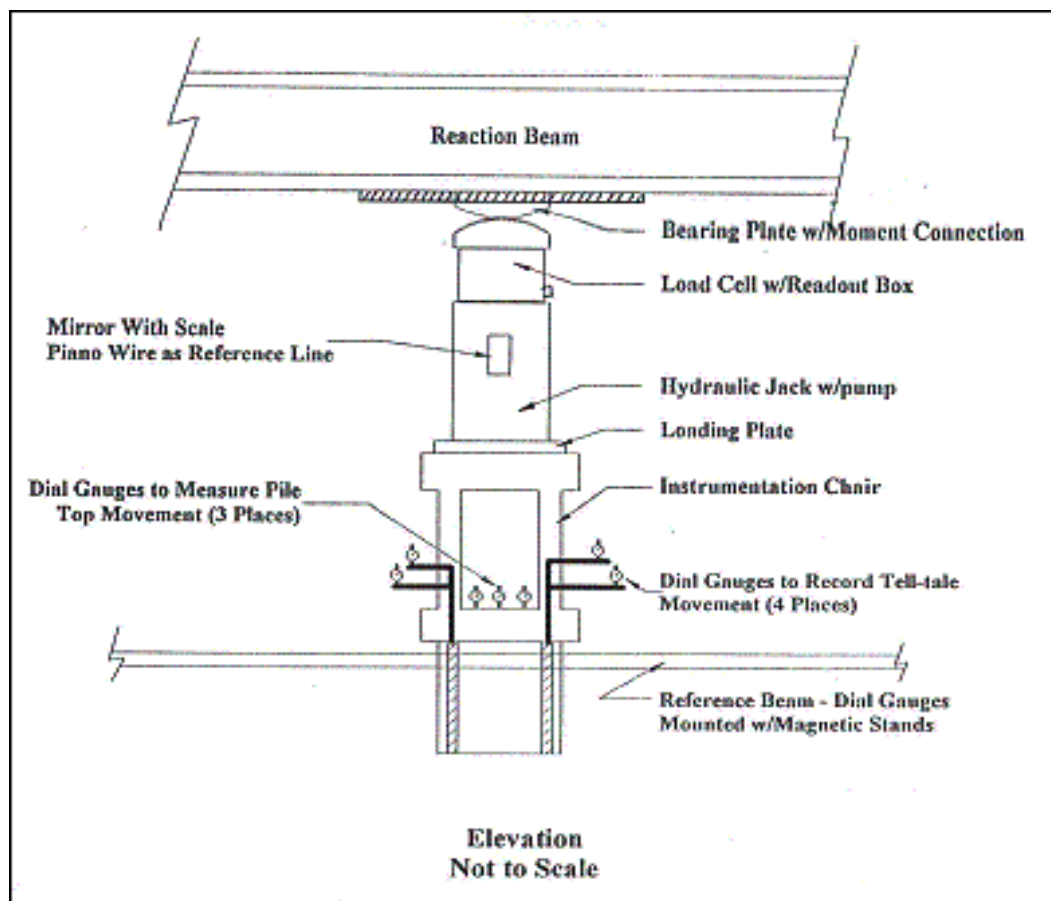


Figure 5.2. A typical static load test arrangement showing instrumentation

(<http://www.fhwa.dot.gov>)

Stresses are transferred from the supports as in method (ii), reaction shafts as in method (iii) or ground anchors as in method (iv) through the soil/rock mass, and such stresses can influence the behavior of the test shaft. Therefore, the supports, reaction shafts or ground anchors must be located well away from the test shaft to minimize this effect. Whitaker (1975) recommends that the supports be more than 1.25 m away from a test pile, to minimize the effect of the supports on pile settlement. He also recommends that any reaction pile should be at least three test-pile diameters from a test pile, center to center, and in no case less than 1.5 m. The specifications of the American Society for Testing and Materials for piling, ASTM D-1243, require that a clear distance of at least five times the maximum of the diameters of the reaction or test piles must exist between a test pile and each reaction pile (ASTM, 1995). According to O'Neill and Reese (1999), a 3.5 diameter center-to-center spacing between each reaction shaft and the test shaft is adequate to minimize the reaction shaft and test shaft interaction for loading tests of large diameter drilled shafts in cohesive soils or rocks.

As the upper portion of an anchor cable does not usually transfer load to the soil/rock, ground anchors can be placed closer to the test shaft than can reaction shafts. The main disadvantages of ground anchors as a reaction system are the axial flexibility of the anchor tendons and the lack of lateral stability of the system. To reduce the extension of the cables during the test it is usual to pre-stress the anchors to as high a proportion of the maximum load as possible. Hence a suitable reaction frame and foundation must be provided to carry this load safely (Zhang, 2004).

Pile head preparation is another important part of the loading tests (Figure 5.3). As the pile head is subjected to highest stress levels, its concrete strength should be safely sufficient during the test. This often requires test pile head preparations with early strength concrete as soon as the test pile is chosen. Normally, a thick-walled steel pipe is encased around the pile head and a thick (3.8 to 5 cm) steel plate is welded on to the top so as to be perfectly level. For large diameter piles, still a thick pipe encasement is used but the pile head is completed with high and early strength concrete which must be finished smooth and level. The test pile area shall be excavated to the proposed pile cut-off elevation. The test piles shall be cut off or built up to the proper grade as necessary to permit construction of the load-application apparatus, placement of the necessary testing, and instrumentation

equipment, and observation of the instrumentation. Where necessary, the unsupported length of the test piles shall be braced to prevent buckling without influencing the test results.



Figure 5.3. Pile head preparation for the static loading test

5.2.4. Measurement of Load

Calibrated pressure gauges are sometimes used in conventional load tests to determine the load applied to the test shaft by measuring the hydraulic pressure in the loading ram. However, calibrated pressure gauges may only provide acceptable accuracy of load measurement for increasing load because the unloading cycle is usually nonlinear owing to friction within the hydraulic jack. Therefore, load measurement is preferably carried out using a load cell. Figure 5.4 shows a typical load cell. The following lists four commonly adopted load-measuring devices (Fleming et al., 1992):

- Hydraulic load cell capsules, maximum capacity 4.0 MN (450 tons) and accuracy about 0.5 per cent
- Load columns, maximum capacity 8.9 MN (1000 tons) and accuracy about one per cent
- Proving rings, maximum capacity 1.8 MN (200 tons) and accuracy about 0.5 per cent
- Strain gauged load cells of various types, maximum capacity 1.8 MN (200 tons) and accuracy about 0.5 per cent.



Figure 5.4. A typical load cell (www.pcb.com)

For very large loads, an array of load columns or strain gauged load cells may be used. It is important that the load cell is calibrated regularly and that, for those devices sensitive to temperature, suitable corrections are made. When tests are run to obtain information on load transfer in side shear and end bearing resistance, several methods can be used to measure the distribution of load along the length of the drilled shaft, some of which being briefly described below (Fleming 1992, O'Neill and Reese, 1999):

- Sister bars: The load distribution along the length of a drilled shaft can be obtained using sister bars. A sister bar is a section of reinforcing steel at the middle of which is placed a strain transducer as shown in Figure 5.5. The sister bar is tied to the rebar cage and its leadwires routed to the surface. The strain transducer can be a vibrating wire gauge or an electrical foil resistance strain gauge. The vibrating wire transducer has the advantage that it tends to be stable over longer periods of time than an

electrical resistance transducer because the later is quite sensitive to the invasion of moisture. However, electrical resistance transducers are more adaptable to data acquisition systems than vibrating wire transducers. The electrical output can be converted to strain in the steel rebar through an appropriate calibration factor, which then can be assumed to be equal to the strain in the concrete section. The internal load in the shaft can then be obtained by multiplying the axial stiffness of the test shaft by the strain obtained at the depth of interest.

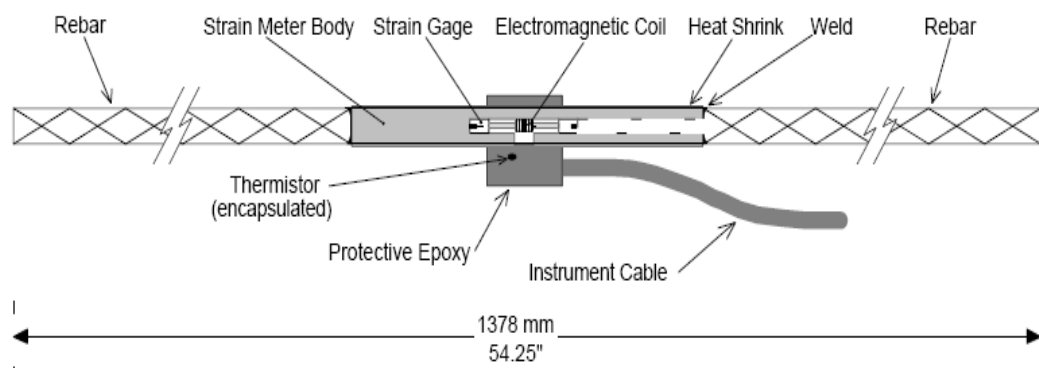


Figure 5.5. Rebar strain meter (Hayes and Simmonds, 2002)

- **Mustran cells:** The mustran cell is mounted on the rebar cage before inserting it into the borehole, in a manner similar to that for sister bars. Because of its electrical circuitry, the Mustran cell indicates strains that are larger than, but proportional to, the actual strains in the concrete. This feature is advantageous for testing large-diameter drilled shafts subjected to relatively small loads. Data from mustran cells are collected and interpreted in a manner very similar to that for sister bars. As with sister bars, it is good practice to place mustran cells at opposite ends of diagonals at any level so that the averaged readings cancel any bending effects that may occur.
- **Telltale:** Telltales are unstrained metal rods that are inserted into one or more tubes that prevent them from bonding to the concrete in the shaft. The telltales extend to a series of depths along the length of the shaft. The shortening of the test shaft over a particular length can be found by using displacement transducers to measure the difference in the movement of the shaft head and the top of the unstrained rod. Such measurements must be made for each of the telltales and for each of the applied

loads. For a particular applied load, the average deformation measured at each depth can be used to plot a compression of the shaft versus depth curve. Differentiation of this deformation versus depth curve with respect to depth will yield the strain of the shaft as a function of depth. The internal load in the shaft can then be obtained by multiplying the axial stiffness of the test shaft by the strain obtained at the depth of interest.

- Pressure cells: Pressure cells placed at the bottom of the cage can be used to measure the base resistance directly. Pressure cells are accurate because they do not require the assumption of a value for the Young's modulus of the concrete. Since stresses across drilled shaft bases are generally uniform, the base resistance can be simply obtained by multiplying the average pressure from the cells with the contact area of the shaft base.

5.2.5. Measurement of Displacement

The most commonly used systems to measure the head displacement of the test shaft are dial gauges or electronic displacement transducers such as linear variable differential transformers (LVDT) that are held by stable reference beams. Dial gauges are simple, robust, mechanical precision instruments, and as such they should be carefully stored and maintained. The reference beams should be long enough so that they can be supported by firm foundations well away from the test shaft and reaction shafts. If possible, adjacent drilled shafts at least four shaft diameters from the test shaft and reaction shafts can be used as the support foundations. It is noted that a compromise must frequently be made between a long beam prone to vibration and temperature-induced displacements, and a shorter beam with support foundations in the zone of influence of the test shaft or reaction shafts.

Optical leveling may also be used to measure the head displacement of the test shaft. With an optical system it is easy to arrange for the instrument and reference point to be well away from the zone of disturbance. When tests are run to obtain information on load transfer in side shear and end bearing resistance, telltales as described earlier can be used

to measure the displacement of the test shaft from point to point along its length (Fleming et al., 1992).

5.2.6. Loading Procedures

Mainly, there are two test methods; maintained load test (ML) and constant rate of penetration (CRP) tests. Moreover, there are less used methods of `quick load test`, `cyclic loading`, and `constant settlement increment loading` methods.

5.2.6.1. Maintained Load Testing (ML). In this procedure, unless failure occurs first, the pile is loaded to 200 per cent of the anticipated pile design load for tests on individual piles or to 150 per cent of the group design load for tests on pile groups, applying the load in increments of 25 per cent of the individual pile or group design load. Each load increment is maintained until the rate of settlement is not greater than 0.25mm/h. but not longer than 2 hours. Provided that the test pile or pile group has not failed, the total test load is removed anytime after 12 hours if the butt settlement over a one-hour period is not greater than 0.25 mm.; otherwise the total load is allowed to remain on the pile or pile group for 24 hours. After the required holding time, the test load is removed in decrements of 25 per cent of the total test load with one hour between decrements. If pile failure occurs the pile is continued jacking until the settlement equals 15 per cent of the pile diameter or diagonal dimension.

Time, load, and movement readings are taken and recorded before and after the application of each load increment or the removal of each load decrement. During loading, provided that the test pile or pile group has not failed, additional readings are taken and recorded at intervals not exceeding 10 minutes during the first 30 min. nor 20 min. thereafter for each load increment. After the total load has been applied, provided that the test pile or pile group has not failed, readings are taken and recorded at intervals not exceeding 20 min. during the first two hours, not exceeding one hour for the next 10 hours, and not exceeding two hour for the next 12 hours. If pile failure occurs, readings are taken immediately before removing the first load decrement. During unloading, readings are taken and recorded at intervals not exceeding 20 min. A final rebound is taken 12 hours after all loads have been removed (ASTM D-1143).

5.2.6.2. Constant Rate of Penetration Load Testing (CRP). The applied load is varied as necessary to maintain a pile penetration rate of 0.25 to 1.25 mm/min. for cohesive soil or 0.75 to 2.5 mm/min. for granular soils. The pile is continued loading until no further increase in the load is necessary for continuous pile penetration at the specified rate unless the specified capacity of the loading apparatus is reached. If pile penetration continues, the load, which is required to achieve the specified penetration rate is hold, until the total pile penetration is at least 15 per cent of the average pile diameter or diagonal dimension at which time release the load. If the pile stops penetrating under the maximum applied load, the load is released. The rate of penetration may be controlled by checking the time taken for successive small equal increments of penetration and by adjusting the rate of jacking accordingly. Alternatively, any mechanical or electrical device may be used to monitor and control the penetration rate so that it remains constant (ASTM D-1143). In tensile testing, the hydraulic jacks shall have minimum travel of 25 per cent of the average test pile diameter, and pile is tested at an uplift rate within a range of 0.5 to 1 mm/min. The required load is hold to achieve the specified uplift rate until the total pile withdrawal is at least 200 mm or 25 per cent of the average pile diameter or diagonal dimension, whichever is less (ASTM D-3689).

Time, load, and settlement readings are taken and recorded at least every 30 seconds or at sufficient intervals to determine the rate of penetration being achieved. If automatic monitoring and recording devices are used, they are operated continuously during each test. When the test pile has achieved its specified rate of penetration, readings are continued to be taken and recorded for the duration of the loading, and the maximum load applied is determined. Immediately after unloading, readings of time, load, and rebound are taken and recorded. Final readings are taken and recorded one hour after all load has been removed (ASTM D-1143).

5.2.7. Interpretation of Test Data

Static pile load tests` main data are loads, deformations and time. Loads and deformations are recorded at least twice at each loading and unloading stage. The results are presented in load vs. deformation or time, and deformation vs. time diagrams. The most common load vs. settlement diagrams give us information on the elastic and plastic

settlements, and indications on reaction and end-bearing capacities. A typical load-settlement curve for the CRP test and a load-settlement curve for ML test are shown in Figure 5.6.

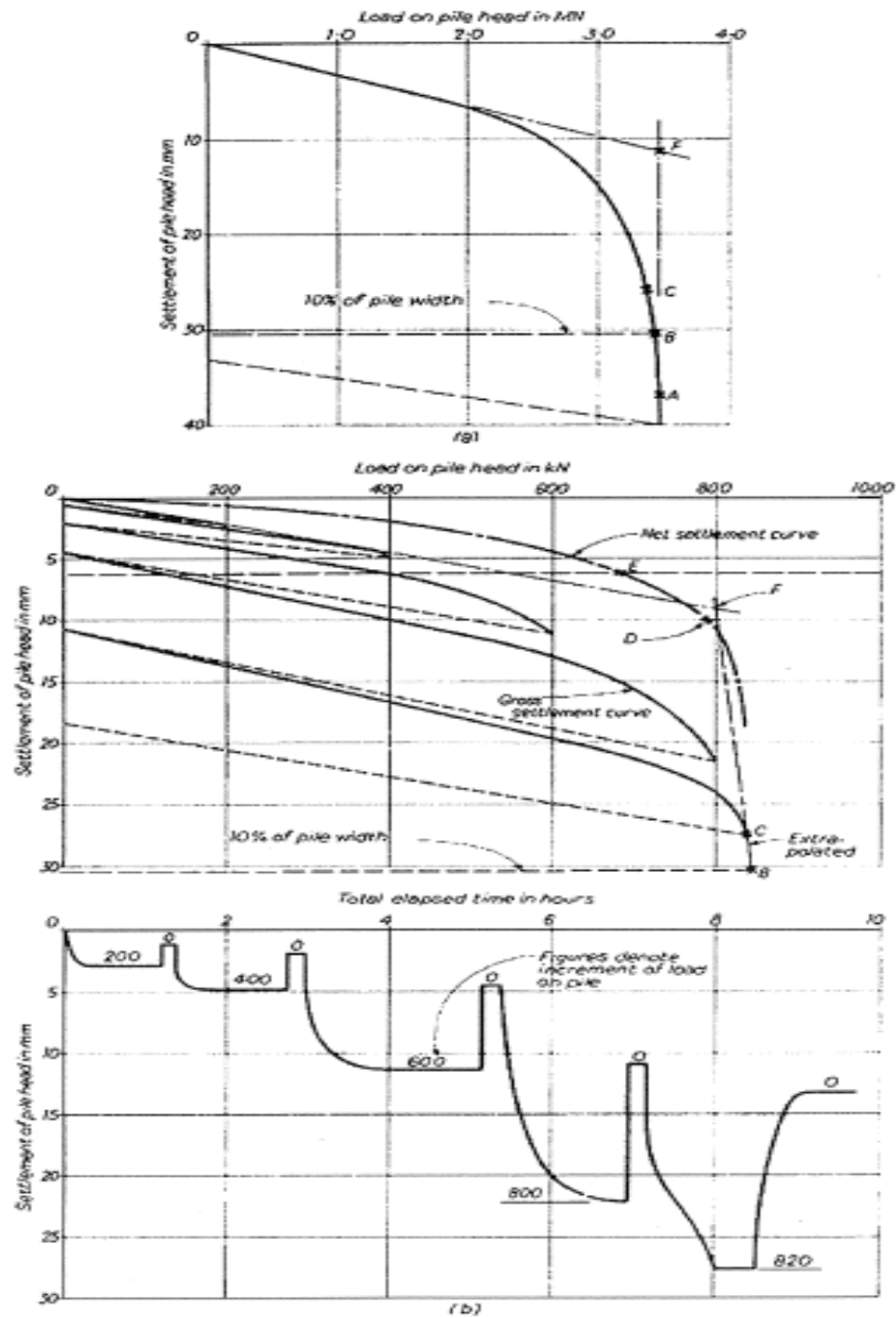


Figure 5.6. Typical load-settlement curves (Tomlinson, 2004)

The ultimate or failure load condition can be interpreted in several different ways. With reference to Figure 5.6, some of the recognized criteria for defining failure loads are listed as follows (Tomlinson, 2004):

- The load at which settlement continues to increase without any further increase of load (Point A).
- The load causing a gross settlement of 10% of the least pile width (Point B).
- The load beyond which there is an increase in gross settlement disproportionate to the increase in load (Point C).
- The load beyond which there is an increase in net settlement disproportionate to the increase of load (Point D).
- The load that produces a plastic yielding or net settlement of 6mm (Point E).
- The load indicated by the intersection of tangent lines drawn through the initial, flatter portion of the gross settlement curve and the steeper portion of the same curve (Point F).
- The load at which the slope of the net settlement is equal to 0.25 mm per 10kN of test load.

If the pile is classified as a long pile, the occurrences such as the unnegligibility of elastic shortening of the pile, the soil-pile behavior and structural considerations force the engineer to rely on more mathematical based definitions of pile capacity. As a result, mathematical method criterions described below are more realistic and widely used for defining the pile capacity (Fellenius, 2001).

- The City of New York (1985 Building Code) defines the allowable load on a pile as one-half of the load at which the net settlement after the removal of the load does not exceed 0.25 mm per 10 KN of test load where the net settlement is the gross settlement due to the test load, minus the rebound after removing 100% of the test load, or one half of the load causing a net settlement of 19 mm where the net settlement is as defined above, less the elastic shortening of the pile due to the total test load.
- Davisson Offset Limit Load method, defining the load at the intersection with the load-movement curve and a line parallel with the elastic line of the pile rising from

the movement axis at a value equal to 4 mm plus the pile diameter divided by 120. This method is based on the assumption that capacity is reached at a certain small toe movement and tries to estimate that movement by compensating for the stiffness of the pile (Fellenius, 2001).

- De Beer Yield Load method, plotting the load-movement data in a double-logarithmic diagram and observing the point where the slope of the line changes as shown in Figure 5.7. This intersection point of the two lines is called the ‘Yield Load’, showing the load of ultimate capacity (Fellenius, 2001).

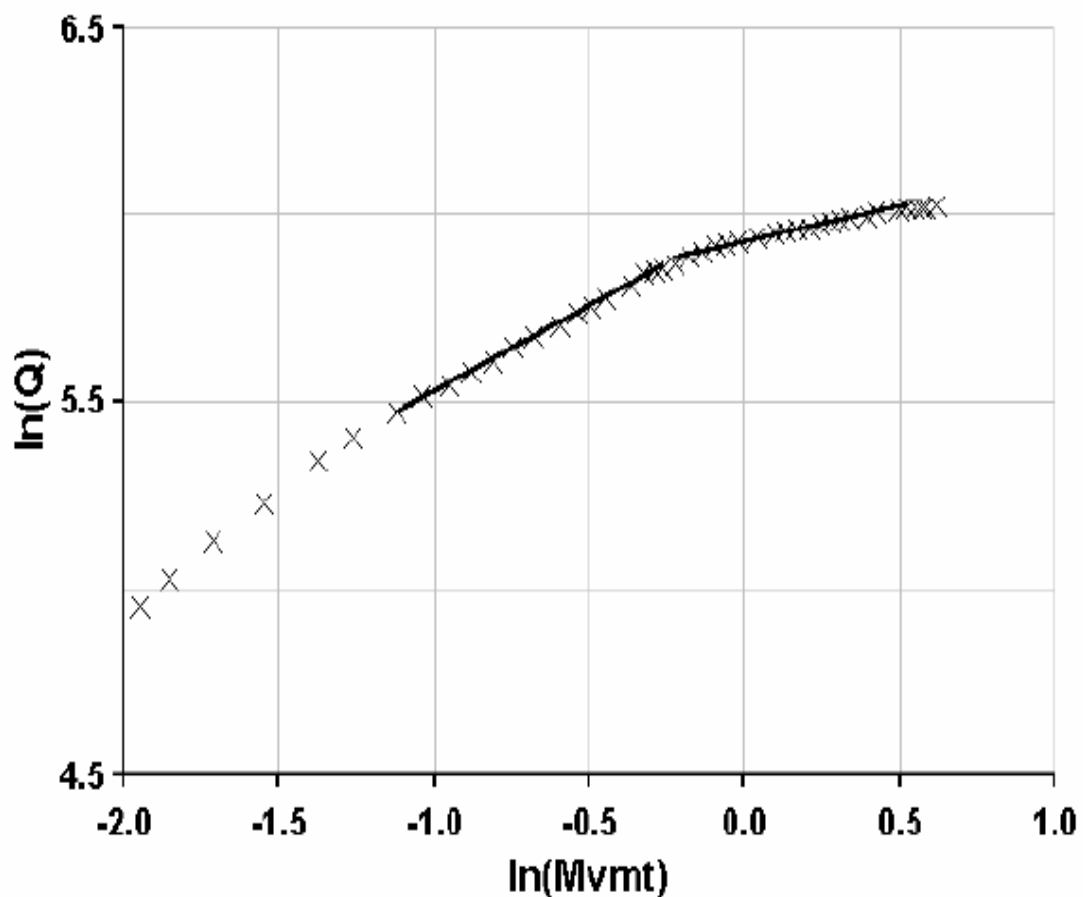


Figure 5.7. DeBeer's double logarithmic method (Fellenius, 2001)

- The Hansen 80 per cent Criterion method, proposing that the pile capacity is the load that gives four times the movement of the pile head as obtained for 80 per cent of that load. According to this method, the following simple relations can be derived for computing the capacity of ultimate resistance (Fellenius 2001):

$$Q = \frac{\sqrt{\delta}}{C_1\delta + C_2} \quad (5.1)$$

where,

Q = Applied load

δ = Movement

C_1 = Slope of the straight line

C_2 = The y-intercept of the straight line

An example curve of the Hansen 80 per cent Criterion Method is illustrated in Figure 5.8.

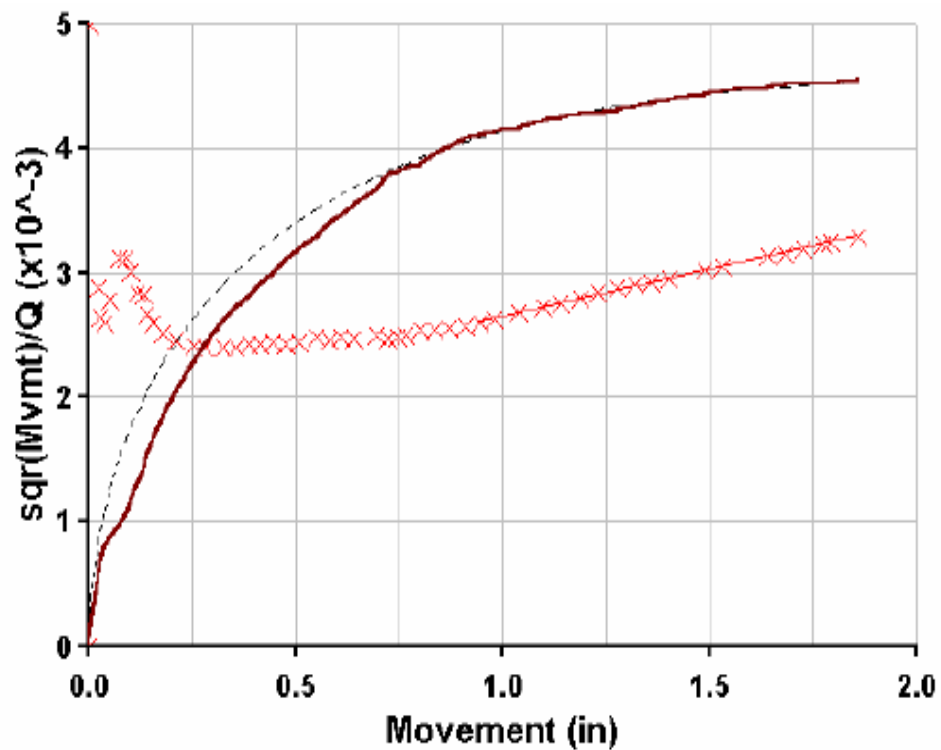


Figure 5.8. Hansen 80 per cent criterion (Fellenius, 2001)

There are also methods such as ‘Chin Kondner Criterion’ and ‘Decourt Extrapolation’ that can be used for the assessment of capacity for long piles, which give similar results with criteria described above (Fellenius, 2001).

With experience the load-settlement curve from a compression test can be used to interpret the mode of failure of a pile. A defective pile shaft is also indicated by the shape of the curve. Some typical load-settlement curves and their interpretation are shown in Figure 5.9.

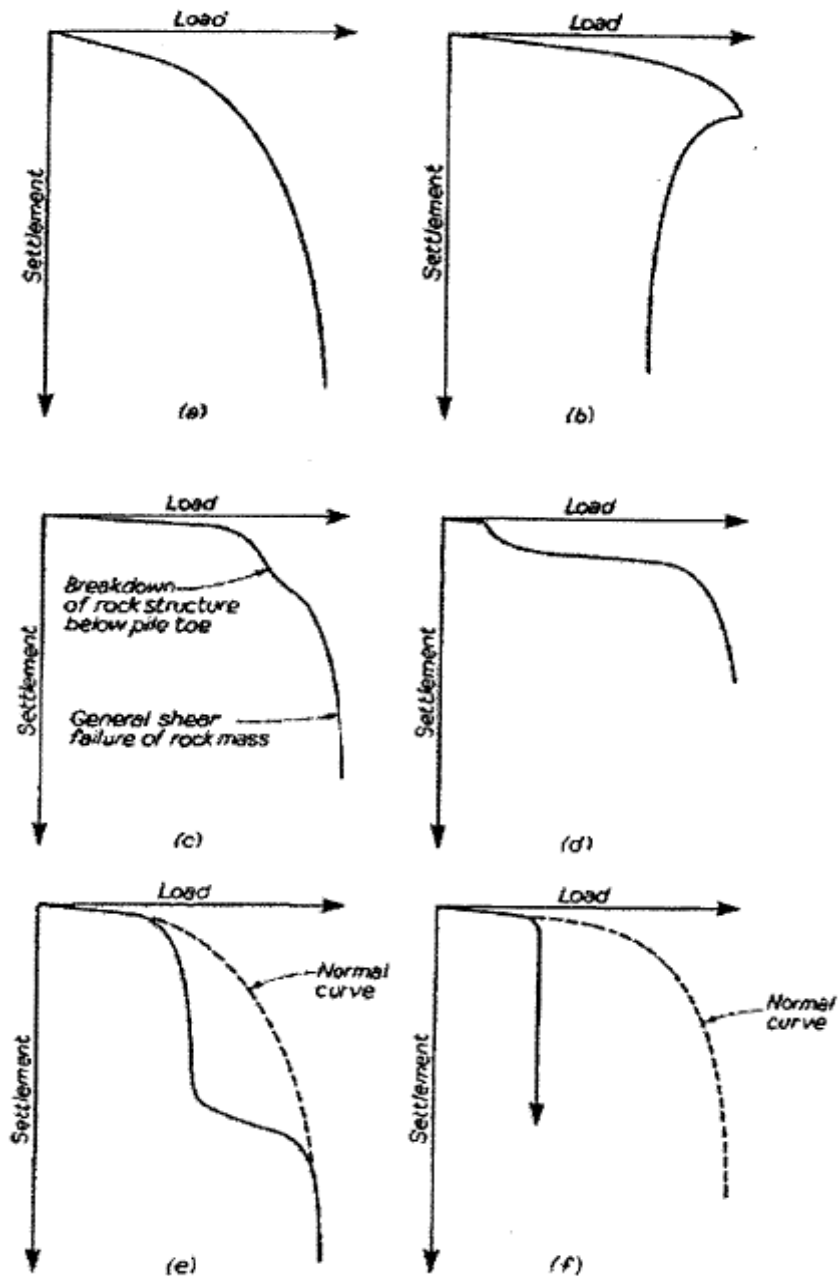


Figure 5.9. Typical load-settlement curves for compressive load tests (Tomlinson, 2004)

According to Figure 5.9 (Tomlinson, 2004);

- (a) Friction pile in soft-firm clay or loose sand
- (b) Friction pile in stiff clay
- (c) Pile end bearing on weak porous rock
- (d) Pile lifted off seting on hard rock due to soil heave and pushed down by test load to new bearing on rock
- (e) Gap in pile shaft closed up by test load
- (f) Weak concrete in pile shaft sheared completely through by test load.

If the deformations under a load level do not stabilize, the pile is deemed to have reached the failure load. As pile deformation continues for some time to establish full failure, it is hard to clearly distinguish elastic and plastic settlements of piles failed during the test (Candogan, 2008).

Fellenius (2004) mentions the effect of residual load to the interpretation of the results of a static load test in Figure 5.10. It shows the load-movement curves of a test; the lower curve was calculated by assuming no residual load present in the pile and the upper one by assuming a residual load amounting to one-third of the 'ultimate' toe resistance. When the Offset limit line ('Davisson line') was added to show the difference of the result, 20 per cent difference was found, so the importance of including residual load effect to results is proved.

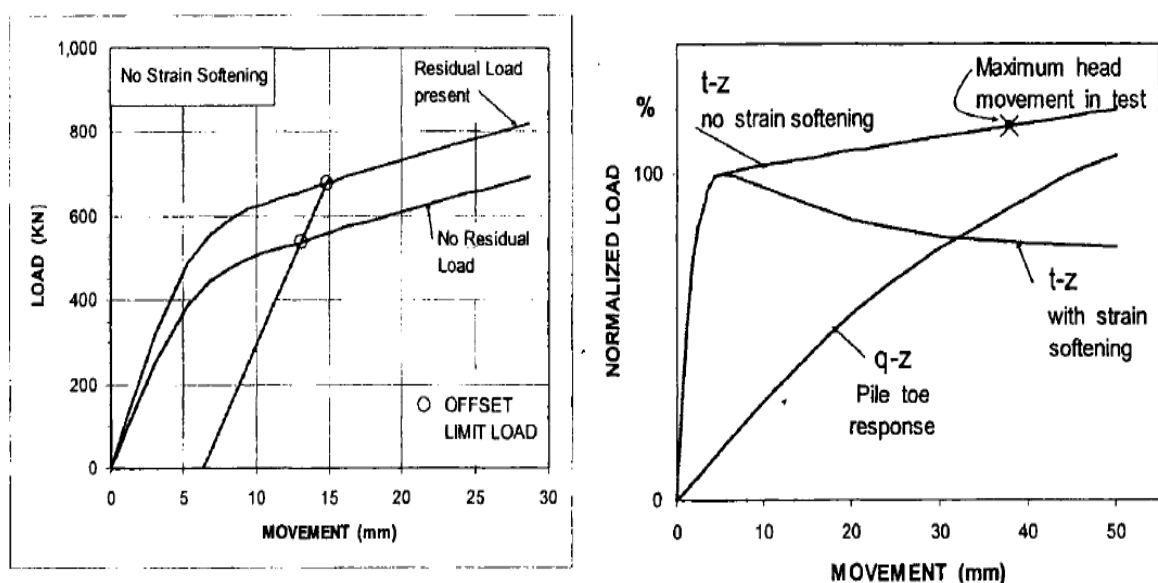


Figure 5.10. (Left) load movement curves for pile unaffected and affected by residual load and (Right) shaft and toe response t-z and q-z curves (Fellenius, 2004)

5.3. Osterberg Cell Loading Test

5.3.1. Introduction

Dr. Jorj Osterberg, Professor Emeritus at Northwestern University, developed and patented the test which now carries his name. He and Case Foundation, Inc. first used it in an experimental bored pile in 1984. By this method, full sized bored piles can be statically tested to near their ultimate capacity more conveniently, economically and safely. Generally the high capacity of bored piles, in combination with the high cost of top load systems make conventional static load testing too costly or impractical for routine testing. The Osterberg cell static load testing method, providing high capacities at affordable cost, has therefore become an attractive alternative method for testing bored piles. Because of these advantages over the conventional test systems, osterberg cell is now widely used in axial loading test of drilled shafts. The osterberg cell is a sacrificial jack like device which the engineer can have installed at the tip of a driven pile or on the reinforcement cage of a bored pile. It provides the static loading and requires no overhead frame or other external reaction system. As discussed in Section 5.2, a static load test loads the drilled shaft in compression at its top using an overhead reaction system or dead weight. Side shear and end bearing combine to resist the top load and the engineer can only separate these components approximately by analysis of strain or compression measurements together with modulus estimates. An osterberg cell test also loads the drilled shaft in compression, but from its bottom, by the help of a hydraulic system. As the osterberg cell expands, the end bearing provides reaction for the side shear, until reaching the capacity of one of the two components or until the osterberg cell reaches its capacity (Schmertmann and Hayes, 1997).

Tests using the osterberg cell automatically separate the end bearing and side shear components. When one of the components reaches ultimate capacity at an Osterberg cell load, the required conventional top load to reach both side shear and end bearing capacity would have to exceed two times that load. Thus, an osterberg cell test load placed at or near the bottom of a drilled shaft has twice the testing effectiveness of that same load placed at the top as shown in Figure 5.11.

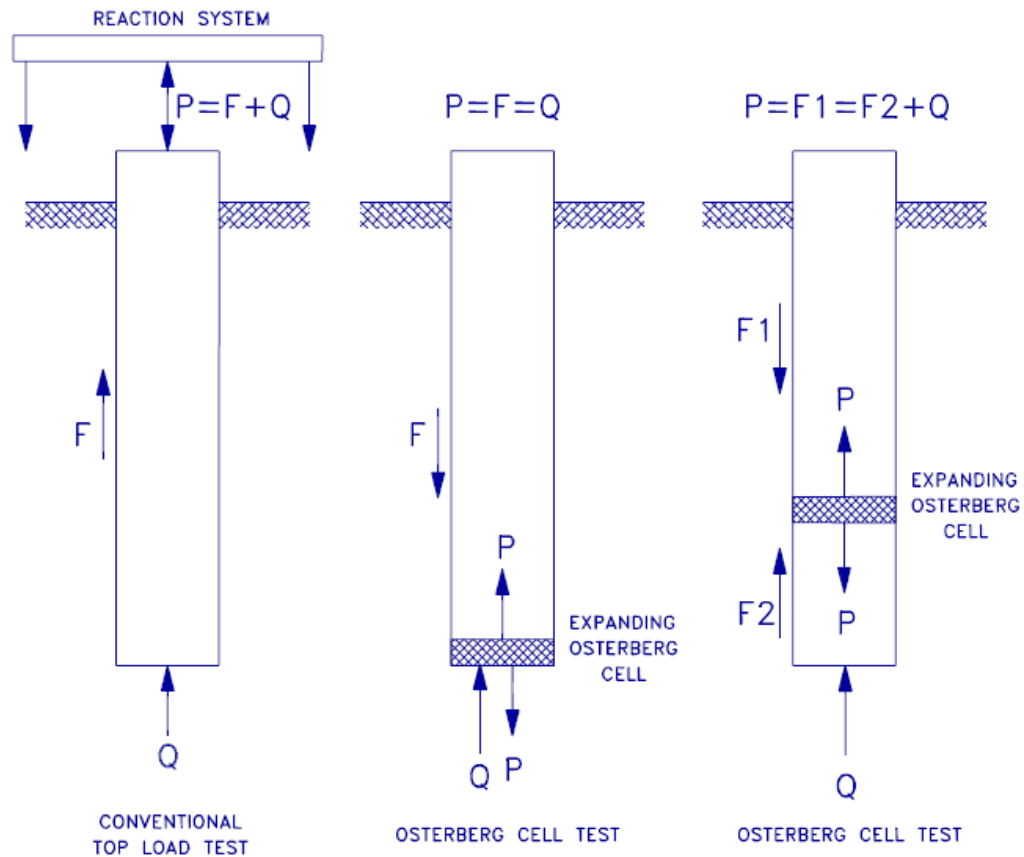


Figure 5.11. Comparison of osterberg load test with conventional top loaded test
(England, 2003)

5.3.2. Testing Equipment

For a proper testing; osterberg cell, with a desired capacity in each direction and its equipment of hydraulic lines, fittings, pressure source, pressure gage, telltale devices and vibrating wire-type strain gauges are needed. Besides, welding equipment, air compressor for pump operation during load testing, enough labor, electric power and materials sufficient to construct a stable reference beam system for monitoring movements of the shaft during testing are needed for the test.

Tell-tale extensometers with linear vibrating wire displacement transducers connected to a computerized data acquisition system are used for measurement of the movement of the bearing plates. The lower ends of the transducers attached to the bottom plate of the O-cell measure the total opening, or extension of the O-cell and the steel tell-

tales which extend to the top of the O-cell as shown as "C" and "D" in Figure 5.12 measure the upward movement of the top of the O-cell and the compression of the test pile. Subtracting the upward movement of the top of the O-cell from the total extension of the O-cell (as determined by the transducers) provides the downward movement of the pile. A high-pressure bourdon gauge and calibrated electronic pressure transducer is used to monitor the hydraulic cell pressure, from which the loading is derived by applying the calibration factor of the hydraulic cell to the pressure. The upward movement of the top of the test pile is measured with digital gages mounted on a reference beam and set over the top of the test pile as shown as "A" and "B" in Figure 5.12 (Schmertmann and Hayes, 1997).

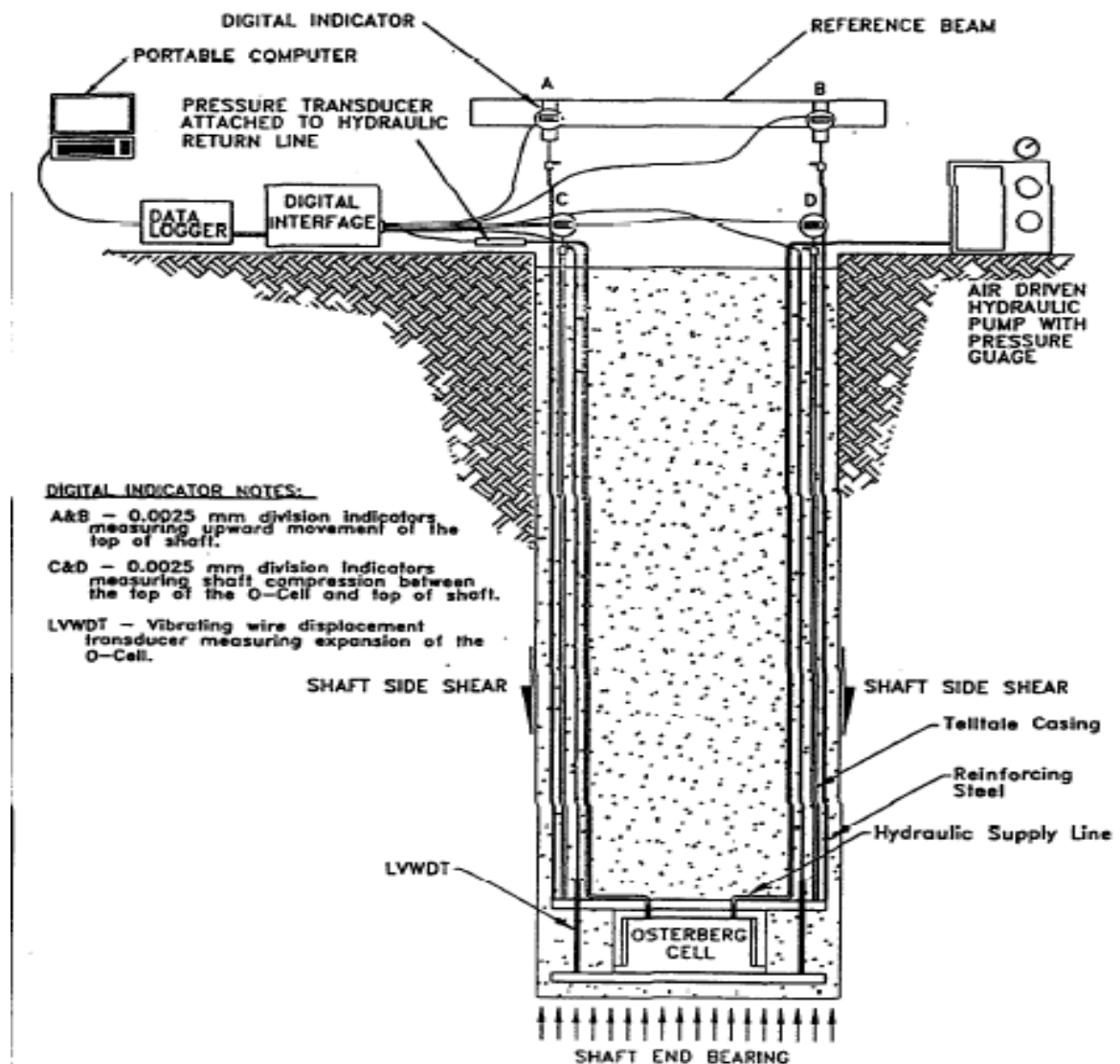


Figure 5.12. Typical test instrumentation (Schmertmann and Hayes, 1997)

5.3.3. Installation

Bored pile excavation proceeds under the bored pile contractor's work plan as approved by the engineer. Upon reaching the final toe elevation, the pile bottom is cleaned and approved by the engineer for concrete placement. The hydraulic cell assembly, related hydraulic supply, and instrumentation is lowered into the pile attached to the steel cage. The steel cage is fabricated in a number of pieces, depending on the pile length and spliced together over the bored hole. The number of cages should be kept to a minimum to speed up the installation process. The first section of the reinforcing cage containing the hydraulic cell assembly is lowered into the borehole and is temporarily supported on the steel casing. The second cage section is then lowered vertically into position and spliced to the top of the first cage. After the entire reinforcing cage has been lowered into the shaft, the cage may be supported on the steel casing during concrete placement. Utmost care should be used in handling the test equipment assembly so as not to damage the instrumentation during installation. An Osterberg test schematic is presented in Figure 5.13.

Concrete placement commences utilizing a suitable size tremmie pipe of sufficient length so as to extend beyond the hydraulic cell assembly to the toe of the pile. Cutouts of sufficient sizes are provided in the hydraulic cell steel bearing plates to accommodate the tremmie pipe. A 'funnel' is also constructed between the opening in the top plate of the hydraulic cell assembly and the main vertical rebar to guide the concrete tremmie pipe through the steel bearing plates. The funnel also serves as a means of preventing the tremmie pipe from accidentally hitting the hydraulic fittings on the cell top by forming a physical barrier apart from serving as a guide. Further protection to the hydraulic hoses is in the form of foam shields and protection bars leading from the hydraulic fittings to the cell top the cage vertical rebars which protect the hoses from the effects of flowing concrete. The concrete mix should allow for workability and should contain sufficient retarder to maintain workability for a considerable time. Reinforcing steel or steel angle iron are also welded between the top and bottom bearing plates prior to the lifting process. These temporary supports are cut out when the cage is being lowered into the hole.

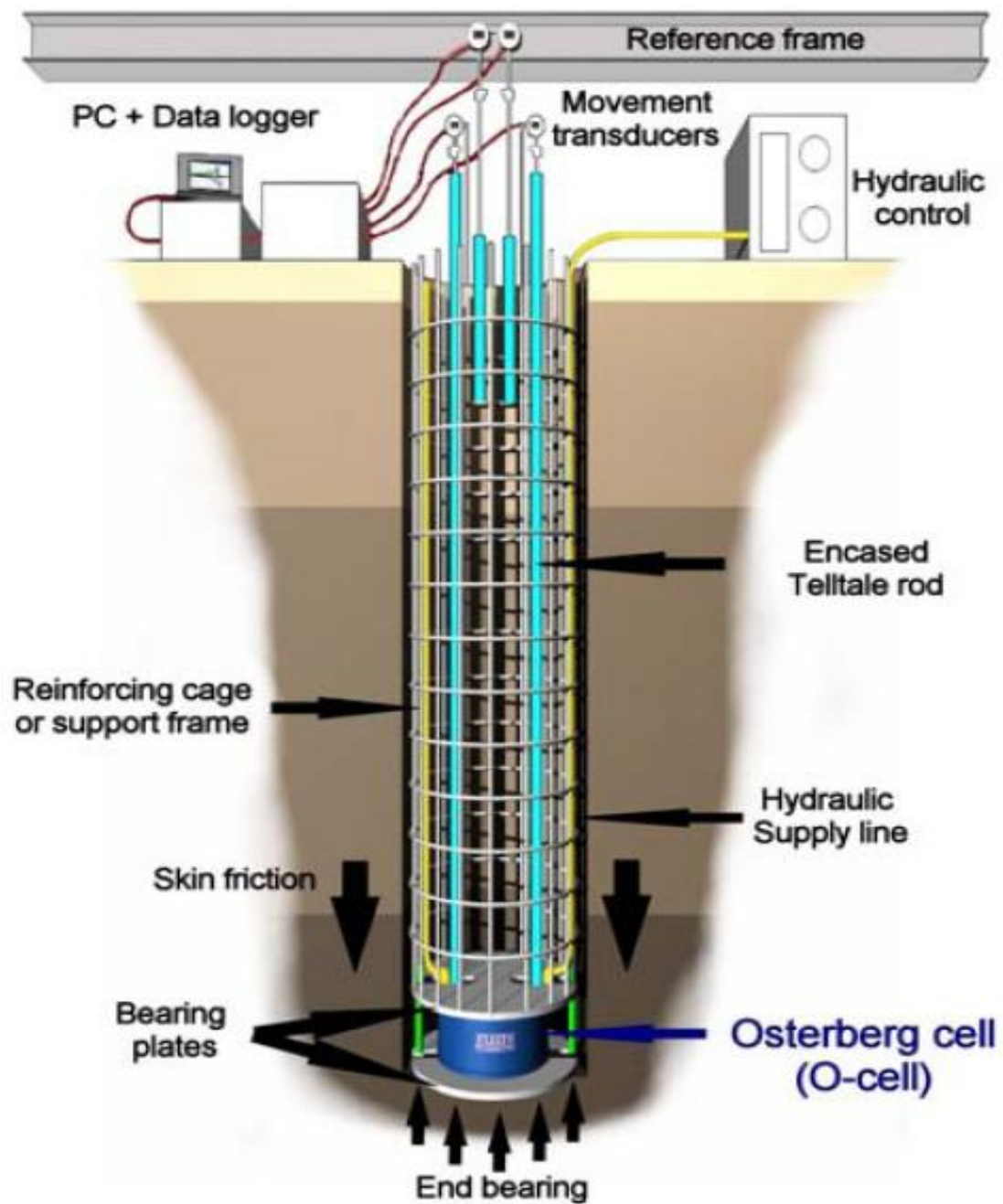


Figure 5.13. Osterberg test schematic (England, 2003)

Hydraulic cell positioning is determined based on soil data. This is used as a basis to compute the expected skin friction and end-bearing capacities of the pile. The main aim of positioning the assembly will be to equalize the bi-directional forces in the pile so that failure in one direction does not occur prematurely.

5.3.4. Testing Procedure and Measurements

The Osterberg static load test is carried out when the concrete strength of the pile is adequate to sustain the maximum required test load. Prior to commencement of the test, all hydraulic cells forming part of the assembly are pressurized to break the welded seal of the cells. Movement and pressure is recorded during this operation. The hydraulic cell is internally pressurized using a common hydraulic system which ensures uniform and synchronised pressurization of both hydraulic cells, creating an upward force on the shaft in upper friction and an equal, but downward force in combined lower shaft friction and/or end-bearing. As the pressure increases, the telltale moves downward as the load in end bearing increases and the shaft moves upward as the side shear on the shaft is mobilized. As mentioned, the hydraulic cell load is determined by relating the applied hydraulic pressure to load calibration. A high-pressure bourdon gauge will be used to read the pressure on the pump line and a calibrated pressure transducer will read the pressure on the return line. The difference between the telltale rod and first dial gage gives the compression of the concrete. From the data obtained as the load is increased, the load-upward movement curves and the load-downward movement curves can be plotted. After the test is completed, the area below the bottom of the O-cell and the cell chamber can be grouted if the pile is to be used as a working pile.

Moreover, two cells, with one at the bottom and the other at a predetermined distance up from the bottom, can be used. By this, the ultimate side shear of the pile above the upper cell, the side shear of the pile below the upper cell and the ultimate end bearing can all be determined. To accomplish this, the upper cell is pressurized to determine the side shear of the pile above the cell. With upper cell extended but not under pressure, the lower cell is pressurized to determine the side shear of the pile between the two cells. Then, with the pressure valve of the upper closed, the bottom cell is pressurized and using the entire pile in shear, the end bearing is determined. This will only work, if the end bearing ultimate strength is less than the total side shear of the pile.

The load will be removed and testing finished when one of the following conditions prevails (Osterberg, 1998):

- The test pile reaches its ultimate capacity in either the upward or downward direction (when either one of the upward or downward displacement exceeds the limit set by the Engineer – without any limits set, it is assumed to be 75 mm either direction).
- The hydraulic cell reaches its maximum loading capacity.
- The maximum travel of the cell is reached.

The loading of the load test will be carried out in stages as shown in Table 5.1.

Table 5.1. Test loading schedule (ASTM D1143-81)

Stage	Percentage of Work Load	Minimum Holding Time
1	0	0
2	25	10 min
3	50	10 min
4	75	10 min
5	100	1 hour
6	75	10 min
7	50	10 min
8	25	10 min
9	0	10 min
10	100	10 min
11	125	10 min
12	150	10 min
13	175	10 min
14	200	2 hours
15	175	10 min
16	150	10 min
17	125	10 min
18	100	10 min
19	75	10 min
20	50	10 min
21	25	10 min
22	0	10 min

The load shall be sustained at a constant magnitude until the rate of movement of the pile head is less than the lesser of:

- 0.25 mm per 10 minutes.
- One per cent of the total pile movement per 10 minutes.

The movement of the pile is closely monitored and the displacement transducers are adjusted if the movements exceed 100 mm. This is in keeping with the normal practise in maintained load tests where pile movements exceed the range of the displacement measurement instruments. Pile compression under load will be measured directly at the top of the shaft using displacement transducers and non-encased telltale rods attached to the pile top. The displacement transducers and pressure gauge are connected to a pile sentinel data logger or equivalent equipment. The data logger is, in turn, connected to a laptop computer. This arrangement allows displacement transducers and pressure gauge to be recorded and stored automatically at 1-minute intervals during the test. All measurements are made with reference to a reference frame constructed at platform level.

5.3.5. Interpretation of Test Data

Osterberg cell test provides separate load-deflection curves for side shear and end bearing so it is possible to determine at each increment of load how much of the total load is in side friction and how much is in end bearing. As the deflection of the shaft is small when the ultimate load is reached in side shear compared to the deflection at which the ultimate end bearing is reached, the majority of the load is taken in shear as the load is applied and shifts to end bearing as the load is increased. Besides, the ultimate side shear is larger than design values in most tests and if the side shear is low, it is usually due to construction methods. Therefore, the working load applied to the top of the pile is mostly resisted by the side shear and little would reach the bottom, which can be used as a more economical design for a bored pile.

5.3.5.1. Construction of the Top Down Equivalent Curve. Osterberg static load test can provide a good estimate of a curve showing the load versus settlement of a top-loaded bored pile with the following assumptions, which consider good sense and conservative (Osterberg,1994):

- The end bearing load-movement curve in a top loaded shaft has the same loads for a given movement as the net (subtract buoyant unit weight of pile above hydraulic jack) end bearing load-movement curve developed by the bottom of the hydraulic jack when placed at or near the bottom of the shaft.
- The side shear load-movement curve in a top loaded shaft has the same net shear, multiplied by an adjustment factor 'F' for a given downward movement as occurred in the Osterberg test for that same movement at the top of the jack in the upward direction. The same applies to the upward movement in a top-loaded tension test. Unless noted otherwise, a factor $F=0.95$ for compression in cohesionless soils and $F=0.80$ for tension tests in all soils is used.
- The pile behaves as a rigid body, but include the elastic compressions that are part of the movement data obtained from an osterbergl static load test. Procedure 1 interprets an equivalent top-load test movement curve and procedure 2 corrects the effects of the additional elastic compressions in a top loaded test.
- The part of the shaft below the hydraulic jack (one or multi level) has the same load-movement behavior as when top-loading the entire shaft. The subsequent 'end bearing movement curve' refers to the movement of the entire length of shaft below the jack.

For an osterberg cell load test, the side resistance versus side movement curve and the base resistance versus base movement curve can be used to construct the load-displacement curve equivalent to applying the load at the top of the shaft (Osterberg, 1999). This is done by determining the side resistance at an arbitrary displacement point on the side resistance versus side movement curve. If the shaft is assumed rigid, the top and bottom of the shaft move the same amount and have the same displacement but different loads. By adding the side resistance and the base resistance at the same displacement, a single point on the top equivalent load-displacement curve is obtained. By repeating this process for different displacement points, the top equivalent load-displacement curve can

be obtained. As shown in Figure 5.14. It is very often that the side shear or the end bearing reaches ultimate before the other. In this case, two procedures are possible. The first, which is extremely conservative, is to assume that the component that has not reached ultimate has also reached ultimate and that no further load increase occurs as displacement increases. The other more likely procedure, as shown in Figure 5.14, is to extrapolate the curve that has not reached ultimate. Suitable hyperbolic curve fitting technique can be used for extrapolation of the side shear curve to produce end bearing movement data. Using the same movement matching procedure, the equivalent curve can also be extended. The dashed line on Figure 5.14 signify that this part of the equivalent curve depends partly on extrapolated data (Osterberg, 1998).

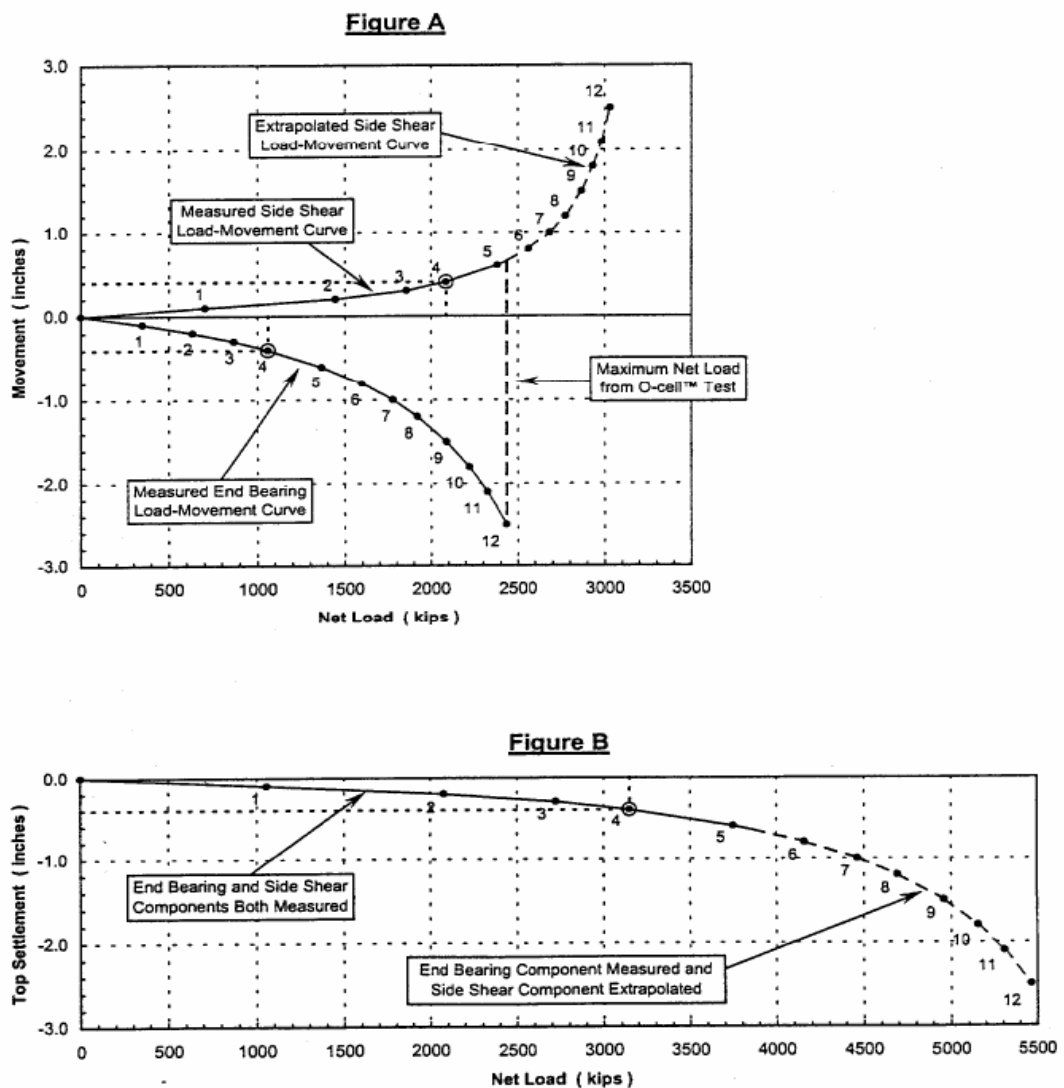
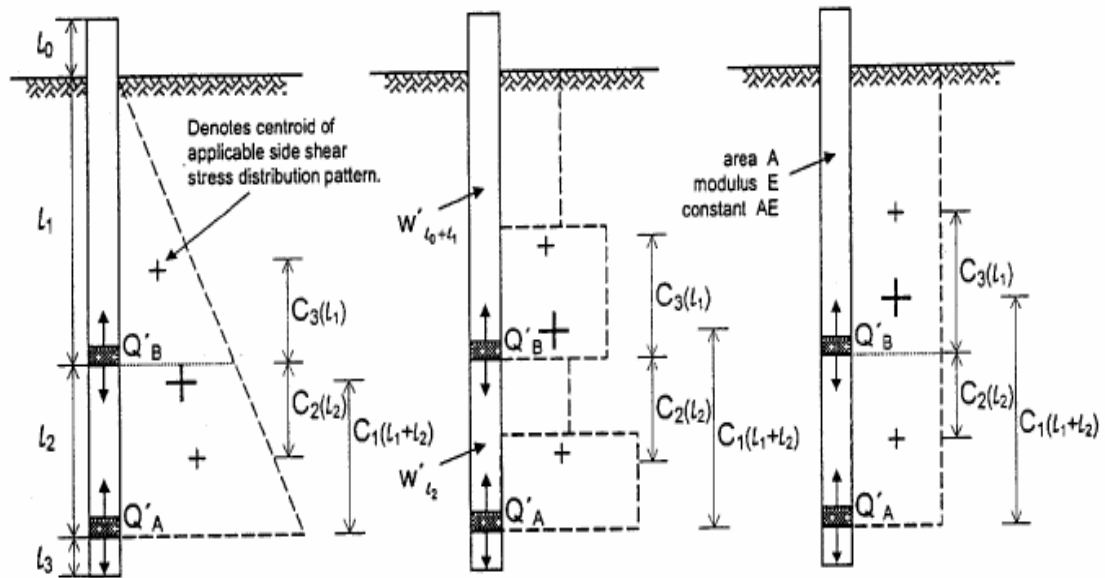


Figure 5.14. Example of the construction of an equivalent top-loaded settlement curve (Figure B) from osterberg cell test results (Figure A) (Osterberg, 1998)

The elastic compression in the equivalent top load static test always exceeds that in the osterberg test. It produces more top movement and also additional side shear movement, which then generate more side shear, more compression, etc. An exact solution of this load transfer problem requires knowing the side shear vs. vertical movement (t-y) curves for a large number of pile length increments and solving the resulting set of simultaneous equations or using finite element or finite difference simulations to obtain an approximate solution for these equations (Osterberg, 1998).

The attached analysis in Figure 5.15 gives the equations for the elastic compressions that occur in the osterberg load test (OLT) with one or two levels of hydraulic jacks. Analysis in Figure 5.16 gives the equations for the elastic compressions that occur in the equivalent top loading static test (TLT). Both sets of equations do not include the elastic compression below the hydraulic jack because the same compression takes place in both the OLT and the TLT. This is equivalent to taking $I_3 = 0$. Subtracting the OLT from the TLT compression gives the desired additional elastic compression at the top of the TLT. The additional elastic compression is then added to the rigid equivalent curve obtained from the equivalent top loaded settlement curve to obtain the final, corrected equivalent load-settlement curve for the TLT on the same pile as the actual OLT.

The analysis in Figure 5.17 provides an example of calculated results on a hypothetical one stage, single level OLT using the method described above with the centroid of the side shear distribution 44,1 per cent above the base of the hydraulic jack. The analysis in Figure 5.18 provides an example of calculated results on a hypothetical three stage, multi level OLT using the simplified method described above with the centroid of the combined upper and middle side shear distribution 44,1 per cent above the base of the bottom hydraulic jack. The individual centroids of the upper and middle side shear distribution lie 39,6 per cent and 57,9 per cent above and below the middle hydraulic jack, respectively. Figures also include comparison of the corrected with the rigid curve.



1-Stage Single Level Test (Q'_A only): $\delta_{OLT} = \delta_{T(l_1+l_2)}$

$C_1 = \frac{1}{3}$	Centroid Factor = C_1	$C_1 = \frac{1}{2}$
$\delta_{T(l_1+l_2)} = \frac{1}{3} \frac{Q'_{TA}(l_1+l_2)}{AE}$	$\delta_{T(l_1+l_2)} = C_1 \frac{Q'_{TA}(l_1+l_2)}{AE}$	$\delta_{T(l_1+l_2)} = \frac{1}{2} \frac{Q'_{TA}(l_1+l_2)}{AE}$

3-Stage Multi Level Test (Q'_A and Q'_B): $\delta_{OLT} = \delta_{Tl_1} + \delta_{\downarrow l_2}$

$C_3 = \frac{1}{3}$	Centroid Factor = C_3	$C_3 = \frac{1}{2}$
$\delta_{Tl_1} = \frac{1}{3} \frac{Q'_{TB}l_1}{AE}$	$\delta_{Tl_1} = C_3 \frac{Q'_{TB}l_1}{AE}$	$\delta_{Tl_1} = \frac{1}{3} \frac{Q'_{TB}l_1}{AE}$
$C_2 = \frac{1}{3} \left(\frac{3l_1+2l_2}{2l_1+l_2} \right)$	Centroid Factor = C_2	$C_2 = \frac{1}{2}$
$\delta_{\downarrow l_2} = \frac{1}{3} \left(\frac{3l_1+2l_2}{2l_1+l_2} \right) \frac{Q'_{\downarrow B}l_2}{AE}$	$\delta_{\downarrow l_2} = C_2 \frac{Q'_{\downarrow B}l_2}{AE}$	$\delta_{\downarrow l_2} = \frac{1}{2} \frac{Q'_{\downarrow B}l_2}{AE}$

Net Loads:

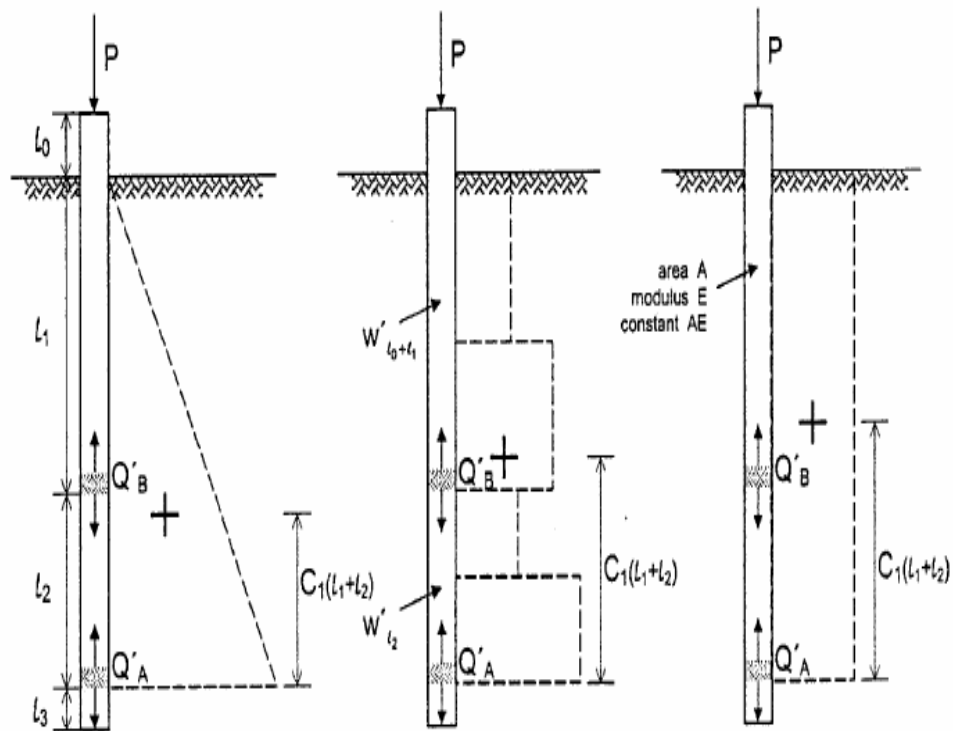
$$Q'_{TA} = Q_{TA} - W'_{l_0+l_1+l_2}$$

$$Q'_{TB} = Q_{TB} - W'_{l_0+l_1}$$

$$Q'_{\downarrow B} = Q_{\downarrow B} + W'_{l_2}$$

W' = pile weight, bouyant where below water table

Figure 5.15. Theoretical elastic compression in O-cell test based on pattern of developed side shear stress (Osterberg, 1998)



Top Loaded Test: $\delta_{TLT} = \delta_{\downarrow l_0} + \delta_{\downarrow l_1+l_2}$

$\delta_{\downarrow l_0} = \frac{Pl_0}{AE}$	$\delta_{\downarrow l_0} = \frac{Pl_0}{AE}$	$\delta_{\downarrow l_0} = \frac{Pl_0}{AE}$
$C_1 = \frac{1}{3}$	Centroid Factor = C_1	$C_1 = \frac{1}{2}$
$\delta_{\downarrow l_1+l_2} = \frac{(Q'_{\downarrow A} + 2P)(l_2 + l_3)}{3 AE}$	$\delta_{\downarrow l_1+l_2} = \frac{[(C_1)Q'_{\downarrow A} + (1-C_1)P](l_1 + l_2)}{AE}$	$\delta_{\downarrow l_1+l_2} = \frac{(Q'_{\downarrow A} + P)(l_1 + l_2)}{2 AE}$

Net and Equivalent Loads:

$$Q'_{\downarrow A} = Q_{\downarrow A} - W'_{l_0+l_1+l_2}$$

$$P_{\text{single}} = Q'_{\downarrow A} + Q'_{\uparrow A}$$

$$P_{\text{multi}} = Q'_{\downarrow A} + Q'_{\uparrow B} + Q'_{\downarrow B}$$

Component loads Q selected at the same ($\pm$) Δ_{OLT} .

Figure 5.16. Theoretical elastic compression in top loaded test based on pattern of developed side shear stress (Osterberg, 1998)

Given: $C_1 = 0.441$
 $AE = 17000 \text{ MN}$ (assumed constant throughout test)
 $l_0 = 1.80 \text{ m}$
 $l_1 = 14.69 \text{ m}$ (embedded length of shaft above O-cell™)
 $l_2 = 0.00 \text{ m}$
 $l_3 = 0.00 \text{ m}$
 Shear reduction factor = 1.00 (cohesive soil)

Δ_{OLT} (mm)	Q'_{LA} (MN)	Q'_{TA} (MN)	P (MN)	δ_{TLT} (mm)	δ_{OLT} (mm)	Δ_s (mm)	$\Delta_{OLT} + \Delta_s$ (mm)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.54	1.57	3.14	4.71	3.37	1.20	2.17	4.71
5.08	2.82	6.43	9.25	6.52	2.45	4.07	9.15
7.62	3.86	8.27	12.12	8.61	3.15	5.46	13.08
10.16	4.72	9.29	14.01	10.05	3.54	6.51	16.67
15.24	6.08	10.60	16.68	12.14	4.04	8.10	23.34
20.32	7.11	11.40	18.50	13.60	4.34	9.26	29.58
25.40	7.90	11.94	19.85	14.70	4.55	10.15	35.55
30.48	8.55	12.33	20.88	15.55	4.70	10.85	41.33
38.10	9.30	12.75	22.05	16.53	4.86	11.67	49.77
45.72	9.88	13.05	22.93	17.27	4.97	12.29	58.01
53.34	10.34	13.27	23.61	17.84	5.06	12.79	66.13
63.50	10.83	13.48	24.31	18.44	5.14	13.30	76.80

Figure D

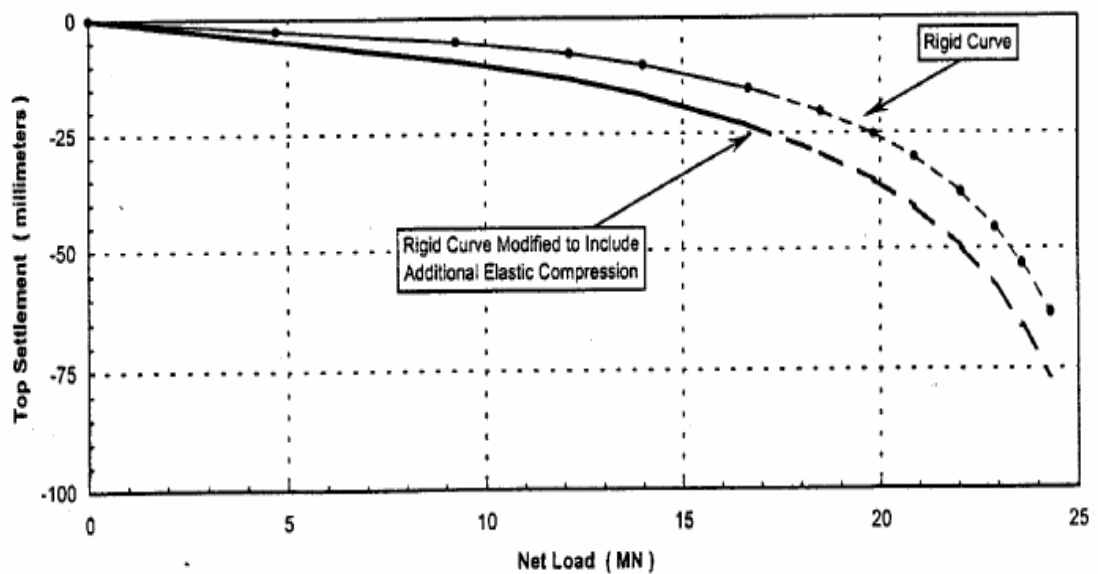


Figure 5.17. Example calculation for the additional elastic compression correction for single level test (Osterberg, 1998)

Given:

$C_1 = 0.441$
 $C_2 = 0.579$
 $C_3 = 0.396$
 $AE = 17000 \text{ MN}$ (assumed constant throughout test)
 $l_0 = 1.80 \text{ m}$
 $l_1 = 9.14 \text{ m}$ (embedded length of shaft above mid-cell)
 $l_2 = 5.55 \text{ m}$ (embedded length of shaft between O-cells™)
 $l_3 = 0.00 \text{ m}$
 Shear reduction factor = 1.00 (cohesive soil)

Δ_{OLT} (in)	Q'_{1A} (kips)	Q'_{1B} (kips)	Q'_{TB} (kips)	P (kips)	δ_{TLT} (in)	δ_{OLT} (in)	Δ_s (in)	$\Delta_{OLT} + \Delta_s$ (in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.54	1.57	1.10	2.04	4.71	3.37	0.64	2.73	5.27
5.08	2.82	2.25	4.18	9.25	6.52	1.31	5.21	10.29
7.62	3.86	2.89	5.37	12.12	8.61	1.69	6.92	14.54
10.16	4.72	3.25	6.04	14.01	10.05	1.90	8.15	18.31
15.24	6.08	3.71	6.89	16.68	12.14	2.17	9.97	25.21
20.32	7.11	3.99	7.41	18.50	13.60	2.33	11.27	31.59
25.40	7.90	4.18	7.76	19.85	14.70	2.44	12.26	37.66
30.48	8.55	4.32	8.02	20.88	15.55	2.52	13.03	43.51
38.10	9.30	4.46	8.29	22.05	16.53	2.61	13.92	52.02
45.72	9.88	4.57	8.48	22.93	17.27	2.67	14.60	60.32
53.34	10.34	4.64	8.62	23.61	17.84	2.71	15.13	68.47
63.50	10.83	4.72	8.76	24.31	18.44	2.76	15.68	79.18

Figure F

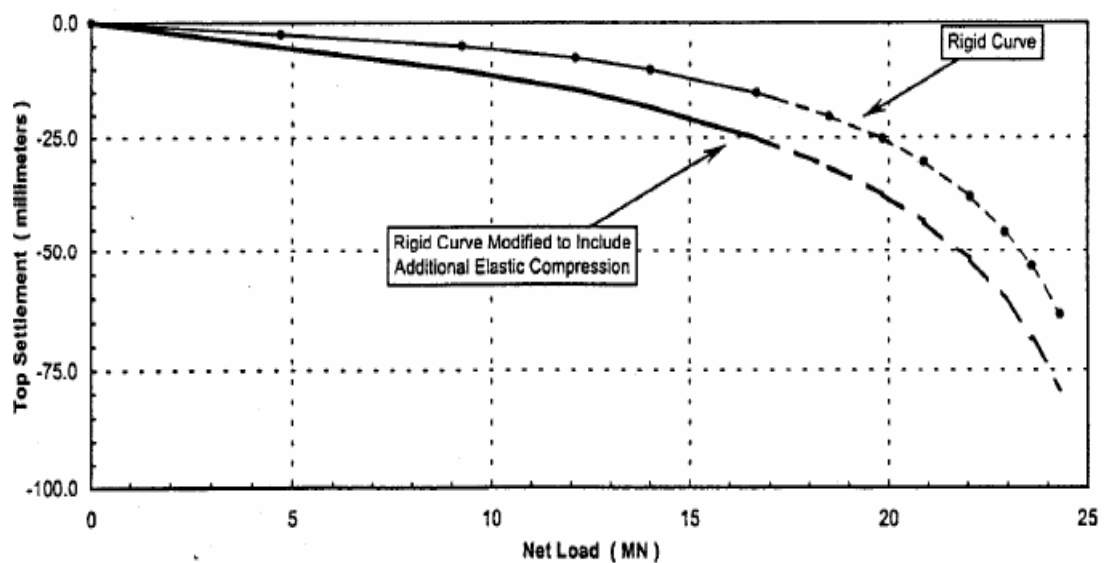


Figure 5.18. Example calculation for the additional elastic compression correction for multi level test (Osterberg, 1998)

5.3.5.2. Typical test curves.

Figure 5.19 shows an example test result in which the ultimate load in end bearing is reached, meaning that if the working load is less than 1350 tons all the load is taken by side shear.

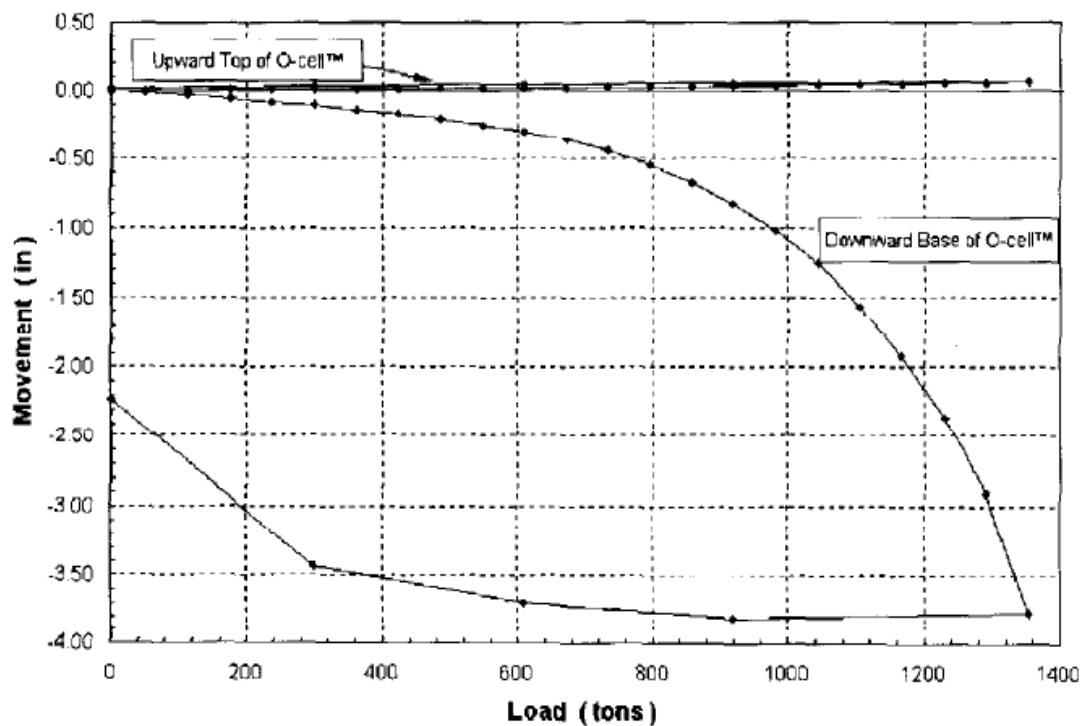


Figure 5.19. Example osterberg cell load-movement curve when ultimate load is reached in end bearing (Osterberg, 2004)

Figure 5.20 shows an example test curve in which the ultimate load in side shear is reached, as typically seen in clay soils and some rocks. It is seen that when the ultimate in side shear is reached, continuous movement occurs at instant load. When a drilled shaft is constructed on a rock socket, usually the ultimate side shear strength is higher than design values because in the field the rock is subjected to the weight of overburden and the higher normal pressure makes the shear strength between the laminations increase. Figure 5.21 shows the load - upward movement and load - downward movement curves for a drilled shaft where the design load is 500 tons and the test is made to 6000 tons. However, although the capacity of the O-cell was reached so the test discontinued, neither the ultimate load in shear or end bearing is reached. (Osterberg, 1994).

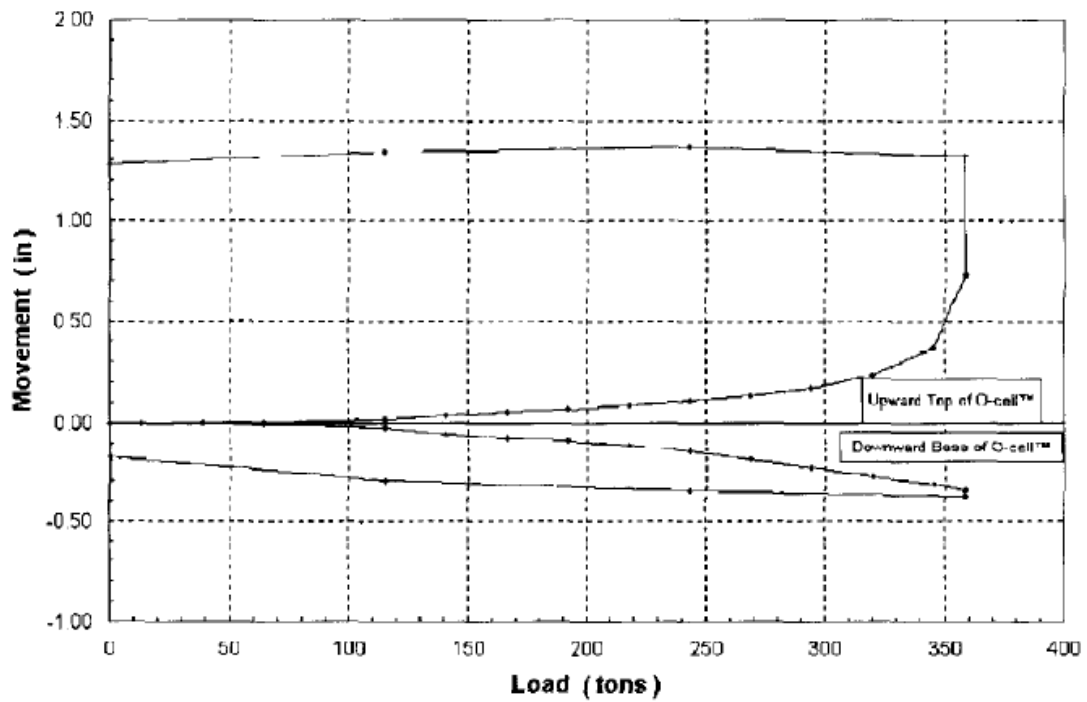


Figure 5.20. Example osterberg cell load-movement curve when ultimate load is reached in side shear (Osterberg, 2004)

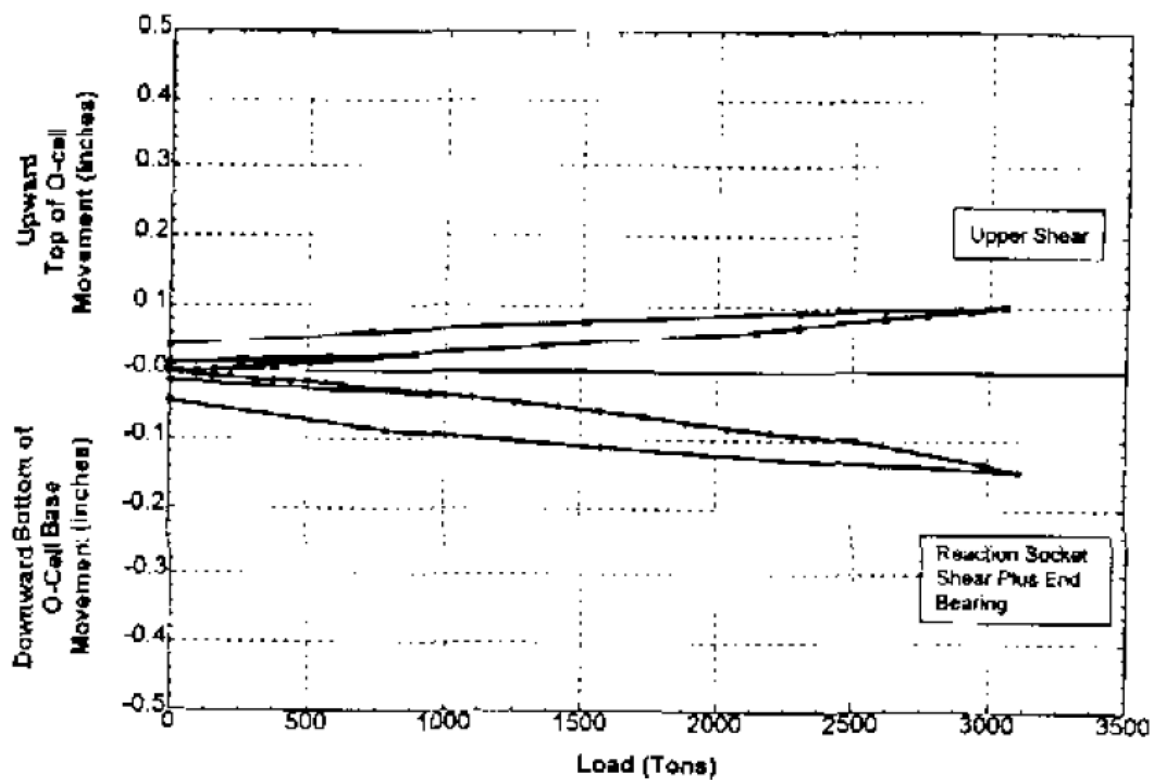


Figure 5.21. Load upward and downward movement curves for a drilled shaft (Osterberg, 2004)

5.3.6. The Role of Construction Technique in Osterberg Load Testing

The techniques of bored pile construction play an important part in subsequent load capacity, and an apparently minor change in construction technique can have a major impact on one or both components of pile capacity. Generally, poor cleanout procedures for the pile bottom (dry hole method) decreases end bearing capacity and improper use of drilling tools, poor construction procedures have negative impact on side shear.

Drilling tool with oscillating circular motion may polish the side wall of the rock, so decrease the side shear. In some soils cleanout buckets and too fast drilling may also decrease side shear, disturbing the hole. Moreover, allowing the hole to remain too long time without desanding will cause the stabilizing slurry, such as bentonite to create a mudcake on sides or bottom of hole, resulting a decrease in side shear. Figure 5.22 shows an example curve of improper use of drilling tools (Osterberg, 2004).

End bearing capacity of the piles are majorly effected by unadequate cleaning of pile bottoms from disturbed material. If the pile bottom is not properly cleaned, during testing the O-cell first encounter the disturbed material and considerable downward movement can occur to fully compress the material and transfer the load to the undisturbed material below. So, a common observation for bored piles is that a second circle of downward loading will yield a much smaller deflection, indicating that the first cycle precompresses the material for subsequent loads including the design load. Figures 5.23 and 5.24 show the effects of poor cleanout procedures in various soils. In Figure 5.23, the O-cell could not reach the undisturbed material so results are unrealisti. In Figure 5.24, O-cell test results show that due to poor concreting test results are poor in the first loading stage but remedial test has good results since O-cell reaches undisturbed material in second stage.

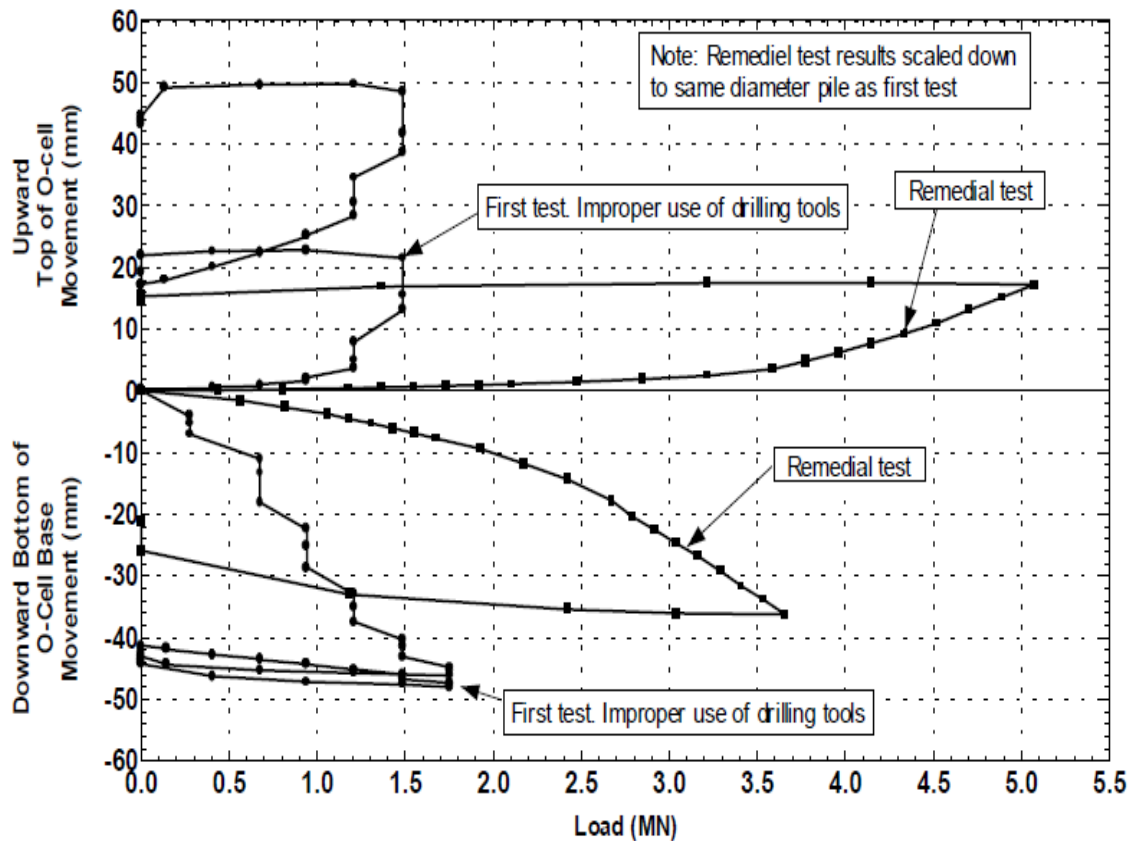


Figure 5.22. Test curves of improper use of drilling tools (Osterberg, 2004)

As a result, osterberg cell method is better than top loading static test method in revealing the improper construction defects on bored piles, especially the uncleaned hole bottoms, because in top loading test usually the side shear takes most of the load and very little of the load reaches the bottom. Besides, this method shows us that if the pile is constructed properly, the measured pile capacity becomes greater than the estimated value, which helps the designer to design more economically.

25 tests were investigated in which there was enough information regarding the strength of the supporting medium and the engineer's estimated capacity. It was found that the ratio of the measured to estimated capacity (M/E) tends to increase as the strength of the supporting medium increases as shown in Figure 5.25. So, the harder the medium the pile is in, the more the load capacity of the pile is underestimated (Schmertmann, 2004).

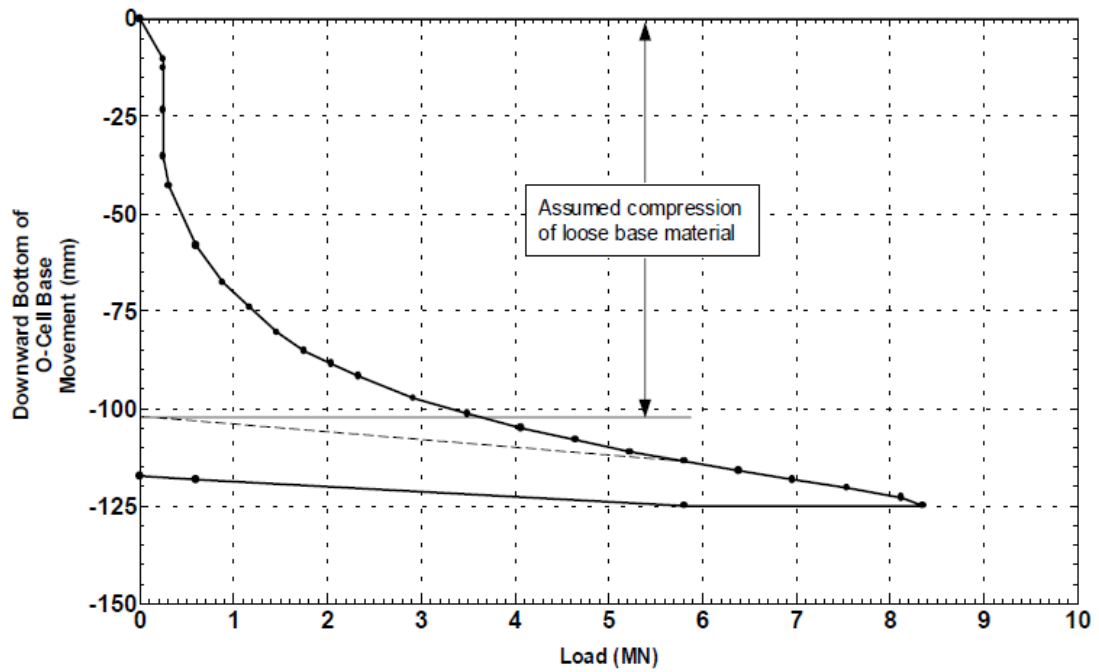


Figure 5.23. An example poor O-cell test result due to compression of loose material (Osterberg, 2004)

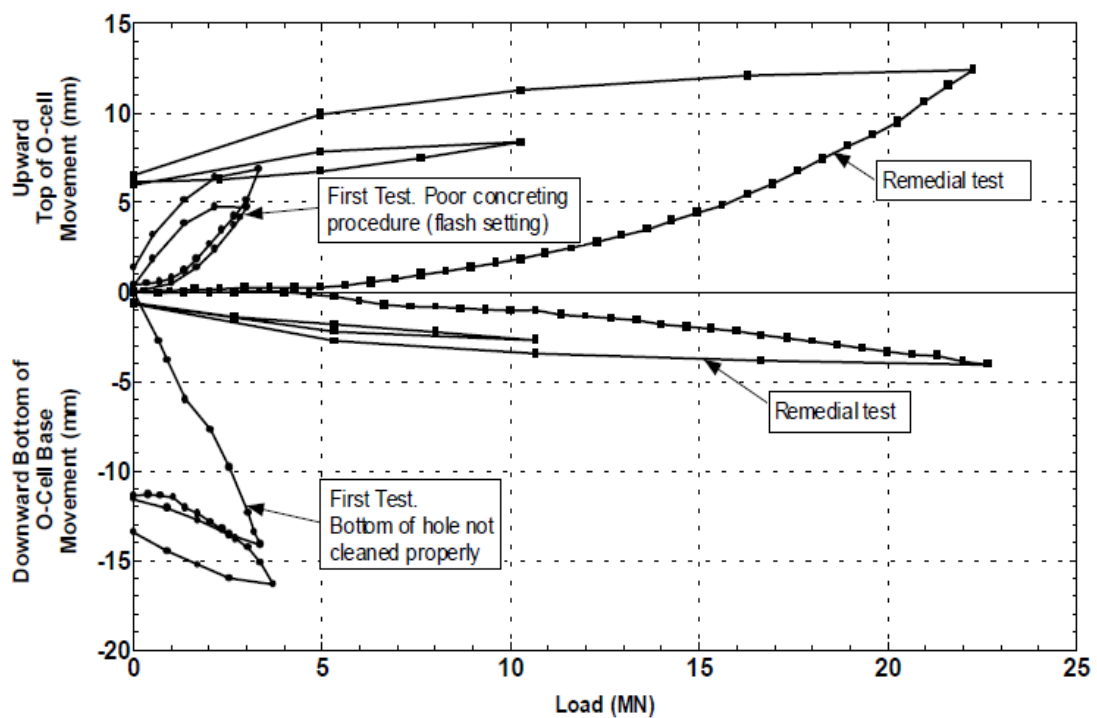


Figure 5.24. Poor test results due to loose material at pile toe (Osterberg, 2004)

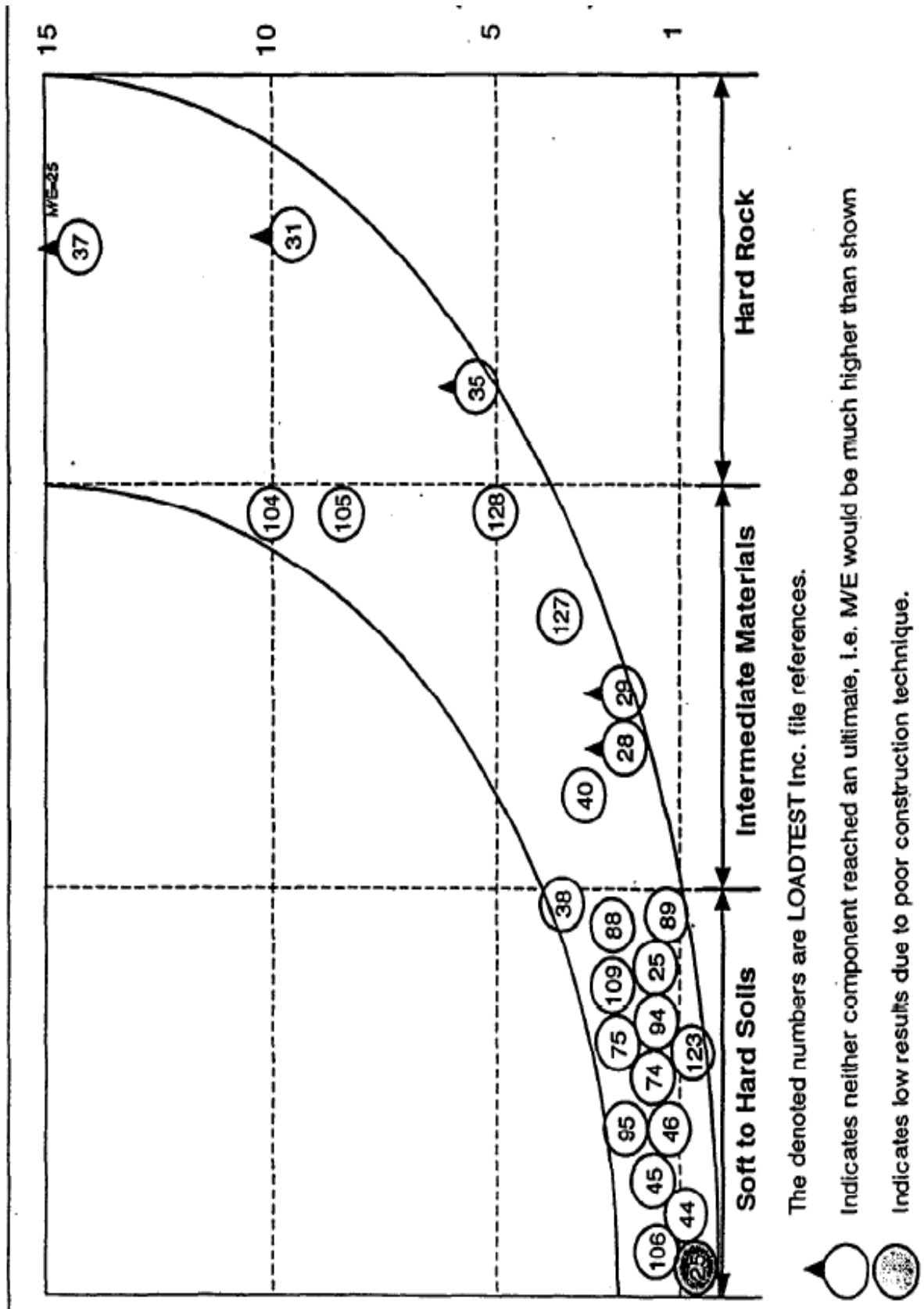


Figure 5.25. Ratio of measured to estimated ultimate loads (Schmertmann, 2004)

5.3.7. Advantages and Limitations when Compared with Conventional Static Testing

Static top down loading and osterberg tests were compared and the advantages and disadvantages of O-cell testing in terms of mobilized loads, evaluating skin friction in rock sockets, applied stress, test pile selection, testing procedure, interpretation of results and estimated costs were discussed in various studies. First, a top down load test is not pratic for high loads, which are more than 10 –20 MN, as transportation and installation of the system is hard, needs more space, uneconomical and time-consuming. Next, both single level and multi level load applications might be used in O-cell testing to control the maximum load by the extension of the jack, which provides more detailed pile capacity data. Then, skin friction and end bearing capacity of a bored pile might not be fully determined by a top-down load testing, expacially in rock sockets so O-cell testing give more realistic results as the load is directly applied to rock socket. Tested piles might be used as a working pile in both test methods, provided that the pile meets any differential settlement requirments. O-cell tested piles, however, need fully grouting in and around the cell. One of the main differences in these test methods is that O-cell test only applies half of the stress to the pile than top-loading test to maintain a ultimate capacity, making the test faster but the structural performance of pile top remains untested. Inversely, in top loading test more time is needed for load transfer along the length of the pile. Usually, osterberg testing method is preferred for preliminary testing as it is fast and give more realistic data for design parameters. In proof tests main advantage of this test is its working area availability. At testing sequence, both testing methods use most up to date computerized system however osterberg testing instrumentation is higher as two load test (up and down) are monitored at same time. The interpretation of test results are more clear for Osterberg test; as stated before, not only the seperate behaviour of skin friction and end bearing mobilized are more informative than Fleming (1992)`s interpretation method but also assesment of the impact of construction technique is easier (England, 2003 and Schmertmann, 1998).

When Osterberg test results are compared as in a curve with top loaded static tests, the validity of the test can be assured as the curves result in close agreement as in Figure 5.26.

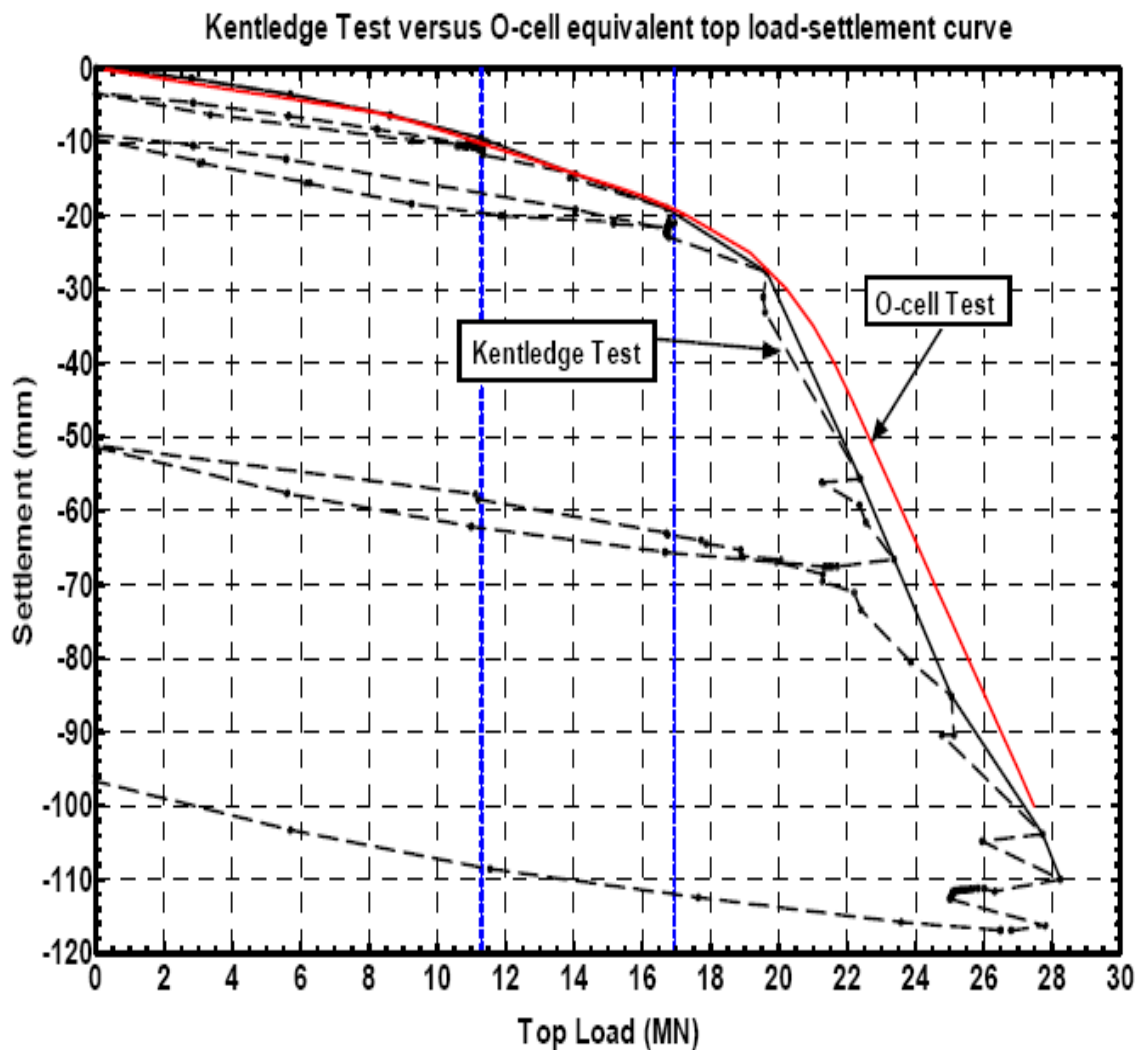


Figure 5.26. Top loaded (kentledge) static test curve versus O-cell equivalent top-loaded settlement curve (M. England, 2003)

The current comparison between cost of top down and osterberg load tests is illustrated in Figure 5.27. The costs per MN is similar at loads up to 5 MN. At higher loads, osterberg tests become increasingly more cost effective. Despite all these advantages of osterberg cell method, also some limitations of the test compared to the top loaded static load test should be taken into account. First, the O-cell must be installed prior to construction and advance installation is required, including qualified engineer, welders, operators and labor. The installation of tremie from the opening on O-cell is hard and might be unsafe, as the holes are adjacent to the side of borehole. Unproper funnel construction may lead to a tremie sticking during casting operation, resulting in loss of tremie pipes. The cables belonging to all measurement devices should be kept under

protection; a thick nylon bag would be suitable to cover all cables standing outside the borehole during and after casting to protect them from any risk of damage. Besides, to be used as a working pile, around of O-cell should be well grouted after the completion of the test. Moreover, O-cell method is not suitable for steel and H piles. Last, the equivalent static top load-deflection curve can be estimated with conservatism, but it still remains an estimate.

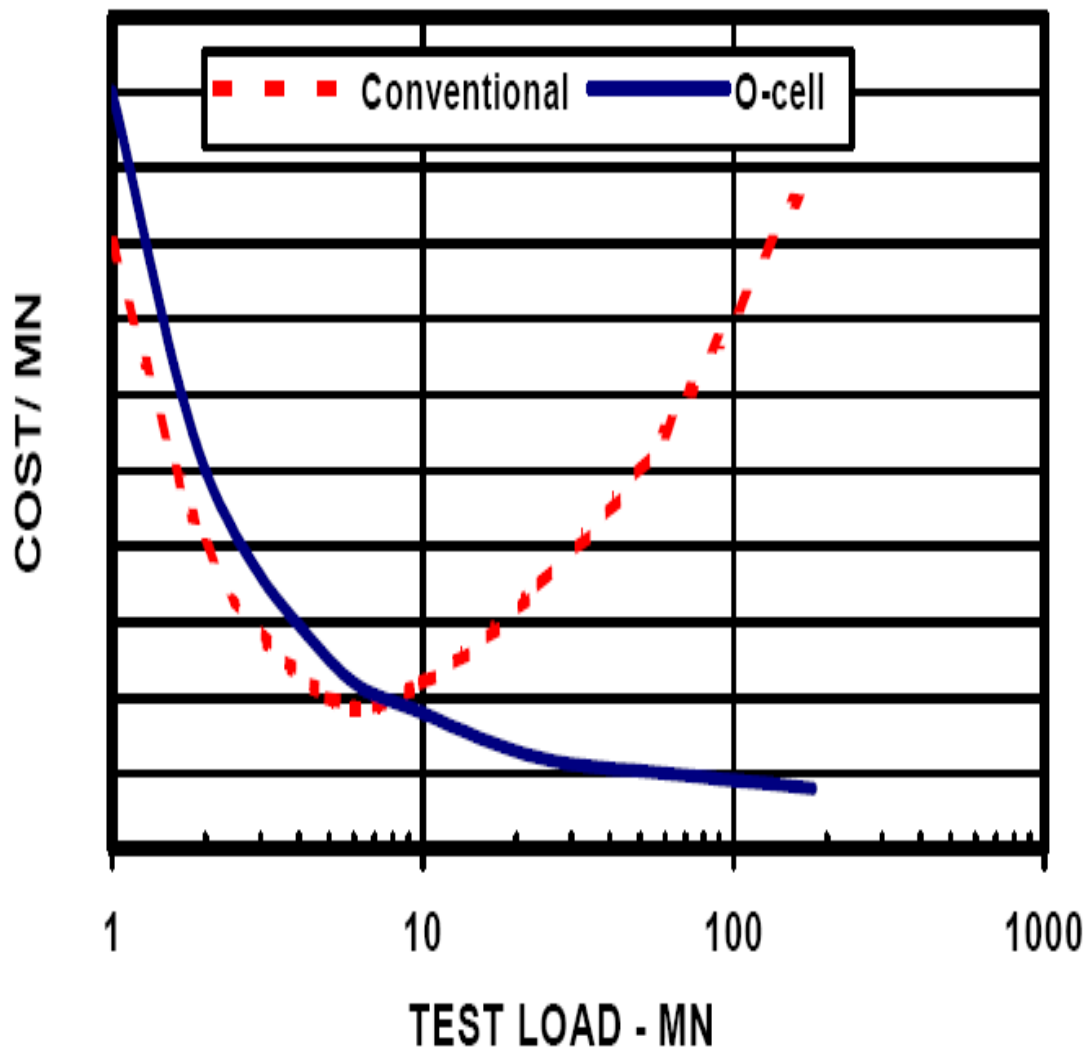


Figure 5.27. Comparison of load testing costs; conventional static test vs. o-cell
(England, 2003)

5.4. Dynamic Load Testing

5.4.1. Introduction

Dynamic testing was pioneered by Dr. G.G. Goble and his colleagues at Case Western Reserve University in Cleveland Ohio and is now a routine pile capacity evaluation method. High strain dynamic testing is the procedure of testing vertical piles individually to determine the force and velocity response of the pile to an impact force applied axially by a pile driving hammer or similar device that will cause a large strain impact to the top of the pile as shown in Figure 5.28. This strain impact is required to be representative of a force in the pile having the same order of magnitude, or greater, than the ultimate capacity of the pile. This test method provides data on strain or force and acceleration, velocity or displacement of a pile under impact force. The data are used to estimate the bearing capacity and the integrity of the pile, as well as hammer performance, pile stresses, and soil dynamics characteristics, such as soil damping coefficients and quake values. Moreover, with this method also determination of the acceleration and displacement of the pile head, and the energy transferred to the pile is possible (ASTM-D4945). Dynamic testing is not only an efficient and reliable alternative to static load tests but also costs lower than other static tests. This test allows contractors and approval agencies to reliably test all types of deep foundations. This method even works where static testing is impossible, such as on bridge foundations on water, oil platforms and other near shore and offshore sites (Whitaker, 2002).

5.4.2. Testing Equipment and Installation

For applying the impact force to pile, a driving system is needed, provided it is capable of generating a net measurable pile penetration. This system includes a drop weight and hammer dropping device as illustrated in Figure 5.29 and 5.30. The device should be positioned so that the impact is applied axially to the head of the pile and concentric with the pile. Next, a shaft top cushion consisting of thin sheets of plywood to be applied on top of the pile to be tested is needed for distributing the loads as shown in Figure 5.31.



Figure 5.28. Dynamic testing overview



Figure 5.29. Testing hammer



Figure 5.30. Drop weight



Figure 5.31. Steel cushion

To obtain the dynamic measurements, two strain transducers capable of independently measuring strain versus time at a specific location along the pile axis during the impact event and two acceleration transducers capable of measuring acceleration versus time which can be integrated to velocity are needed. These transducers should be inserted at equal radial distances on diametrically opposite sides of the pile and also at the same axial distance from the bottom of the pile so that the measurements compensate for bending of the pile. When near the upper end, they should be attached at least one or one-half pile diameters from the pile head, as decided by the engineer depending on the site conditions. Also, an analog or digital computer based system which not only records and reduces the signals from the transducers to determine the force and velocity versus time but also displays the data by an oscilloscope or graphics screen as shown in Figure 5.32 is needed. If accelerometers are used, the apparatus should integrate the acceleration over time to obtain velocity. If displacement transducers are used, the apparatus should differentiate the displacement over time to obtain velocity. The device also should provide the possibility to make an internal calibration check for the attached transducers. Results for each blow are displayed and then stored in a data file which is then automatically plotted for report quality presentations (ASTM-D4945).



Figure 5.32. Pile driving analyser on field

5.4.3. Test Procedure and Analysis

To start with, the test should not commence until seven days or more time elapsed since the casting date of the concrete. If a permanent casing is not used as a feature to construct the pile or where reinforcement rods extend above the pile top, than the pile top shall be extended by length equal to a minimum of one times the pile diameter. This test area also should be reinforced with steel reinforcement. If the pile top is below grade, then the surrounding soil shall be removed so as to completely expose a test area of the pile. At the center of the pile a hole, usually with diameter and depth of 100 mm. is to be made for the hammer for placing the testing rod (guide of hammer) at the center of the test area, thus allowing concentric impact blows. Before excavating the site area, care should be taken to provide enough space for crawler crane to easily move and carry the testing equipment on the test pile. Figures 5.33 and 5.34 illustrate the test preparation.



Figure 5.33. Installing drop weights



Figure 5.34. All drop weights installed

The apparatus for applying the impact force should be positioned so that the force is applied axially and concentrically with the pile. The apparatus for recording, reducing and displaying data, named as 'Pile Driving Analyser' should be set up so that it is operational and the force and velocity signals are zeroed. Next, the number of impact for a specific penetration is recorded. For hydraulic hammers, the kinetic energy from the hammer readout is recorded. Usually an initial small impact is applied to check the instrumentation and alignment. Blows with increasing drop height are then applied until either the stresses reach the strength limits of the pile, or until the set per blow exceeds about three millimeters which activates the full capacity, or until the result indicates a capacity sufficiently in excess of the requirements for the project, whichever comes first (G. Likins, 2004). The recommended drop weight is at least one per cent of the required ultimate capacity to be proved for shafts installed in clay soils or into rock sockets (Hussein, 1996). For piles with larger expected end bearing contributions, the recommended percentage increases to at least two per cent of the load to be tested. The energy delivered to the pile is directly computed as the work done on the pile from the integral of force times incremental displacement which is the force times velocity integrated over time.



Figure 5.35. Strain transducer and accelerometer mounted on a pile (<http://www.whitakerlab.net>)

A series of force and velocity measurements are taken from signals of transducers and recorded and displayed. At this stage, the force and the product of velocity and impedance are compared at the moment of impact. The measurements also allow direct computation of the compression stress at the pile toe and the tension stresses along the shaft. Pile integrity can be evaluated by inspecting the measurements for early tension returns caused by pile damage prior to the reflection from the pile toe; lack of such reflections assures a pile with no defects.

Consistent and proportional signals from the force or strain transducers and the acceleration, velocity and or displacement transducers shown in Figure 5.35 are the result of the transducers systems performing properly and the ‘Pile driving analyser’ being properly calibrated. If the signals are not in proportionally agreement, the situation should be checked and corrected. The theory of these computations are presented in Chapter 5.4.3.2.

The force and velocity is obtained from the readout of the apparatus for reducing data or from the display apparatus. The impact force, velocity and the maximum and minimum forces for the selected representative blows should be recorded. The displacement should be obtained from the pile driving record, and from the displacement transducer or by integration of the velocity versus time record. The maximum energy transferred to the location of the transducers is obtained (ASTM-D4945).

5.4.3.1. The Case Method and Wave Equation Analysis. The modern era of dynamic pile testing based on stress-wave measurements and analysis to evaluate hammer, pile and soil performance utilizing routine field measurements and real-time data processing started in the 1950s at Case Institute of Technology in Cleveland, Ohio, USA. Original scientific studies, innovative experimental work and creative applied research at Case under G.G. Noble and with the contributions of Robert Scanlan and Fred Moses, produced a body of scholarly literature and state-of-the-art hardware measurement technology which made possible the practical application of stress-wave theory to high-strain pile analysis. The measurements of both force and acceleration at the pile top by transducers at early 1960s made possible the improvement of the analysis. By assuming the pile and total driving system as a single rigid body it was possible then to apply Newton's second law at the instant zero velocity. Improved analytical tools were developed by the Case research team to assess the soil response, and hence pile capacity and the original method was modified to represent the pile as an elastic rod. The resulting solution for bearing capacity took into account the stress wave effects on long elastic rods. This solution could be applied and solved in real time for each hammer blow by special purpose analog electronic equipment. Results were obtained, giving answers for the capacity at the time of testing. By the mid 1970's with the advent of digital computation, improved analysis methods become available to improve the accuracy of the capacity prediction (DiMaggio and Hussein, 2004).

Using soil models similar to those developed for the wave equation analysis, the Case researchers also developed an analytical method called CAPWAP (Case Pile Wave Analysis Program) in the late 1960's which replaced the hammer model of the wave equation with pile top measurements. This program is a rigorous numerical analysis procedure which uses the derived force and velocity data to solve for soil resistance

parameters. The pile is divided into segments of approximately one meter in length and soil resistance are assigned to every embedded pile element to model the shaft friction and an additional below the pile toe to model the toe bearing. The soil model for each soil element contains a static resistance represented by an elastic-plastic spring with an ultimate resistance, and a limiting elastic displacement termed the quake. Soil damping is modeled as a viscous dashpot with a damping factor which relates the magnitude of dynamic soil resistance to the pile loading velocity as shown in Figure 5.36. The measured pile top velocity is traditionally imposed as an input and trial values are assigned to all soil model parameters. The required pile top force is then computed and the solution compared with the measured force. The agreement between computed and measured pile top force is iteratively improved by modifying the assumed soil model parameters until good matching is obtained between computed and field measured records. The final soil parameters then represent the best match dynamic soil model (DiMaggio and Hussein, 2004).

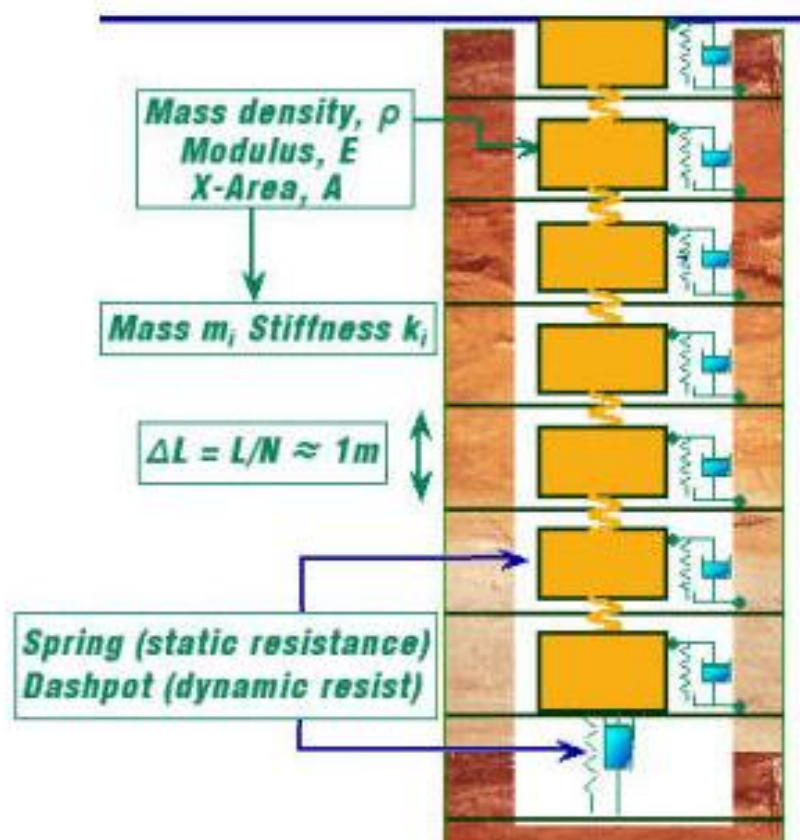


Figure 5.36. Soil modelling in wave equation analysis (<http://www.whitakerlab.net>)

5.4.3.2. Interpreting Test Data. During dropping the test hammer, energy is transmitted from the hammer to the pile and as the pile penetrates into the soil, a compressive stress wave travels down and propagates through its shaft at a speed defined as wave speed (ASTM-D4945) which is a property of pile composition. Against this wave, soil resistances are generated and these soil resistance forces along the shaft and at the toe also cause wave reflections that travel and are felt at the top of the foundation. The impact from the strike of hammer to test pile also induces a force 'F' and a partial velocity 'v' as the strain wave passes by, at the top of the foundation. The force is computed by multiplying the measured signals from a pair of strain transducers attached near the top of the pile by the pile area and modulus of elasticity. The velocity measurement is obtained by integrating signals from a pair of accelerometers also attached near the top of the pile, as stated above.

Therefore, the wave speed should be determined for the analysis. For this, the pile is placed on supports or level ground free and clear from neighboring piles and obstructions. The accelerometer is attached to one end of the pile and the other end of the pile is stroke with a sledge hammer of suitable weight. The accelerometer signals are recorded and displayed. The time between acceleration peaks shall be measured for as many cycles of reflection as possible. This time is divided by the appropriate travel length of the strain waves during this interval to determine the wave speed (ASTM-D4945).

The wave speed is a function of the elastic modulus and mass density, and the modulus of elasticity for bored piles can be calculated from the square of wave speed:

$$E = pc^2 \quad (5.2)$$

where,

c = Wave speed

p = Mass density

Pile impedance, which indicates the resistance a pile has to a sudden impact change in velocity as stated above, can also be calculated from the wave speed (ASTM-D4945):

$$Z = \frac{AE}{c} \quad (5.3)$$

where:

Z = Pile impedance

A = Cross-sectional area

E = Young's modulus of elasticity

c = Wave speed of pile, and

p = Unit specific density.

As long as the waves travel in one direction, force and velocity are proportional and this can be used as a confirmation of data quality of the test (ASTM-D4945):

$$F = Z.v \quad (5.4)$$

where,

v = Velocity

Z = Pile impedance

F = Force

Total computed soil resistance includes both static and viscous dynamic components. The static resistance can be obtained by subtracting the dynamic component from the total soil resistance. The dynamic component is computed as the product of the pile velocity times a soil parameter called the 'Damping Factor', which is related to soil grain size.

The accuracy of dynamic testing capacity predictions has been the subject of many past studies. In order to reliably use the dynamic testing results, many have compared the dynamic testing results with static load tests on the same piles. Of course both should be run to failure, and ideally both would be performed with similar wait times after installation so that known strength changes with time would be minimized. Table 5.2 statistically compares the dynamic testing results with signal matching (e.g. CAPWAP)

against static load tests for three major studies. On the table; 'n' refers to the number of piles, 'avg' to the average CAPWAP result divided by the static test result, and 'cov' to the coefficient of variation which measures the dispersion of results or reliability. It should be generally noted that correlation of one definition of failure versus another on the same piles results in similar statistical confidence and thus it could be concluded that CAPWAP is statistically similar to other evaluation procedures (Likins, 2008).

Table 5.2. Statistical evaluation of dynamic pile testing (G. Likins, 2008)

Study (1)	avg (2)	cov (3)	n (4)	Notes (5)	Time Ratio (Dynamic to Static) (6)
1980	1.010	0.168	77	Original Goble study	
1996	0.964	0.223	83	Automatic only	
1996	0.931	0.166	83	Best match	
1996	1.012	0.097	83	Radiation damping	
1996	1.019	0.092	61	Radiation damping	Dynamic / static > 0.25
SW	0.993	0.165	143	All piles	
SW	0.983	0.156	119	All driven	
SW	1.037	0.199	23	All drilled and cfa	
SW	0.972	0.147	45	All piles	Dynamic / static > 0.25
SW	0.910	0.183	96	All piles (CW/max)	
SW	0.968	0.101	24	All piles CW/SLT 20 mm	
All	0.980	0.169	303	(1996 data uses 'best match' method)	
2000	0.930	0.146	75	Static versus static-Paikowsky	

5.5. Cross-hole Sonic Integrity Logging (CSL) Testing

5.5.1. Introduction

Deep foundation elements, drilled shafts and cast in-situ concrete piles carry very high design loads. Such conditions created a need for a high level of quality assurance and control applied to each constructed deep foundation elements. In order to check or test the quality of such high load carrying foundation piles constructed in multi-layered soil

conditions, non-destructive integrity tests are employed. Amongst these tests, cross-hole logging test is one of the most reliable advanced and economical techniques for assessing the integrity of in-situ constructed deep foundation elements. Cross-hole logging tests are typically performed on concrete, particularly concrete drilled shafts.

When cast in-situ piles are constructed, the following defects may occur (Candogan, 2008):

- Honeycombing due to inadequate vibration and segregation due to over vibration and improper concrete placement methods.
- Washout of cement due to groundwater flow.
- Cracks in pile shaft due to shrinkage or vibrations at close to fresh concrete.
- Inclusion of foreign material causing contamination of the concrete.
- Bulging out of the pile due to insufficient passive pressure of the soil and necking of the pile due to insufficient passive pressure of the wet concrete.

The main objectives in the cross-hole sonic testing (CSL) is to provide the clients/consultants information about these defects on the piles by evaluating the integrity of the deep foundation such as cracking, voids, poor quality concrete and soil-slurry intrusions as stated above. This method of testing deep foundations was first developed in France by the Experimental Center for Research and Studies in Building and Public Works (CEBTP) in the late 60's. CSL techniques have been designed to assess the quality of piles after installation, without destroying what has been built and without development of more efficient and reliable constructed systems. CSL provides the engineer with an inexpensive method of testing concrete and of locating and assessing the extent of any irregularity. The basic principle of the test is that the velocity of an ultrasonic pulse through concrete is directly proportional to the density of the concrete. By measuring the propagation time and relative energy of an ultrasonic pulse along a known distance between parallel preformed steel or plastic tubes installed in the borehole named access ducts, a measure of the density and hence, quality of concrete can be determined. The method uses high frequency compressional sonic waves as the energy source which are transmitted between two access tubes with source and receiver in separate tubes. The waves passing through concrete are influenced by the density and elastic modulus. Fractured or weak concrete zones lower the

velocity of the waves so exhibit delayed arrival times with reduced signal strength and, thus, can be detected. It offers excellent results in pinpointing the locations and assessing the magnitude of a defect both horizontally and vertically (S. Niyama, 2000).

This method does not give the exact type of defect, but rather only that a defect exists. Moreover, it is limited primarily to testing the concrete between the access ducts and thus gives little information about the concrete outside the reinforcement cage to which the access ducts are attached when the tubes are attached to the inside of the reinforcement cage. This method is most applicable when performed between tubes that are installed during pile construction (ASTM-D6760).

5.5.2. Testing Equipment

First, access ducts, which are made of steel or plastic tubes, are needed for the internal inspection of the pile by providing access for the probes. Steel tubes are more usable than plastic ones since more aligned and frequent attachment to reinforcement cage is needed, and there is a risk of deformation of plastic material during the high temperatures of concrete curing. The access duct diameter should be large enough to allow easy passage of ultrasonic probes however too large ducts may cause improper arrival time estimations. Therefore according to ASTM-D6760 access ducts typically have an internal diameter from 38 to 50 mm.

Next, two piezoelectric probes shall be placed and used in the access ducts, consisting of ceramic transducers in sealed tubes, for one to generate and transmit the ultrasonic pulse and the other to receive the pulse. The signals from the transmitter and receiver probes and the depth-measuring device shall be transmitted to a field rugged, computerized apparatus for recording, processing and displaying the data in the form of an ultrasonic profile as shown in Figure 5.37. The apparatus shall read the depth-measuring device and assign a depth to each digitized ultrasonic pulse. The apparatus shall store these raw digitized ultrasonic pulses and the processed data from each ultrasonic profile for each pair of access ducts. Also the computerized apparatus should be capable of analyzing all recorded data to identify at least the first arrival and energy of the transmitted ultrasonic pulse at the receiver probe for each depth interval. Last, the apparatus should also be

capable of displaying both the raw receiver ultrasonic pulses for data acquisition, and the raw data of each ultrasonic pulse along the entire pile length (ASTM-D6760).



Figure 5.37. The cross hole ultrasonic monitor system (<http://www.piletest.com>)

5.5.3. Installation

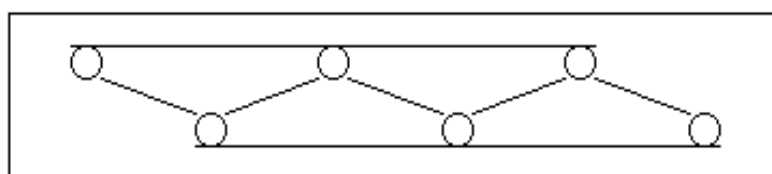
First, the access tubes should be installed into reinforcement cage before lowering of the cage to drilled shaft. The tube number and spacing according to pile diameter is stated at Table 5.3 and Figure 5.38 (ASTM-D6760).

Table 5.3. Access tube number variation with pile diameter (ASTM-D6760)

Diameter	Min. No. Of Tubes	Tube Spacing
$D < 750$	2	180^0
$750 < D < 1000$	3	120^0
$1000 < D < 1500$	4	90^0
$1500 < D < 2500$	6	60^0
$D < 2500$	8	45^0

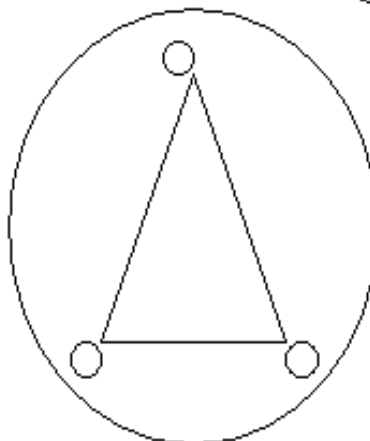
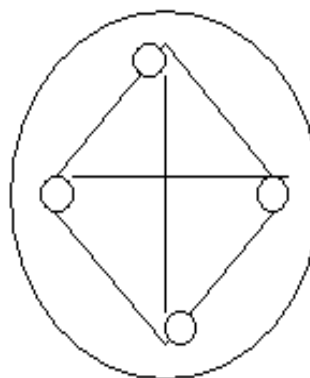
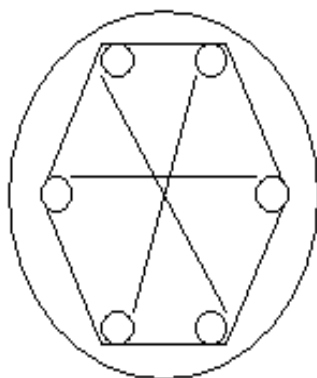
Prior to installation to reinforcement cage, it should be checked that the access tubes are straight, free from obstructions and contaminations such as oil, dirt and loose rust. Besides, the tube ends should be undamaged and suitably prepared for the end cap and coupling installation. The access tube should be close-ended at the bottom and fitted with removable end caps at the top to prevent entry of concrete or foreign objects, which could block the tubes prior to testing operations. The couplings are used if extension of the access tubes is necessary due to long tube lengths.

Diaphragm wall panel



Pile Diameter 1500 to 2100 mm

Pile Diameter 1000 to 1400 mm



Pile Diameter < 1000 mm

Figure 5.38. Typical access duct configurations (ASTM-D6760)

The access tubes shall be installed such that their bottom is as close as possible to the bottom of the concrete pile or structure so that the bottom condition can be tested. The tubes should be extended about one meter above the top of the shaft to compensate for the water displaced by the transducer, receiver and cables. After installation of the cages, the top end caps shall be temporarily removed and the tubes shall be inspected to verify they are clear of obstructions. A measurement tape might be used to check the clearness of the inside of tubes. Finally, tubes should be filled with water prior to, or within one hour of, concrete placement to assure good bonding of the concrete to the tube after the concrete cools (ASTM-D6760).

If more than one cage is lowered into the shaft due to the depth, the tube end should be maintained about 20 – 40 cm above from the beginning of the overlap (end of stirrups) for the first cage and for the second cage the tube should end 20 – 40 cm above the bottom of the cage so that during installation of steel cages, it would be easier for the labors to connect tubes between neighbouring cages.

5.5.4. Testing Procedure and Analysis

The main measurement of cross-hole sonic logging techniques is the transit time of the signal from emitter to receiver. The source transducer is lowered to the bottom of one of the tubes and the receiver transducer is lowered to the bottom of another tube placed in the same pile by not exceeding 0.5 m/s. Both probes are placed at the same vertical level as shown in Figure 5.39.

During testing, transmitter and receiver are pulled simultaneously and the transmitter is triggered proportional to the advancing vertical position of the transducers at the selected increments. By the time, a wheel controls the resolution of the collection data. A broad band longitudinal ultrasonic waves are passed through the concrete of the pile from the transmitter to the receiver, and they are delayed and attenuated based on the physical properties of the concrete. This observation is controlled by computerized system as shown in Figure 5.40 during testing.



Figure 5.39. Probe installation for cross-hole test



Figure 5.40. Data collection from transmitters

If an ultrasonic pulse is not obtained for any depth interval during raising of the probes, than the probes shall be lowered past that depth and the test repeated until all depth intervals have an associated ultrasonic pulse. In homogeneous concrete, free of defects and variations in quality, the propagation velocity of the acoustic waves is constant at about 4000 m/s. The source and receiver are pulled to the top of each shaft at a speed of 300 mm/sec. thus giving a complete assessment of the concrete quality between tubes. Cross hole sonic tests are typically performed between all the perimeter tubes to check the perimeter of the shaft. The test results are displayed on the screen of computerized apparatus as shown in Figures 5.41 and 5.42.

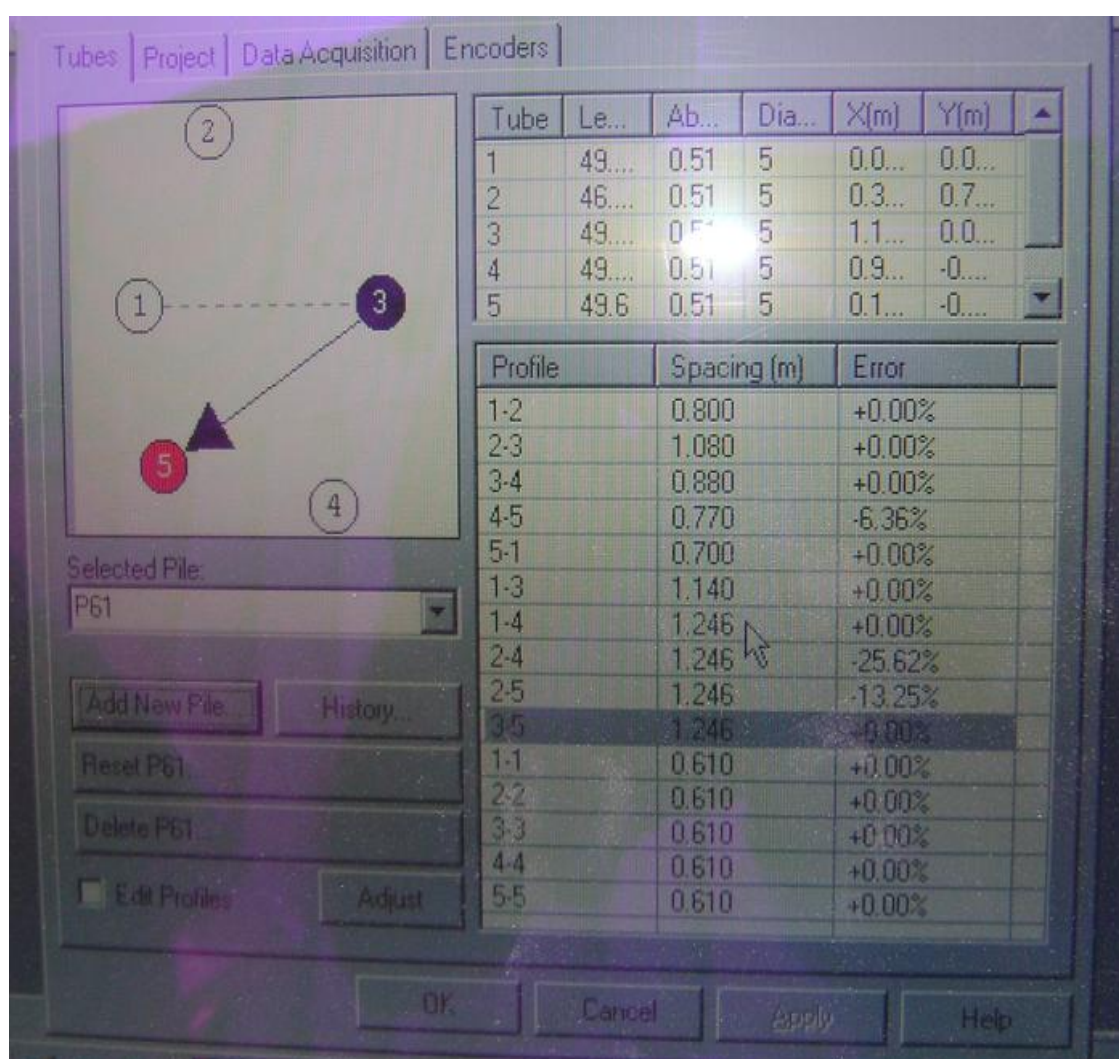


Figure 5.41 Test data display screen

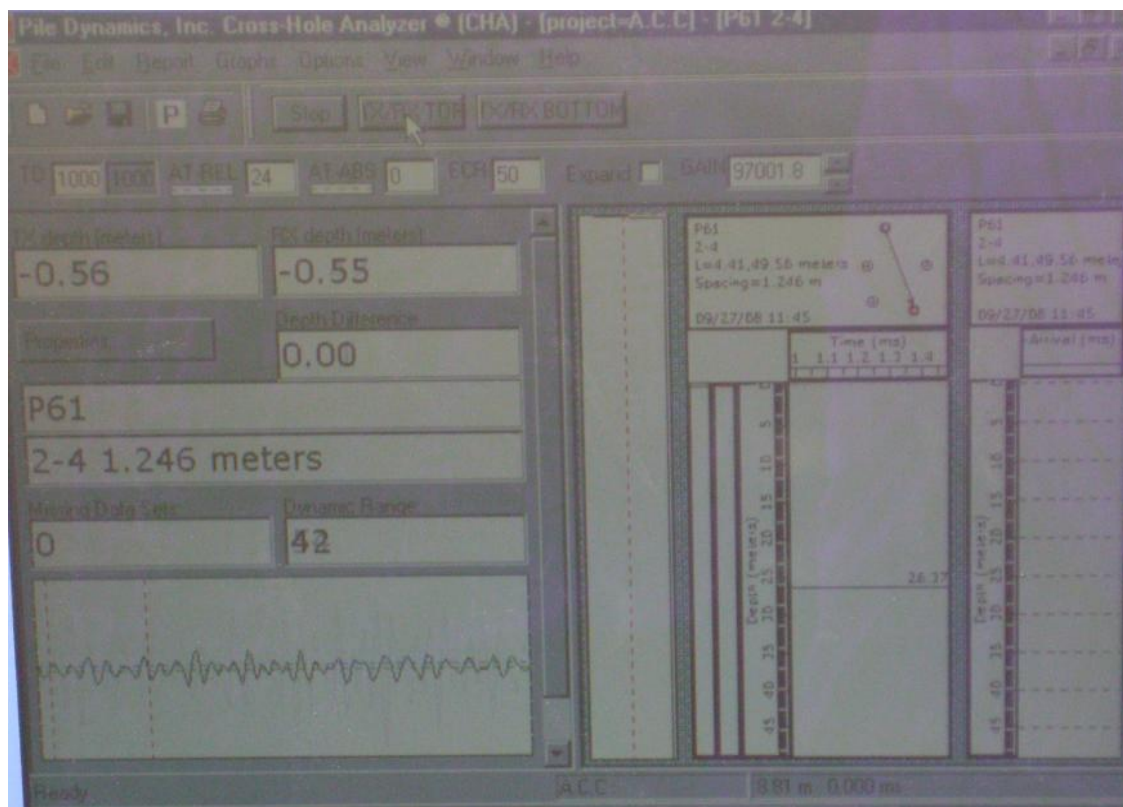


Figure 5.42. Original data screen

The variation in signal arrival time enables one to assess and locate areas of low density, and so damaged concrete. Concrete containing soil inclusions, gravel, bentonite or honeycombing has a much lower propagation velocity so that the presences of these irregularities are immediately obvious. The amplitude and the sinusoidal shape of the signal will change as well if there is anything but sound concrete along the path of the wave path. Signal attenuation, or loss of energy, is sign of poor quality zones because more energy is transmitted through sound concrete than through poor concrete. The test is more suitable for integrity testing of deep drilled shafts because signal attenuation problems do not occur as with the surface methods, and the travel distance between the two probes at any depth is relatively short and essentially constant. If the ultrasonic profile indicates an anomaly, then the suspect anomaly zone may be further investigated by special test procedures such as fan shaped tests, tests with the probes raised at a fixed offset distance, or other tomographical techniques as illustrated in Figure 5.43 (ASTM-D6760).

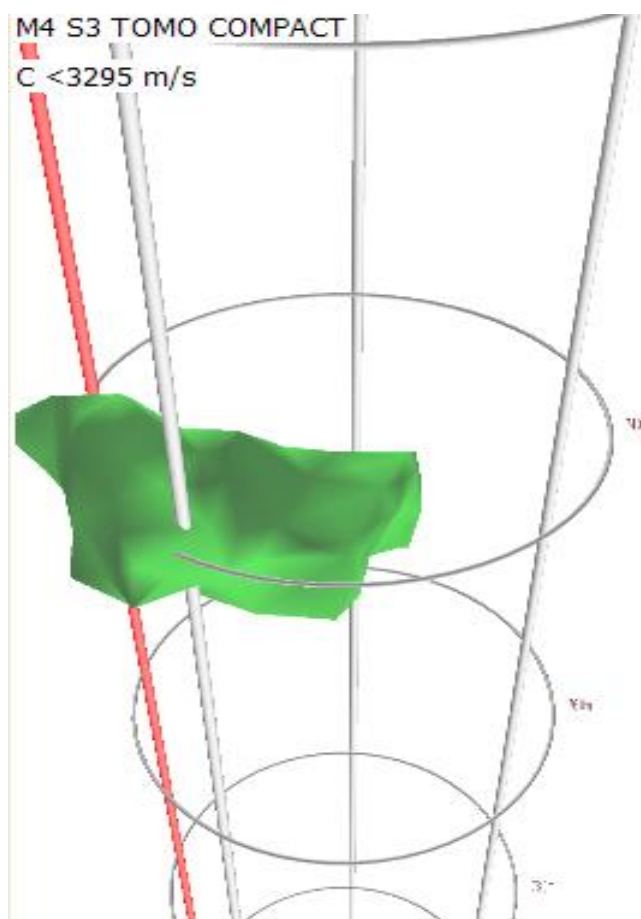


Figure 5.43. 3D tomography output showing a pile defect (<http://www.piletest.com>)

5.6. Low-Strain Integrity Testing with Pulse-Echo Method

5.6.1. Introduction

The low-strain integrity test with pulse-echo method is used to prove the quality of deep foundation elements such as bored or driven piles. It is used for routine quality control and also to detect defects when observations during pile installation raise doubts in their quality. The method is able to detect pile defects or irregularities such as:

- Cracks especially in unreinforced piles.
- Reduction of the cross sectional area, e.g. due to stratification or faulty pile installation.
- Deviations in the pile length.

- Poor concrete quality indicated by a low wave speed.

The low strain method of integrity testing can be regarded as the most suitable and cost effective tool when compared with other available methods such as cross hole sonic logging, coring and excavation so this test has become the standard method of quality assurance for deep foundations in many countries. Figure 5.44 summarizes information obtained from independent testing institutions and the experience of individual members of the working group which can be deemed as representative of 25 years pile integrity testing (Klingmüller and Kirsch, 2004).

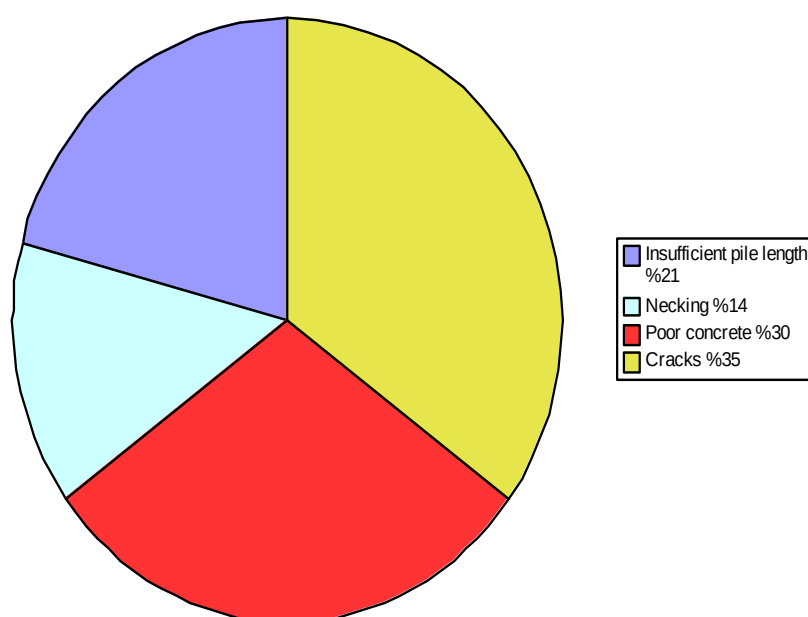


Figure 5.44. Shortcomings with tested piles (Klingmüller and Kirsch, 2004)

Figure 5.45 illustrates the purpose of low-strain integrity tests carried out by an independent institute. Special purposes are concerned with testing of piles with I-beams inserted, additional test following faulty cross hole tests, testing of sheet pile walls, concrete pile walls or of piles below existing structures or even testing of historic buildings and medieval timber pile foundations.

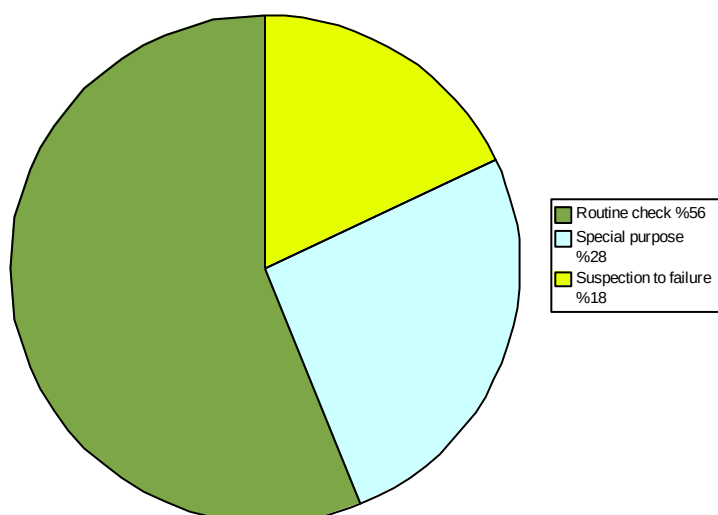


Figure 5.45. Purpose for low-strain pile integrity testing (Klingmüller and Kirsch, 2004)

The integrity testing procedure according to the low-strain method is based upon the introduction of an impact into the pile and the subsequent measurement of the pile head movements. These movements allow the interpretation of the shock wave propagation and its reflection, so one dimensional wave theory, as like in Section 5.4.3.1, is used in the analysis.

5.6.2. Testing Equipment and Installation

Measurement of the pile head movements are normally taken by piezoelectric accelerometers. These are connected by cable with a portable device containing an amplifier with filters and converters as well as a digitizing and storage unit. An example of such a device together with accelerometer and hammer is shown in Figure 5.46. Data from this unit is transmitted to a normal personal computer (PC) for further processing. To carry out the integrity test, the pile head needs to be easily accessible and cleaned to the sound concrete. Any extra concrete close to the pile head leading to an enlarged pile cross section needs to be removed. The concrete itself has to be of sufficient strength to transmit the shock wave. Although concrete strength is heavily dependent on its composition, a rule of thumb is that pile testing should not commence before seven days after concreting (ASTM-D5882).

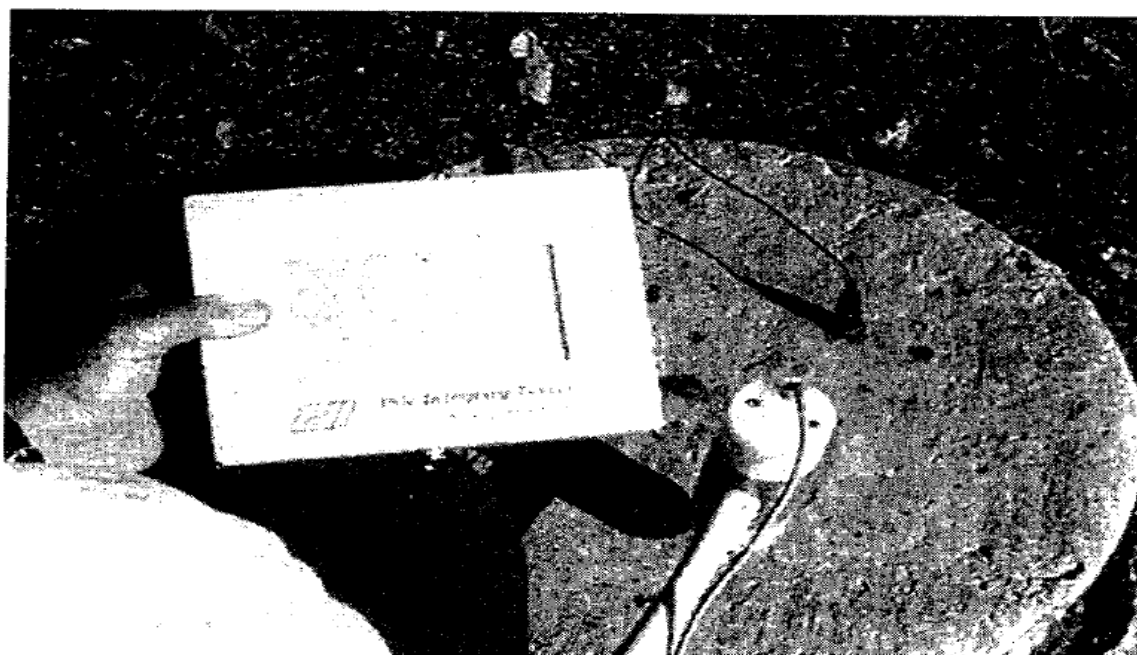


Figure 5.46. Device with accelerometer and hammer used in pulse-echo method
(Klingmuller and Kirsch, 2004)

5.6.3. Test Procedure

The shock wave is introduced to pile by special hammers. The shape and duration of the shock input needs to be adapted to the particular purpose of the measurement. Rigid hammer tips lead to a short duration of the impulse with a favourable resolution even of small deficiencies. Conversely a soft hammer tip results in a longer impulse duration with a longer reach but reduced resolution ability. It is good testing practice to vary the location of the hammer blow over the pile head in order to achieve a good representation of the measuring signal. In special cases it may become necessary to gring the surface of the pile head before the tests (Klingmüller and Kirsch, 2004).

On its way through the pile the shock wave loses energy, a phenomenon which is summarised in general under the term of damping. Thereby the echo returning at the pile head is rather small and can only be recognised directly under favourable conditions. Only after signal amplification the echo is clearly identifiable when it reaches the pile head again.

5.6.4. Interpreting Test Data

The visual assessment of the velocity-time plot is common practice in the analysis of low-strain integrity tests. The pile tester classifies the measuring signal as the following (Kirsch,2004):

- i. Class 1: The pile is in order.
- ii. Class 2: The pile is not in order, few quality deficiencies.
- iii. Class 3: The pile is not in order, considerable quality deficiencies.
- iv. Class 4: The measuring signal cannot be utilised.

Class 2 is somewhat problematic since it must be decided if and what kind of remedial measures have to be adopted, whereas with Class 3 as a rule the pile cannot be used for load transfer purposes without additional measures. Figure 5.47 shows the flow chart for classified quality classification.

The low-strain method can only be adopted in order to check the pile integrity. Statements on the pile capacity are only possible in an indirect manner, since changes in the cross section have effect on it. Although the testing procedure is truly simple and easy to understand, its application in the field and the subsequent signal analysis calls for adequate qualification. Besides, since often there is a certain time pressure for all parties involved in the pile construction process, carrying out the integrity testing and to evaluate the test results or to even carry out any necessary remedial work is regrettably not allowed for in the overall project schedule. It can therefore only be recommended and hoped for that responsible architects convince the project owners to allow sufficient time in their project schedules to carry out pile quality assurance measures and to at least earmark the possibility of remedial measures should they become necessary (Klingmüller and Kirsch, 2004).

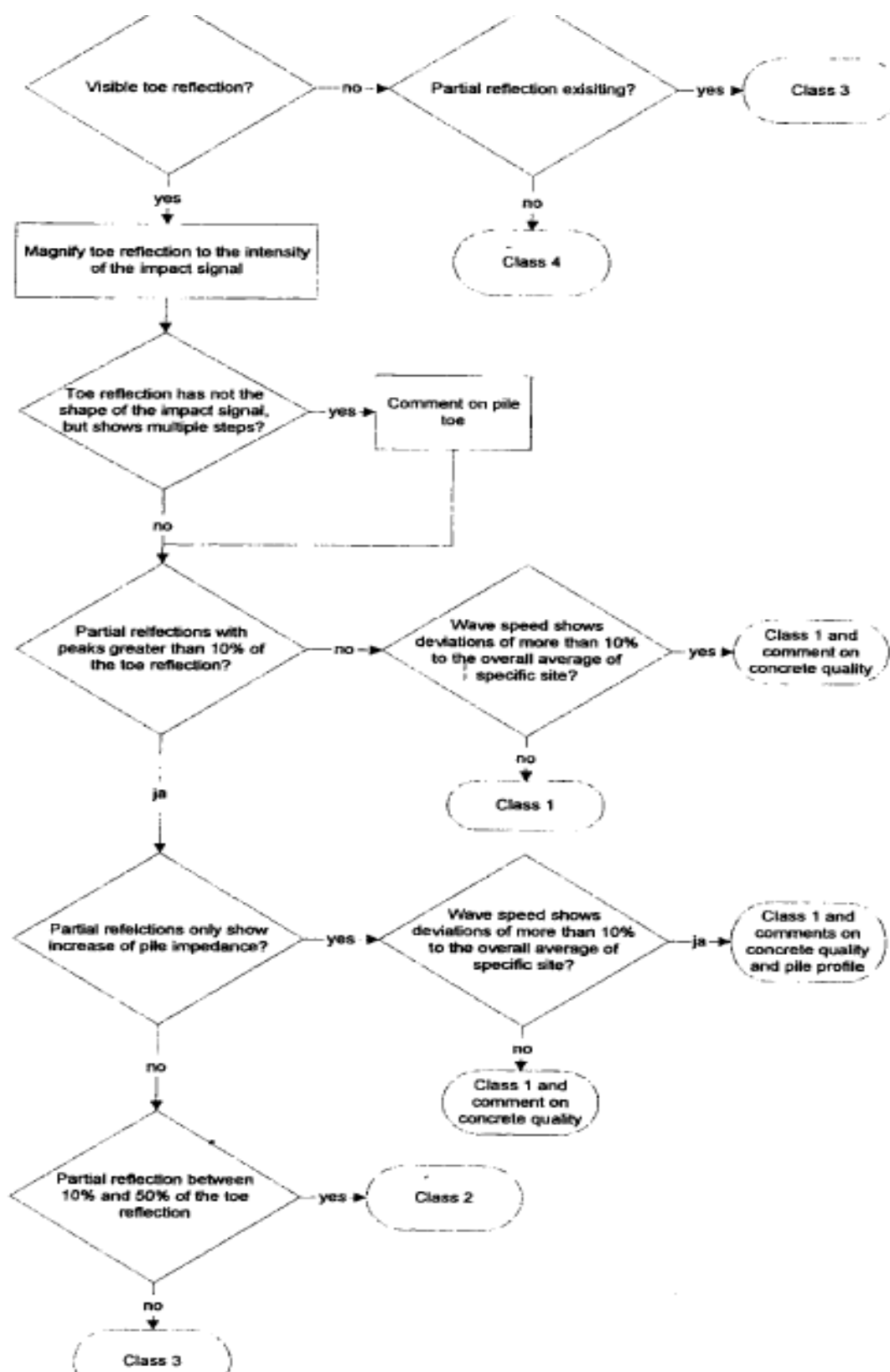


Figure 5.47. Flow chart for quality classification (Klingmüller and Kirsch, 2004)

5.7. Selection and Economy of Safety Factors in Pile Load Testing

Required pile capacity, which should exceed the applied loads by a sufficient margin to avoid unacceptable settlements, depends on the test method and frequency of testing for verification of the pile capacity. Pile testing is an effective way to decrease uncertainties from unknown loads or loading conditions, site variations and inaccuracies in load determination methods. Less testing and less accurate tests increase the risk of a failed foundation, expacially for bored piles since they have a higher probability of structural failure due to variability in the construction process (Likins, 2004).

Statistical methods used to determine pile capacities can always assess risk, so safety factors are used to decrease these risks. There are two main designs for safety factors; allowable stress design (ASD), and load and resistance factor design (LRFD). In ASD, the ultimate pile capacity is divided by a global safety factor to find the allowable or working load on the pile, and thus all uncertainty is lumped into this single factor. Whereas, LRFD designs recognize that different loading conditions have different uncertainty and therefore assign different load factors to these load conditions. Codes, both in public and private sectors, have been developed for both load and resistance factor design (LRFD) and allowable stress design (ASD) to minimize the risk of foundation failures. The selection of global safety factors to satisfy a foundation design with tolerable risk and at an acceptable cost relates to the type and amount of capacity verification performed. Based on a review of several codes, as shown in Table 5.4, a comparison of traditional methods of dynamic formulae and static analyses with modern methods of analysis and testing is made, demonstrating the economics of the modern methods through the reduction in safety factors.

To start with, static analysis which depends on numerous correlations and empirical correction factors for soil strength developed from SPT, CPT or other soil sampling tools are highly inaccurate, therefore the PDCA code requires a safety factor of 3.5 for static analysis and modern codes for driven piles even states as 'not applicable' due to the general practice that static analysis methods are generally only used to estimate pile lengths in the design process and driven piles are almost never installed to a depth by this method (Likins, 2004).

Table 5.4. Global safety factors for various deep foundation codes (Likins, 2008)

Code (1)	EC7 (2)	Australia (3)	PDCA (4)	AASHTO (5)	IBC (6)
Year	2001 (d)	1995 (c)	2001	1992	2000
design load-tons	50% DL	50% DL			>40
static analysis		2.12 to 3.44	3.5	3.50	6.00
Dynamic formula		2.50 to 3.06	3.5	3.50	NA
wave equation		2.50 to 3.06	2.5	2.75	NA
dynamic test (a)				2.25	2.00 (b)
low (# tests)	2.23 (#=2)	2.12 (<3%)	2.1 (2%)		
high (# tests)	1.95 (#>20)	1.72 (>15%)	1.9 (10%)		
static test				2.00	2.00
low (# tests)	2.29 (1)	1.93 (<1%)	2 (<0.5%)		
high (# tests)	1.64 (>5)	1.53 (>3%)	1.65 (>5%)		
static & dynamic (a, b)	(g)	(g)	(e)	1.90	(g)

Code (1)	ASCE driven piles (4)	ASCE driven piles (5)	ASCE driven piles (6)		
Year	1996 (f)	1996 (f)	1996 (f)		
design load-ton	16 to 40	40 to 100	>100		
static analysis	NA	NA	NA		
dynamic formula	2.0 – 2.4	NR	NR		
wave equation	1.8 to 2.2	1.9 to 2.3	NR		
dynamic test (a)	1.6 to 2.0	1.7 to 2.0	2.0 to 2.4		
low (# tests)					
high (# tests)					
static test	1.5 to 1.8	1.6 to 1.9	1.8 to 2.2		
low (# tests)					
high (# tests)					
static & dynamic (a, b)	(g)	(g)	(g)		

Notes: a	dynamic testing requires signal matching
B	requires at least one correlating static test
C	dynamic formula for sands only - not clays
D	draft code
E	3 dynamic tests can be substituted for 1 static test
F	depends on pile type, site variability, load conditions, etc.
G	no specific guidance
NA	not applicable
NR	not recommended

Dynamic formulae is also decreasing in usage due to simplicity and poor prediction accuracy, with similar safety factors as static formula. Then, wave-equation analysis introduces not only a series of assumed resistances and associated predicted blow counts producing a 'bearing graph' to establish a suggested driving criteria but also a computer simulation of the pile with numerical models so decreases the factor of safety values compared with static and dynamic formulae. However, because of the increased uncertainty associated with the assumptions, the risk is still high and thus safety factors in

the AASTHO and PDCA codes are suggested as 2.75 and 2.5, respectively. To sum up, analytical methods without pile testing not only includes high uncertainties but also are simple and far from construction influences, resulting in high safety factors which are not economical.

Static testing has traditionally been the standard for evaluating soil strength and thus ultimate pile capacity. The oldest static test method is the vertical top down loading test. However, these tests rarely fails on site, so ultimate failure load can only be estimated, increasing the factor of safety due to assumptions. Due to the fact that foundation costs can often be reduced when the ultimate failure load is determined, the quick procedure static test method and osterberg cell loading test method are getting more common. Moreover, it is not practical to statically test every pile because of time and cost constraints, such testing is usually limited to a very small sample of piles on any site (typically one per cent or less on large projects, or only one per site, if any, for small projects). The PDCA code committee, under the leadership by Dr. G.G. Goble, assigns lower global safety factors for testing more piles, because the uncertainty is reduced. The safety factor varies depending on the amount of testing, ranging from a safety factor of 2,0 for testing only 0,5 per cent of the piles, to a safety factor of 1,65 if 5 per cent of the piles are tested (Likins, 2004).

Extensive correlations between static and dynamic testing have verified the dynamic method's reliability (Likins, 1996), and a discussion of accuracy was included in Section 5.4.3.2. After correlating the static and dynamic tests, the PDCA code allows substitution of three dynamic tests for one static test in determining the quantity of further testing. Thus, with at least one successful correlation, the PDCA suggests that five per cent static testing can be translated into testing 15 per cent of the piles dynamically, for the same suggested global safety factor of 1,65. It is probably implicitly assumed that the large number of tests allow site variability to be properly assessed and hammer performance to be evaluated periodically throughout the project duration. Being rapid and economical tests, dynamic load testing method is widely used together with static load tests on both system and trial piles at piling sites.

6. CASE STUDIES

6.1. Introduction

Three case histories are presented in this chapter, including various tests conducted on bored piles which are all described in Chapter 5. Minimum two different types of load tests and integrity tests are conducted on each site, to give the chance to compare the results of each test with each other. All tests are conducted on either system or trial bored piles, depending on testing equipments and test preparation methods described in Chapter 5. The load tests focused on determining mainly the displacement behavior of piles under test loads and if piles carry the design requirements or not; however some test results would give extra information about load bearing capacities and even quality of the piles, which would not only guide the engineer to design more economic ways but also force him/her to give extra attention to construction methods and quality. On the other hand, integrity tests are conducted on larger number of piles on Case Study-3 to better understand pile quality, by checking the discontinuities and possible cracks on the bored pile, directly objecting the construction quality.

Another analysis which will be given in this chapter is that the test results will also be compared with allowable bearing capacities and pile displacements under design loads. To do that, first soil parameters, layers and conditions determined by soil investigations are presented for the sites. The location of sites are close and soil parameters of these sites are very similar. The theoretical bearing capacity and settlement formulas, which were presented in Chapters 2 and 3 are used for static calculations and will be compared with test results. Test results will also be utilized for the assessment of decrease in factor of safety values used in pile design. Since increasing the number of testing on a site decreases safety factor values leading to more economical design solutions, the cost of total pile testing will be compared with its profit on design in all cases.

6.2. Case study 1: Duja Tower Site, Dubai, U.A.E.

The proposed Duja Tower is designed as a 50-storey structure with four basements and one ground level. The project site is located at Plot No. 335-107, Trade Center, Dubai. Tower consists of 50 stories plus a ground floor, a mezzanine floor and four basement levels, totaling 195 meters in height, to be located along Sheikh Zayed Road in Dubai, United Arab Emirates. A car-park structure, consisting of 10 stories plus a ground floor, will be constructed adjacent to the tower.

6.2.1. General

The estimated construction cost of the mixed-used commercial and residential tower, including the car-park structure, is 177 million dollars. The tower is expected to have a total built-up area of approximately 135,468 square meters and tower construction covers approximately 60.0 m by 60.0 m area. The site layout of the subject project is shown in Figure 6.1 and design overview is shown in Figure 6.2.

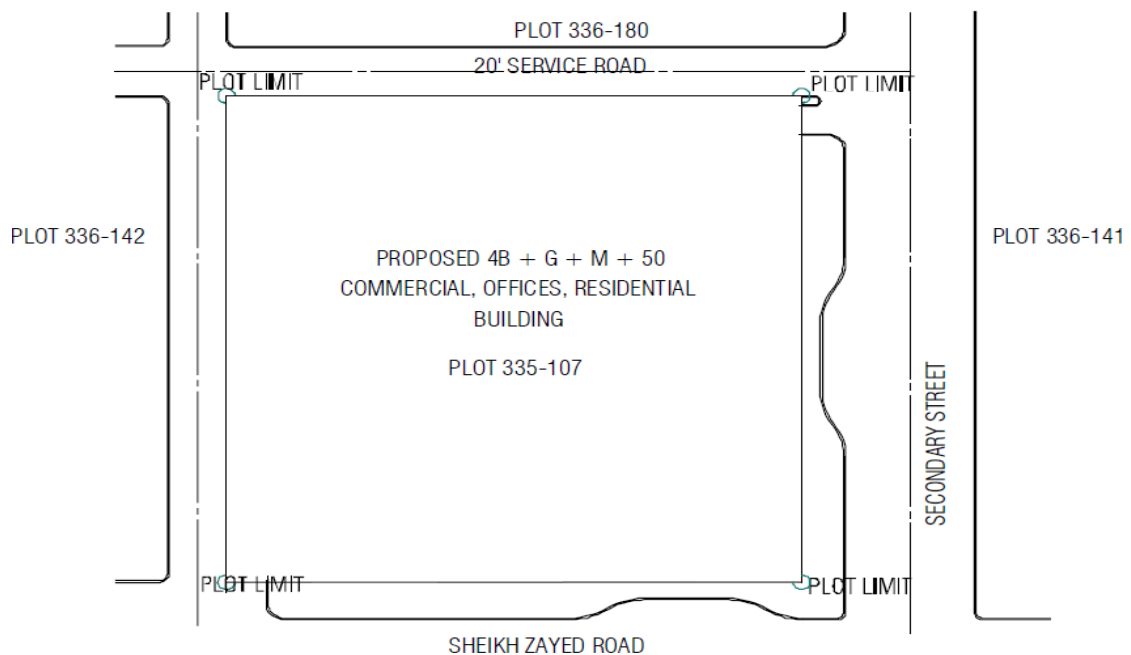


Figure 6.1. Site layout of Duja Tower



Figure 6.2. Duja Tower design overview

Bored piles, installed with rotary drilling system and with bentonite suspension method, was used to carry the superstructure loads. A total of two hydraulic drilling rig machines, two crawler cranes, two de-sanders and two bentonite pumps were used in the construction. The details of the construction method is discussed in Chapter 4. Figure 6.3 presents a picture from construction in day shift and Figure 6.4 in night shift.

Overall foundation engineering evaluations are prepared utilizing the data from four numbers of boreholes each to a depth of 50.00 meters and five numbers of boreholes each to a depth of 80.00 meters from the existing ground surface level. Soil investigation was performed by High Performance European Soil Testing Laboratory, consisting of two boreholes each 60.0m depth, cone penetrometer testing (CPT), laboratory testing including particle size distribution, unconfined compression test and chemical tests.



Figure 6.3. Construction in day shift



Figure 6.4. Construction in night shift

A total of 228 piles were designed to be constructed on the site. Following Table 6.1 summarizes the number of piles according to pile diameter in design.

Table 6.1. Pile details

Pile No.	P1	P2	P3	P4	P5	P6	P7a	P7b	P8a	P8b	P9a	P9b	P9c	P10a	P10b	P10c
Pile Diameter (mm)	1000	1000	1200	1200	1500	1500	1500	1500	1500	1500	1500	1500	1500	1500	1500	1500
Number of Piles	17	25	9	81	32	20	8	4	2	2	9	1	2	13	1	2

Table 6.2 and Table 6.3 situated below show the cost of the piling construction and pile testing respectively.

Table 6.2. Costs of Pile Construction

Item Ref.	ITEM DESCRIPTION	QUANTITY	Unit	Rate (\$)	Amount \$
PILING					
A	Allow for mobilization and demobilization for piling		Item	111.720	111.720,00
Boring and stabilisation of borehole					
B	For 1000 mm diameter	890	m	158,60	141.139,00
C	For 1200 mm diameter	2.385	m	217,17	517.941,00
D	For 1500 mm diameter	3.172	m	369,75	1.172.862,00
Reinforced concrete					
E	Reinforced concrete as specified	13.500	m ³	121	1.633.242,50
REINFORCEMENT BARS					
F	1000 mm diameter	1.200	Ton	1.171,66	1.406.000
G	1200 mm diameter				
H	1500 mm diameter				
TOTAL					4.982.896,20

Table 6.3. Costs of pile testing

Item Ref.	ITEM DESCRIPTION	QUANTITY	Unit	Rate (\$)	Amount \$
CUTTING OFF Pileheads					
A	1000 mm diameter	42	No.	381.47	16.021,80
B	1200 mm diameter	90	No.	545	49.046,32
C	1500 mm diameter	96	No.	844.70	81.090,00
TESTING					
Load Test on Non-working Pile					
D	Testing as required of 1200 mm pile	1	No.	62.670	62.670,00
E	Testing as required of 1200 mm pile (strain gauges)	1	No.	99.455	99.455,00
F	Testing as required of 1500 mm pile	-	No.	166.212	-
(OSTERBERG)					
G	Static Load Test	1	No.	40.872	40.872,00
H	Testing as required of 1000 mm pile	1	No.	59.946	59.946,00
J	Testing as required of 1200 mm pile	1	No.	107.902	107.902,00
	Testing as required of 1500 mm pile				
K	Dynamic Testing	2	No.	1.826	3.651,22
L	Testing as required of 1000 mm pile	5	No.	2.016	10.081,74
M	Testing as required of 1200 mm pile	5	No.	3.052	15.258,86
	Testing as required of 1500 mm pile				
N	Sonic Cross-hole Testing	8	No.	1.580	12.643,05
O	Testing as required of 1000 mm pile	18	No.	1.744	31.389,65
P	Testing as required of 1200 mm pile	19	No.	2.262	42.970,00
	Testing as required of 1500 mm pile				
R	Low Strain Integrity Testing	42	No.	15,00	629,43
S	Testing as required of 1000 mm pile	90	No.	15,00	1.348,77
T	Testing as required of 1200 mm pile	96	No.	15,00	1.438,70
	Testing as required of 1500 mm pile				
					490.256,13
TOTAL					

According to Table 6.3 situated above, one per cent of all piles were static load tested, five per cent of all piles were dynamic load tested, 20 per cent of all piles were sonic cross-hole integrity tested, and 100 per cent of the piles low strain integrity tested with pulse-echo method. As a result, approximately 10 per cent of pile construction cost corresponds to pile testing cost including pile head preparation.

6.2.2. Soil Conditions and Design Parameters

Existing ground levels range between +3,15 Dubai Municipality Datum (DMD) and +3,45 DMD, so ground level is assumed to be +3,25 DMD for the site. The surface deposit at the subject sites are formed by dredged materials, which were formed as a part of the Palm Jumeirah development. According to the soil investigation report the dredged material consists of varying/irregular densities ranging from dense to loose, grayish brown to light grey/dark grey, slightly silty to silty, fine to medium, often becoming coarse sand with shells and cemented pieces of sand with occasional sandstone fragments. Average thickness of this section is about eight meters. A loose to medium dense, fine to medium grained sand was encountered between -1,0 DMD and -4,0 DMD at some boreholes in this zone. A medium dense to dense, dark grey, grayish brown, slightly silty and gravelly sand overlies a dense to very dense sand of the same unit. Very weak, massive, light brown fine to medium grained calcarenite that can be assumed as fine to coarse grained soil is encountered at some locations. In the post vibro-compaction analysis it is stated that the zone between +1,0 m. and -6,0 m. (DMD) is compactable. Graphical cone penetrometer tests (CPT) results showing the tip resistance in sand and analysis of fines by corresponding friction ratios and percentages are also provided.

Soil profile at the cut-off levels of piles (-12.00 DMD) starts with calcarenite rock unit. This unit is followed by sandstone, conglomerate and calcisilite. All boreholes, both terminating at 50,0 m. and 80,0 m. depths, are ended in calcisilite formation. Top rock unit at the cut-off level is a weak to very weak, light grey, fine to medium grained calcarenite with closely to medium spaced fractured. Sandstone encountered in the boreholes is generally a weak to very weak, massive, reddish-brown, fine to medium grained sandstone. Conglomerate's classification can be generalized as moderately weak, massive, reddish brown and dark grey, fine to coarse grained unit. Calcisilite is generally encountered as a weak to moderately weak, massive, off-white to light brown, fine grained form. Rock formation starts from 13,40 m. to 13,65 m. below ground level (BGL). Between 13,50 m. to 16,0 m. BGL a weak, massive whitish grey, fine to medium limestone is reported instead of calcarenite. Although rock units in this zone is named different, evaluation of samples in the field, comparison of borehole photographs and unconfined compressive

strength test results show that they have comparable strength characteristics. Design parameters are obtained by incorporation of soil investigation reports.

At the time of investigation, the ground water level (GWL) was established at a depth of -13,16 m. to -17,20 m. below ground level (approximately -10.00 DMD to -14.00 DMD). This GWL was found to be very low against common ground water level expected in Dubai. Dewatering of nearby constructions was the most likely reason of this low GWL. Therefore, a higher GWL of -1,90 meters below working level which corresponds to around +1,35 DMD was designated for the design to provide safety, assuming that there were no dewatering operations took place around nearby constructions.

6.2.2.1. Layer No. 1 / Sand. The average reduction in site level after vibro-compaction is 0,75 meters. Considering the maximum cut off level of the piles is +1,85 DMD, the surface zone of +3,25 to +1,0 DMD is negligible.

Layer No. 6.2.2.2. / Compacted Sand. The compacted sand layer is taken from +1,0 to -6,0 DMD. According to the post vibro-compaction analysis report the friction ratio between -6,0 and -8,0 DMD is around 2,0 per cent – 2,5 per cent with a very low compressive strength value indicating a zone which will not respond to compaction. In accordance with the tip resistance analysis of CPT tests and the standard penetration test resistance (SPT/N) values of the lower sand layers internal friction angle is taken as ($\phi = 36^0$) for the compacted sand layer. The variation of SPT/N blow count values with depth as obtained within the upper sandy layer in the course of boreholes prior to the ground improvement works is plotted in Figure 6.5.

6.2.2.3. Layer No. 3 / Sand. The following graph illustrating the relation between standard penetration test resistance (SPT/N) and internal friction angle (ϕ) value as proposed by CIRIA is used to determine the internal friction angle value for the soil. Considering Figure 6.5 and the graph given in Figure 6.6 below, internal friction angle is taken as ($\phi = 32^0$) for the loose to medium dense sand with an average SPT/N value of 17.

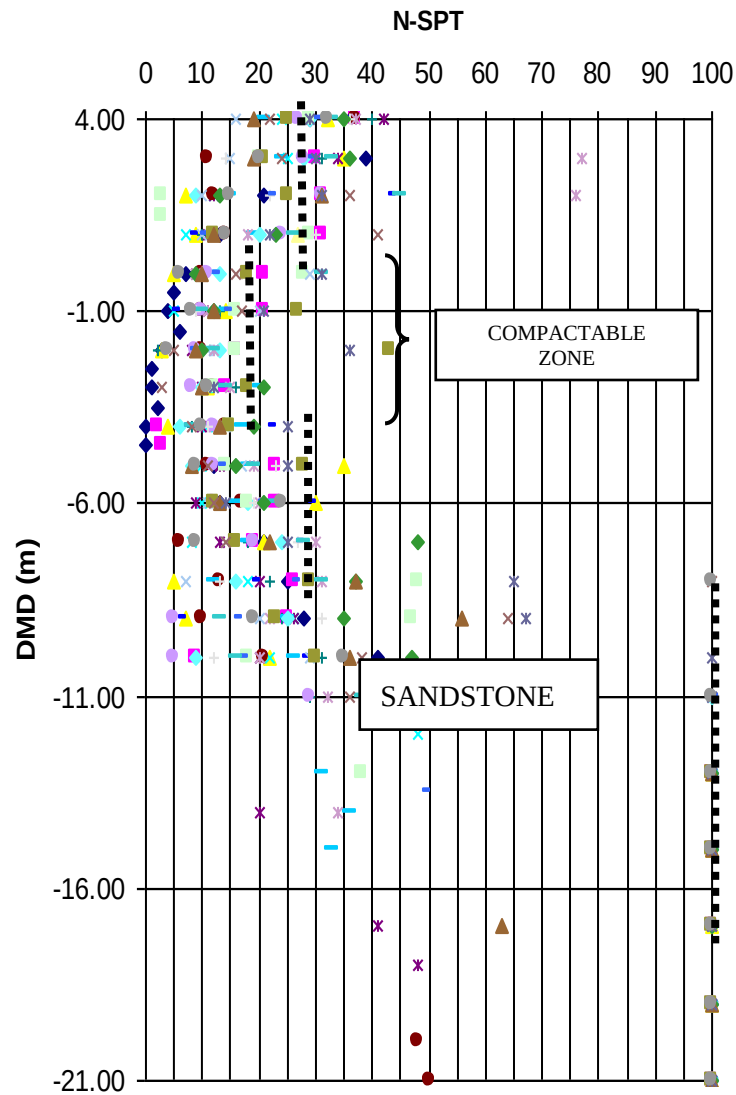


Figure 6.5. Variation of SPT/N values with depth

6.2.2.4. Layer No. 4,5,6,7,8,9 / Sand Stone. The side shear resistance is the primary parameter for the pile vertical capacity. Various empirical relations are available between side shear resistance and unconfined compressive strength (UCS) of rock and are presented in Chapter 2. The following Figure 6.7 plots the UCS values determined by laboratory tests on core samples and shows the selected representative average values for different depth intervals. The average UCS values to be utilized in the pile capacity calculations for the sandstone are summarized in the following Table 6.4.

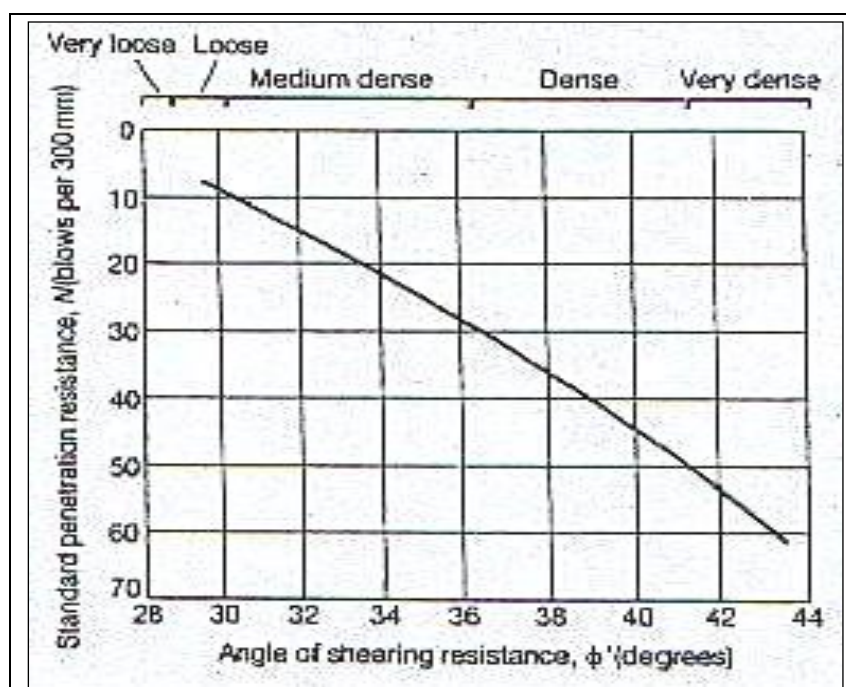


Figure 6.6. Correlation with SPT/N value and ϕ' (CIRIA Report)

Table 6.4. Sandstone average UCS results

From DMD	To DMD	UCS (MPa)
-11.00	-20.00	1.00
-20.00	-26.00	0.60
-26.00	-36.00	1.50
-36.00	-42.00	2.20
-42.00	-65.00	1.50
-65.00	-80.00	2.20

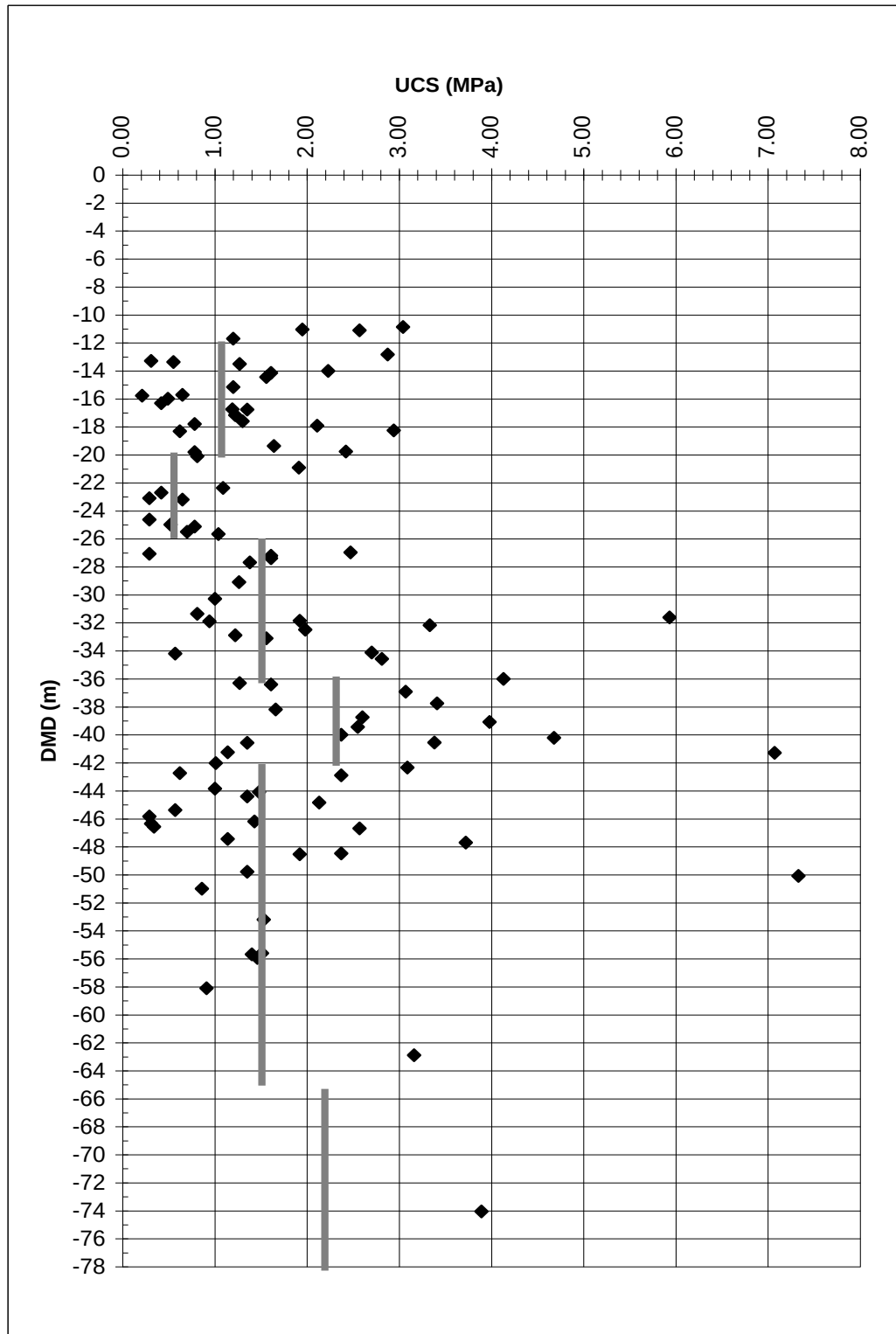


Figure 6.7. The variation of UCS values with depth Soil profile as determined by the above parameters is illustrated below at Figure 6.5.

<i>Layer No</i>	<i>DMD</i>	<i>Description</i>	
	3.25	Natural Ground Level (0.0 m)	
1	<i>GWT</i>	<i>GWT (-1.90 m)</i>	
2	-8.00		
3	-11.00	SAND	
4	-20.00	SANDSTONE = 1.00 Mpa	UCS
5	-26.00	SANDSTONE = 0.60 Mpa	UCS
6	-36.00	SANDSTONE = 1.50 Mpa	UCS
7	-42.00	SANDSTONE = 2.20 Mpa	UCS
8	-65.00	SANDSTONE = 1.50 Mpa	UCS
9	-80.00	SANDSTONE = 2.20 Mpa	UCS

Figure 6.8. Soil profile

6.2.3. Load Capacity and Settlement Calculations by Statical Analysis

The weight of this mega-structure creates large loads, so to safely carry these loads, deep foundation system including piling was accepted to be constructed on the site. Required number and the lengths (toe levels) of the piles were determined by the designer based on the value of total load acting to ground from the structure. First, a static method of analysis was applied to estimate the ultimate load bearing capacity of the piles. Since static load tests were conducted on a P4 type pile, static analysis was also made for the P4 pile type to allow a comparison between the results. The design working load acting on a P4 type pile from the structure was 10000 kN according to the project design. In the analysis, the capacity was obtained from several correlations and empirical correction factors developed from soil strength parameters determined by laboratory test results given in Section 2.2.3. Pile capacities relied only on bearing stresses of the sandstone strata. The skin friction through the upper sand layer was neglected. The equations used in the assessment of pile capacities by statical analysis are provided below:

$$Q_{allow} = \frac{Q_{ota}}{FS} \quad (6.1)$$

$$Q_{total} = Q_{skin} + Q_b \quad (6.2)$$

$$Q_{skin} = \sum_{i=1}^n f_{s,i} A_{s,i} \quad (6.3)$$

$$Q_b = q_b = N_\phi q_u \quad (6.4)$$

where,

Q_{allow} : Allowable Pile Capacity

Q_{total} : Maximum Total Pile Capacity

Q_{skin} : Bearing Capacity of Skin Friction

$Q_{s, layer}$: Skin friction capacity of the layer

Q_b : Bearing Capacity of Tip Resistance

f_s : Unit Skin Friction

A_s : Surface Area of Pile

q_u : Unconfined compressive strength

N_ϕ : Bearing capacity factor:

$$N_\phi = 1 + \frac{2\phi}{\pi} \quad (6.5)$$

Unit skin friction for rock:

$$f_s = \alpha q_u^\beta \quad (6.6)$$

Surface area of the pile:

$$A_s = \pi \cdot D \cdot L \quad (6.7)$$

where,

D : Pile Diameter

L : Layer length

To start with, unit skin friction of the rock layers were calculated as shown in Table 6.5 using the formulas presented in Section 2.2.3. In determining the design unit friction value, the average of the results of all the unit skin friction formulas excluding the maximum value was calculated, to be on the safer side. Since the upper sand layers were neglected in the analysis, unit skin friction of these layers were not calculated.

Table 6.5. Determination of unit skin friction of rock layers

Design Method	$f_s = \alpha \alpha_u^\beta$ (Mpa)		Unit skin friction of pile shafts in rock			
	α	β	Layer 4 Sandstone $q_u = 1.0$ (MPa)	Layer 5 Sandstone $q_u = 0.60$ (MPa)	Layer 6 Sandstone $q_u = 1.50$ (Mpa)	Layer 7 Sandstone $q_u = 2.20$ (MPa)
Horvath and Kenney (1979)	0.21	0.50	0.21	0.16	0.26	0.31
Carter and Kulhawy (1988)	0.20	0.50	0.20	0.15	0.24	0.30
Williams et al. (1980)	0.44	0.36	0.44	0.37	0.51	0.58
Rowe and Armitage (1984)	0.40	0.57	0.40	0.30	0.50	0.63
Rosenberg and Journeaux (1976)	0.34	0.51	0.34	0.26	0.42	0.51
Reynolds and Kaderbek (1980)	0.30	1.0	0.30	0.18	0.45	0.66
Gupton and Logan (1984)	0.20	1.0	0.20	0.12	0.30	0.44
Reese and O'Neill (1987)	0.15	1.0	0.15	0.09	0.23	0.33
Toh et al. (1989)	0.25	1.0	0.25	0.15	0.38	0.55
Migh and Wolshi (1979)	0.22	0.50	0.22	0.16	0.28	0.25
Horvath (1982)	0.20	0.50	0.20	0.15	0.24	0.30
		$f_s(\max)$	0.44	0.37	0.51	0.66
		$f_s(\min)$	0.15	0.09	0.23	0.30
		$f_s(\text{avg})$	0.26	0.19	0.35	0.45
		$f_s(\text{design})$	0.24	0.17	0.32	0.42

Next, skin friction capacity of all the sandstone layers were calculated and the sum of all the layers was determined to find the total skin friction capacity of the pile as shown in Table 6.6:

Table 6.6. Calculation of skin friction capacity

Layer (from pile cut-off)	Unit skin friction capacity (f_{design}) (kPa)	Pile Diameter (D) (m)	Layer Length (L) (m)	Friction capacity of the layer ($Q_{s, \text{layer}}$) (kN)
4	240	1.2	8.0	7234.56
5	170	1.2	6.0	3843.36
6	320	1.2	10.0	12057.60
7	420	1.2	6.0	9495.36

Bearing capacity of skin friction (Q_{skin}) = total $Q_{s, \text{layer}}$ = 32600 kN

Then, bearing capacity of the tip resistance was calculated. Soil conditions presented in Section 6.6.2 describes that the rock layer below the cut-off level of the pile is generally weak and medium fractured sandstone. Therefore, according to Table 2.3 in Section 2.2.3, the friction angle would be taken as 27° , and bearing capacity factor was calculated as:

$$N_6 = \frac{1}{2} \gamma \tan^2(45^\circ - \frac{\phi}{2}) = \frac{1}{2} \gamma \tan^2(45^\circ - \frac{27^\circ}{2}) = 36 \quad (6.8)$$

The average UCS value to be used in the calculation was be 1.33 Mpa, as determined by the ratio of total length to total UCS values of the layers presented in Table 6.7.

Table 6.7. Estimation of average (q_u) value to be used in design

Layer No.	q_u (Mpa)	Layer length (m)	$Q_u \times \text{length}$
4	1.0	8.0	8.0
5	0.6	6.0	3.6
6	1.5	10.0	15
7	2.2	6.0	13.2
		$\Sigma 30$	$\Sigma 39.8$
			$q_{avg} = \frac{39.8}{30} = 1.33 \text{ Mpa}$

Therefore;

$$Q_{tip} = q_b = 2N_\phi q_u = (226613) \times 7100 \text{ (kPa)} \quad (6.9)$$

And finally, the ultimate pile capacity was determined:

$$Q_{total} = Q_{skin} + Q_{tip} = 32631 + 7100 = 39700 \text{ kPa.} \quad (6.10)$$

It is seen that, most of the capacity is developed due to socket skin resistance. Table 5.4 in Section 5.7 was used for the estimation of the safety factor value. According to PDCA code, for the static analysis the safety factor is suggested to be taken as 3,5 because of the large risk due to poor prediction accuracy. So, the allowable working load of the pile was calculated as:

$$Q_{allow} = \frac{Q_{total}}{3.5} = \frac{39700}{3.5} = 11314 \text{ kN} \quad (6.11)$$

Pile's dead weight was also considered in the design. By using the unit weight of concrete as 25 kN/m^3 and the unit weight of steel as 80 kN/m^3 , for a P4 pile type of 1,2 meter diameter and 30 m. length;

$$Q_o = Q_{concrete} + Q_{steel} = 847.8 + 50 = 900 \text{ kN} \quad (6.12)$$

Therefore, the allowable capacity becomes:

$$Q_{allow} = 11314 - 900 = 10400 \text{ kN} \quad (6.13)$$

By checking that allowable pile load was greater than pile design working load value of 10000 kN which is approximately 1000 tons, it was concluded that this pile design would resist the loads so acceptable.

When estimating the settlements on working load, below equation presented in Section 3.2 was used:

$$S = s_e + s_p + s_s \quad (6.14)$$

where,

s_e = elastic compression of pile

s_p = settlement of pile due to the end-bearing of pile into the soil

s_s = settlement of pile due to the frictional resistance build up.

In these equations, elastic modulus of the pile was taken as elastic modulus of concrete; 29 GPa. Elastic modulus of the soil was taken as the deformation modulus of the rock mass surrounding the shaft (E_d), and the equation presented in Section 3.2 was used to determine E_d :

$$E_d = j \times M_r \times q \quad (6.15)$$

where,

j = Mass factor = 0.2 (Table 2.1, since the rock is weak and medium fractured as described in soil report described in Section 6.2.2).

M_r = 300 (according to Table 3.1, since the sandstone layer described in Section 6.2.2 can be categorized as Group 2).

q_c = 1.33 Mpa (the average unconfined compressive strength related to depth was taken, using Table 6.7).

Therefore,

$$E_d = 0.2 \times 300 \times 1.33 \quad (6.16)$$

Using these values, first elastic settlement for design working load was calculated:

$$Q_{wp} = \frac{7100}{3} = 2366 \text{ k} \quad (6.17)$$

$\xi = 0.5$ (friction resistance distribution is similar to Figure 3.8b).

$$Q_{ws} = \frac{3263}{3} = 1087 \text{ k} \quad (6.18)$$

$L = 30$ meters.

$$A = \frac{\pi D^2}{4} = 11.3 \text{ m}^2 \quad (6.19)$$

$E = E_{\text{concrete}} = 29 \text{ Gpa.}$

Using these values, elastic settlement in working load was calculated as:

$$S_e = \frac{(Q_{wp} + \xi Q_{ws})L}{AE} = \frac{(2366 + 0.5 \times 1087) \times 30}{(1.29 \times 10^6)} = 6.71 \text{ mm} \quad (6.20)$$

Next, plastic settlement was calculated:

$D = 1.2 \text{ m.}$

$$Q_p = \frac{2029}{1.3} = 1560 \text{ k} \quad (6.21)$$

$I_{wp} = 0.85.$

$\mu_s = 0.30$ (Section 3.2).

$$E = 29 \text{ GPa.}$$

$$s_p = \frac{Q_p D}{E_s} (1 - \mu_s) = \frac{(1.2)(93.2)}{29000000} \quad (6.22)$$

Last, side settlement due to frictional resistance was calculated:

$$Q_{ws} = \frac{32630}{5} = 6526 \text{ k} \quad (6.23)$$

$$p = \tau D = 37 \text{ k} \quad (6.24)$$

$$L = 30 \text{ meters.}$$

$$D = 1.2 \text{ meters.}$$

$$L_{ws} = 20 \sqrt{\frac{30}{12}} = 37 \quad (6.25)$$

$$\mu_s = 0.30 \text{ (Section 3.2)}$$

$$E_d = 80 \text{ MPa.}$$

$$s = \frac{Q_{ws} D}{E_s} (1 - \mu_s) = \frac{(6526)(1.2)}{(80)(10^6)} = 9.79 \times 10^{-5} \text{ m} \quad (6.26)$$

Same equations were used to calculate design settlements under 1.5 times and 2 times design working load values. As a result, following Table 6.8 was presented including all displacement values under increasing loads.

For comparison of results, settlement of the pile at design load was also calculated using equation 6.27 which assumes that load is carried by rock socket skin friction only:

$$settlement = \frac{Q}{B_d} = \frac{(100801.3)}{(12800)} = 13.5 \text{ mm} \quad (6.27)$$

This result is close to the result calculated by the general settlement formula of 10,4 mm. In this equation, influence factor (I_p) was estimated using Figure 3.9 shown in Section 3.2.

Table 6.8. Settlement calculations for P4 pile type

P4			
E = 29,000,000 kN/m²	Diameter 1200 mm. ; Pile Length = -42.2 m		
	Q_{design}	1.5 x Q_{design}	2.0 x Q_{design}
Load =	10,400 kN	15,600 kN	20,800 kN
Side settlement (mm) =	4.2	6.3	8.4
Tip settlement (mm) =	0.1	0.1	0.1
Elastic settlement (mm) =	6.1	9.2	12.3
Total (under full load) (mm) =	10.4	15.6	20.8
Total (residual) (mm) =	4.2	6.3	8.4

6.2.4. Design Layout

Pile type schedule determined by the designer is provided below at Table 6.9. The designed pile layout is shown in Figure 6.9.

Table 6.9. Pile schedule table

Pile No	Pile Diameter (mm)	Cut off level (DMD)	Pile Length (m)	Toe Level (DMD)	Concrete grade	Pile working load (kN)
P1	1000	-12.00	20.0	-32.00	40	6000
P2	1000	-12.00	23.0	-35.00	40	7100
P3	1200	-12.00	23.0	-35.00	40	8200
P4	1200	-12.00	30.0	-42.00	40	10000
P5	1500	-12.00	24.0	-36.00	40	12500
P6	1500	-12.00	30.0	-42.00	40	14500
P7a	1500	-12.00	34.0	-46.00	60	18000
P7b	1500	-16.10	34.0	-50.10	60	18000
P8a	1500	-16.10	34.0	-50.10	60	20000
P8b	1500	-13.80	34.0	-47.80	60	20000
P9a	1500	-16.10	43.0	-59.10	60	22000
P9b	1500	-16.80	43.0	-59.80	60	22000
P9c	1500	-13.80	43.0	-56.80	60	22000
P10a	1500	-16.10	46.0	-62.10	60	24000
P10b	1500	-16.80	46.0	-62.80	60	24000
P10c	1500	-13.80	46.0	-59.80	60	24000

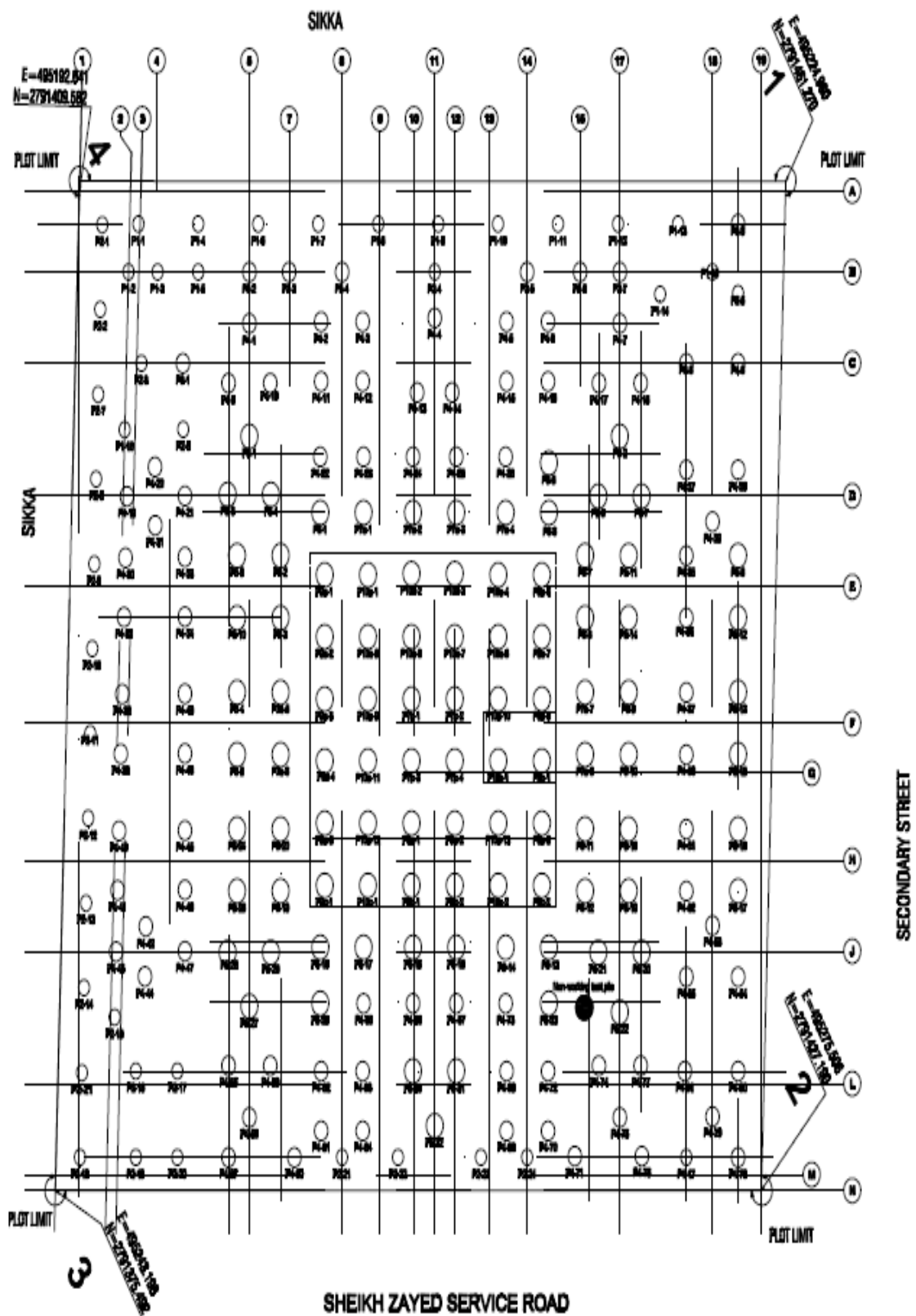


Figure 6.9. Site pile layout

6.2.5. Static Load Testing-1

A preliminary static load test was conducted on a 1200 mm. diameter test pile in order to assess the capacity and settlement behavior of the pile under design working load. The test set-up of the working pile full scale load test is shown in Figure 6.10 below. Pile was tested under compression up to 200 per cent of the working load. Four reaction piles were used for test pile to provide the reaction system to apply the require test load.

The static load was applied and maintained by means of three hydraulic jacks having a capacity of 10,000 kN and were measured with a load cell. The jack was connected to control unit where the hydraulic power was generated by electricity and also where the pressure in the system was monitored by built in manometers. The settlements were read by four micrometers for the compression pile and by one micrometer for each tension pile connected to independent reference beams. The micrometers used had a range of 0 – 50 mm. and precision of 0,01 mm. The displacement monitoring set-up is illustrated in Figure 6.11.

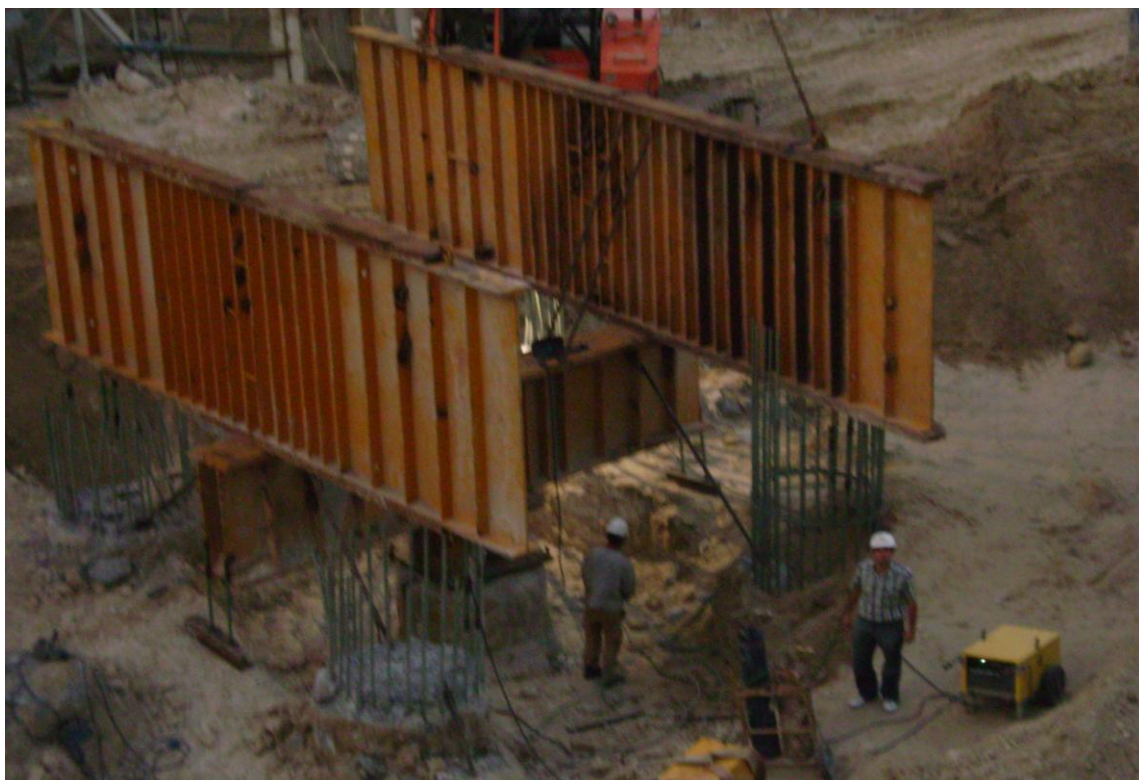


Figure 6.10. Construction of test loading system



Figure 6.11. Micrometers used for displacement readings

The settlement readings are presented at Appendix A. The test results are presented in two graphs based on the settlement readings, first graph illustrates the load-deformation curve and second time-deformation curve in Figure 6.12 and 6.13, respectively. The following results were obtained from the test:

- The maximum settlement up to 100 per cent of the working load (which is 10,000 kN) was 1,2 mm.
- The residual displacement after releasing the load from 100 per cent down to zero was 0,4 mm.
- This first loading and unloading cycle indicates that a normal working pile was expected to experience a maximum vertical displacement of 0,5 – 1,0 mm.
- The loading was continued up to 150 per cent of the working load, namely up to 15,000 kN. The displacement at this load level was 2,7 mm and the residual displacement after unloading to zero was 0,8 mm.

- The loading was continued up to 200 per cent of the working load, namely up to 20,000 kN. The displacement at this load level was 5 mm. and the residual displacement after unloading to zero is 2,2 mm. as presented in Table 6.10.

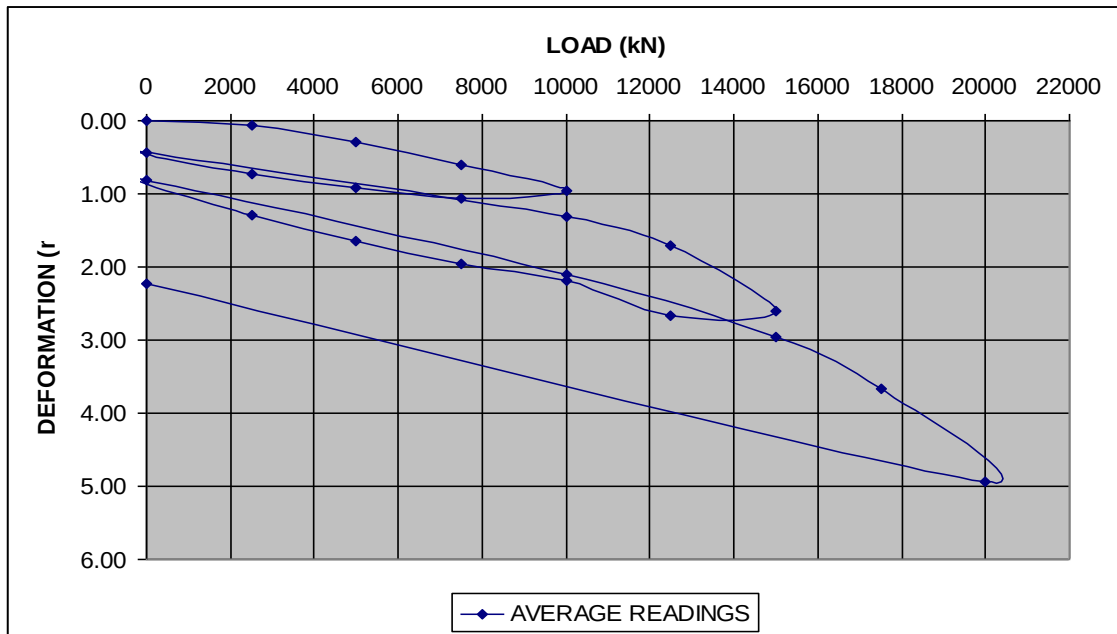


Figure 6.12. Load - Displacement curve

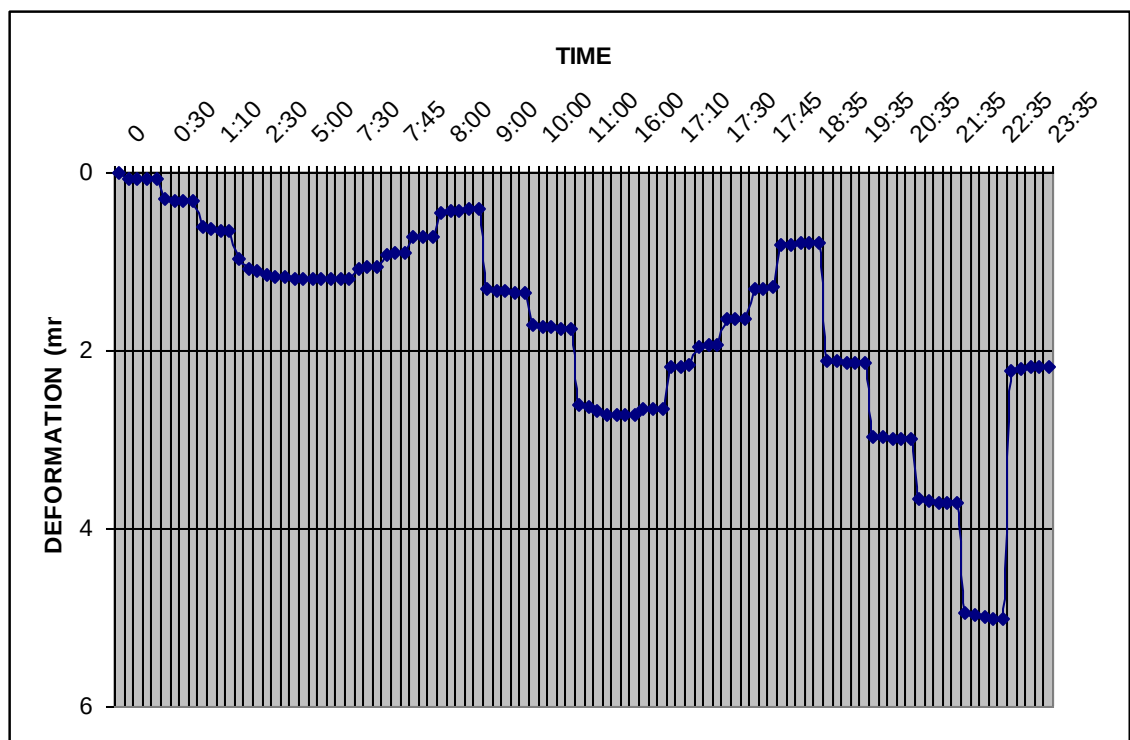


Figure 6.13. Time - Displacement curve

To sum up, the working test pile of diameter of 1200 mm was loaded up to 200 per cent of the working load and settlements of the pile under various load steps were recorded. The pile did not fail up to this load level and it was proven by this full scale test that the service working load of 10,000 kN was carried safely for the designed pile tip elevation corresponding to certain socket length.

Table 6.10. Settlement criteria

Serviceability limit for working load = 10,0 mm.	Test Value (mm)	Design Value (mm)
Total settlement under 2.0 x W.L. (mm)	5,0 (O.K.)	20,8
Residual Settlement (mm)	2,2 (O.K.)	8,4

6.2.6. Static Load Testing-2

In addition to the Static load test-1, one more preliminary static load test was conducted on the site to another 1200 mm. diameter pile with same design characteristics, but now including a vibrating wire strain gauge and load cell system, to evaluate also the load distribution at the strain gauge levels and resultant skin friction along pile shaft. Undoubtedly, this upgraded testing system would give more precise information of design parameters than the first conventional static loading test. Figure 6.14 shows the test preparation for the pile.

The scope of the test works consisted of the installation of vibrating wire strain gauges for four different levels with four strain gauges per level, which were all pre-tensioned and sealed as shown in Figure 6.15. The strain gauges were monitored during compressive load test of the pile for different loading stages. Next, two 500 tonne load cell and one 1000 tonne load cell to accurately measure the load applied to the top of the pile were used. A 30 channel data acquisition system (DAS) was used to monitor and display the signals from the above sensors. The DAS was controlled by a desktop computer operating the control software which processed the raw data from the sensors into engineering units and presented it in simple tabular format. On screen data was available during the test to enable the results to be assessed. Figure 6.16 shows the placement of micrometers.



Figure 6.14. Test pile preparation

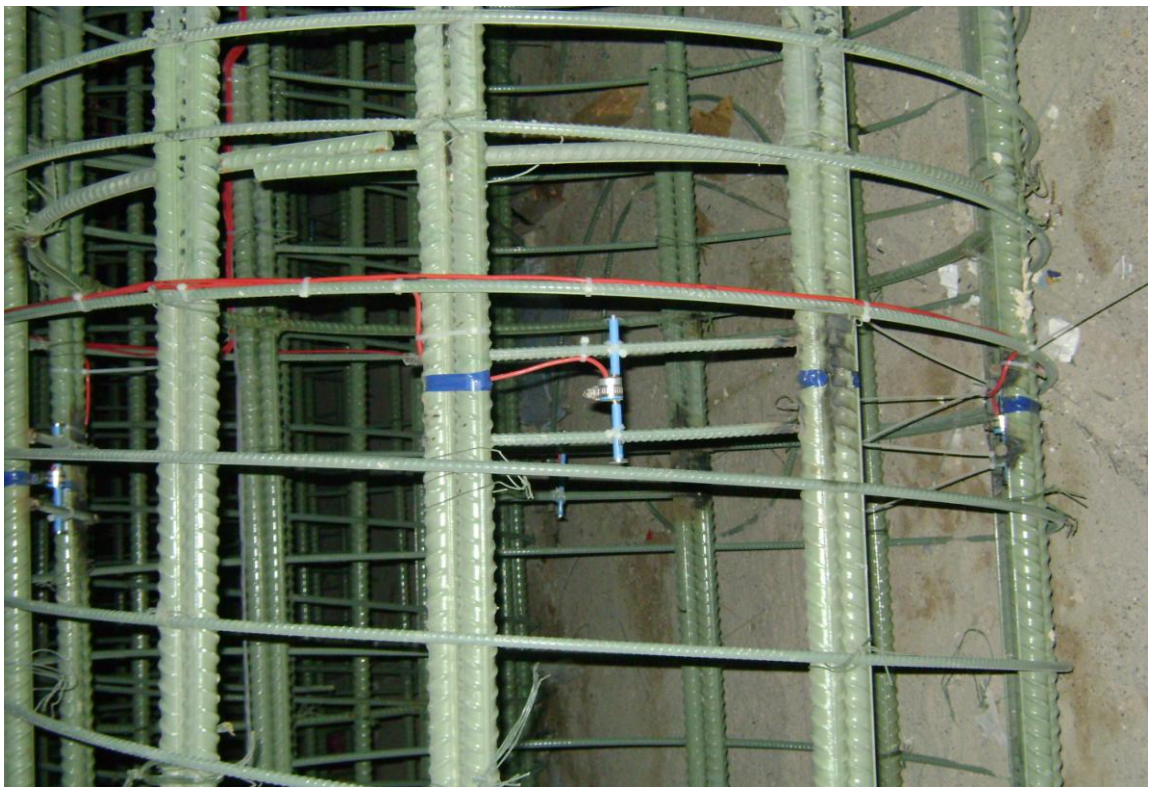


Figure 6.15. Fixing of strain gauges on steel cages



Figure 6.16. Micrometer preparations

Vibrating wire strain sensors were installed in bored cast-in-situ test pile at four different levels. Concrete was poured later, on the same day. The sensors were monitored at one minute intervals during the course of pile load test at zero loads and for each loading cycle. The readings were taken and stored in raw data files.

The strain measured in the pile at different levels is presented in Table 6.11.

Table 6.11. Microstrain ($E_z \times 10^6$) at different levels

Level (m DMD)	(5000 kN)	(10000 kN)	(15000 kN)	(20000 kN)
	50%	100%	150%	200%
-14.50	-50.9	-206.0	-376.7	-554.3
-24.50	-16.6	-67.1	-122.7	-180.5
-29.50	-5.2	-29.3	-64.1	-102.2
-34.50	-0.2	-2.2	-5.9	-9.3

Note: - indicates compressive strain

The calculated load transferred to the pile at the strain gauge levels are presented in Table 6.12:

Table 6.12. Calculated load transferred to the pile (kN)

Level (m DMD)	(5000 kN)	(10000 kN)	(15000 kN)	(20000 kN)
	$E_c = 29.7 \text{ kN/mm}^2$			
	50%	100%	150%	200%
-14.50	-1739	-7037	-12868	-18932
-24.50	-566	-2292.0	-4191	-6166
-29.50	-179	-1000.4	-2188	-3489
-34.50	-8,0	-77	-201	-319

Note: - indicates compressive load

The load was calculated as follows (Fellenius, 2001):

$$\text{Load} = \text{Average strain} \times [(\text{Area of steel} \times E_s) + (\text{Area of concrete} \times E_c)] \quad (6.28)$$

where,

Young's modulus of elasticity for concrete; E_c is assumed to be 29.7 kN/mm^2 .

Young's modulus of elasticity for steel; E_s is assumed to be 210 kN/mm^2 .

Total cross sectional area = 1130973.4 mm^2 .

Area of steel = 3141.6 mm^2 .

Net concrete area = 1127831.8 mm^2 .

Vertical load developed in the area between levels is shown in Table 6.13:

Table 6.13. Load between levels (kN)

Level	(5000 kN)	(10000 kN)	(15000 kN)	(20000 kN)
	$E_c = 29.7 \text{ kN/mm}^2$			
	50%	100%	150%	200%
1 to 2	1173	4745	8677	12765
2 to 3	387	1292	2003	2677
3 to 4	172	924	1987	3170
4 to toe	8	77	201	319

Where,

$$\text{Load between levels} = (\text{Load at upper level} - \text{Load at lower level}) \quad (6.29)$$

Percentage of applied load between levels is shown in Table 6.14 :

Table 6.14. Percentage of applied load between levels (%)

Level	(5000 kN)	(10000 kN)	(15000 kN)	(20000 kN)
	$E_c = 29.7 \text{ kN/mm}^2$			
	50%	100%	150%	200%
1 to 2	23	47	68	64
2 to 3	8	13	13	13
3 to 4	3	9	13	16
4 to toe	0	1	1	2

Where, for 50 per cent of loading:

$$\frac{L_{\text{between}}}{5000} \quad (6.30)$$

Unit skin friction along pile shaft for different applied loads are provided in Table 6.15 :

Table 6.15. Unit skin friction (kN/m^2)

Level	(5000 kN)	(10000 kN)	(15000 kN)	(20000 kN)
	$E_c = 29.7 \text{ kN/mm}^2$			
	50%	100%	150%	200%
1 to 2	31.1	125.9	230.2	338.6
2 to 3	34.2	114.2	177.1	236.7
3 to 4	15.2	81.7	175.7	280.3
4 to toe	4.0	40.7	106.6	169.4

Skin friction was calculated using the formula:

$$\frac{U_{\text{piled}}}{S_{\text{unf}} + U_{\text{piled}}} \quad (6.31)$$

Displacement of pile under compressive load is shown in Tables 6.16 and 6.17; provided by dial gauges results.

Table 6.16. Displacement values of the test pile

Displacement (mm) – Test Pile				
Dial Gauges	50 %	100 %	150 %	200 %
A	1.0	3.1	5.3	8.2
B	0.9	2.5	4.1	6.3
C	0.6	2.1	4.9	8.1
D	0.6	2.1	3.7	5.8
Average Displacement	0.7	2.5	4.5	7.1

Table 6.17. Displacement values of the reaction pile

Displacement (mm) – Reaction Pile				
Dial Gauges	50 %	100 %	150 %	200 %
E	0.7	2.5	4.6	7.1
F	0.1	1.9	6.3	9.7
G	0.2	3.4	6.3	9.4
H	0.1	1.0	2.4	3.9

The results obtained from the test are presented in the Tables 6.11 to 6.17. The outcome indicates good measurement system functionality. The sensors show correct operation and results are consistent with what was expected. The average settlement measured for the test pile under ultimate load is 7,1 mm and the residual settlement when all the load is released is 2,3 mm indicates that the pile should be able to carry the test load with acceptable displacements. Appendix B shows displacement readings on site.

6.2.6. Results

In Case study-1, first a static analysis was made to a designed pile of 1200 mm diameter and 30 m. depth. In the analysis, first total ultimate pile capacity of the pile was calculated using both the correlations and equations presented in Section 2.2.3 and the soil, rock conditions and parameters described in Section 6.2.2. Next, allowable pile capacity was estimated by dividing the ultimate capacity to a selected safety factor of 3,5 from Table 5.4 and was compared with the service load of 10000 kN. Last, settlement values were calculated under 100 per cent, 150 per cent and 200 per cent of the working load values with the aid of the equations and tables presented in Section 3.2 and were compared with static test results. In the end, it was observed that the calculated allowable working load was higher than design working load and displacements were at critical level according to performance criteria, so design parameters were at allowable limit.

Next, two preliminary static loading tests were conducted on the site to two piles of 1200 mm. diameter and with same design parameters. The first test was a conventional top down loading test using micrometers and so only giving displacement values for capacity prediction and the second one was a static test with using load cells and four strain gauges instrumented in the pile, giving transferred load between gauge levels and skin friction values for capacity prediction as well as displacement values. The service load of 10000 kN was used in the test for comparison. Both static load tests conducted on the pile were maintained load test (ML), in which pile failure rarely occurs. Evidently, both piles did not reach the plunging mode and no rapid movement occurred under sustained applied loads on the tests. Therefore, total bearing capacity value could only be estimated by some definition based on the load-movement data recorded in the test. By interpreting the load-movement curve of Static Test-1 above, the capacity value could be estimated. As a result, mathematical methods of Davisson offset limit load method (1972), DeBeer yield load method (1968) and Hansen 80 per cent criterion method (1963) presented in Section 5.2.7 were used to define the pile capacity respectively.

- Load capacity determination by Davisson Offset Method (1972):

To start with, the offset limit was determined by the Davisson Offset Limit load formula (Chapter 5.2.7):

$$\text{Offset limit} = (0.15 \text{ inch}) + \frac{D_{\text{iameter}}}{120} = 0.15 + \frac{45}{120} = 0.525 \text{ inch} = 14 \text{ mm.} \quad (6.32)$$

The offset limit was added to the load-displacement curve of Static Load Test-1. Since the capacity had not reached the ultimate in 200 per cent loading of design load which corresponds to 20800 kN, the displacement values after this loading cycle were extrapolated theoretically assuming that a ratio between the increasing displacement values with load was present and the increase was parabolic. So, following Figure 6.17 was evaluated from Figure 6.12. The equation for the estimated parabolic curve was determined as:

$$y = 0.0001x^2 - 0.001x + 0.001 \quad (6.33)$$

According to the figure, for the 1200 mm diameter pile, the ultimate load was approximately 27000 kN, which is the intersection point of the curve with linear offset limit line, that is parallel to linear, elastic part of the curve.

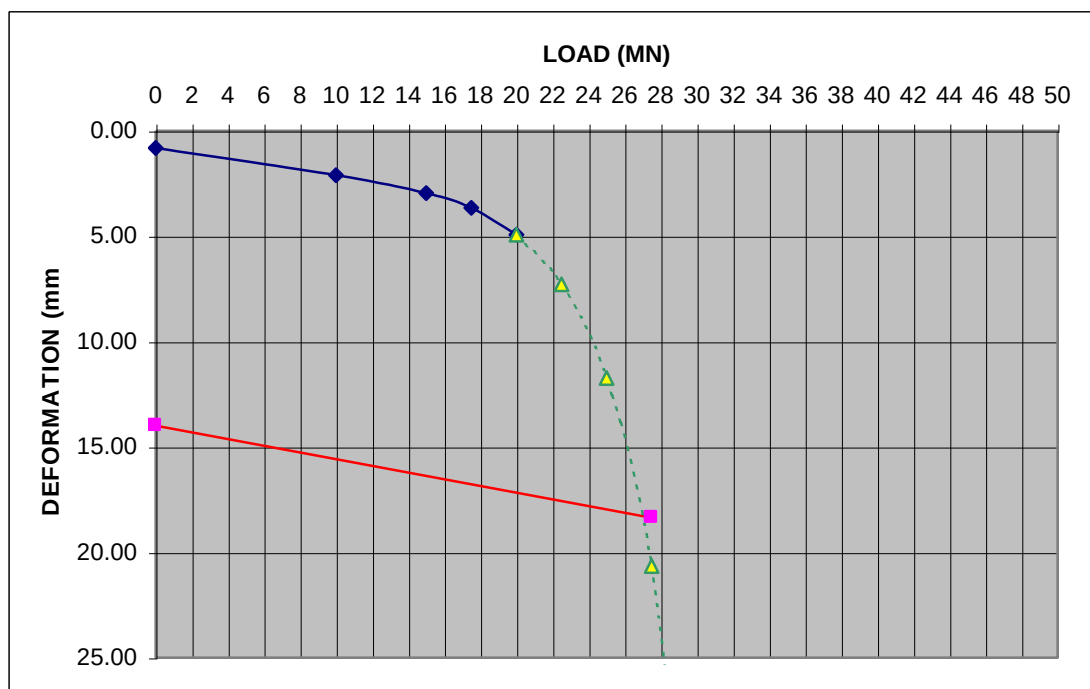


Figure 6.17. Determining the ultimate load by the Davisson Offset Limit load method

- Load capacity determination by DeBeer yield load method (1968):

According to the method, the load-displacement data was plotted in a logarithmic scale rather than a linear scale. If ultimate load is reached in the test, two line approximations appear; one before and one after the ultimate load (Fellenius, 2001). So, in the Figure 6.18 presented below, the ultimate load was estimated to be 26500 kN, since it is clearly figured that below and above that point, the logarithmic lines are in different directions.

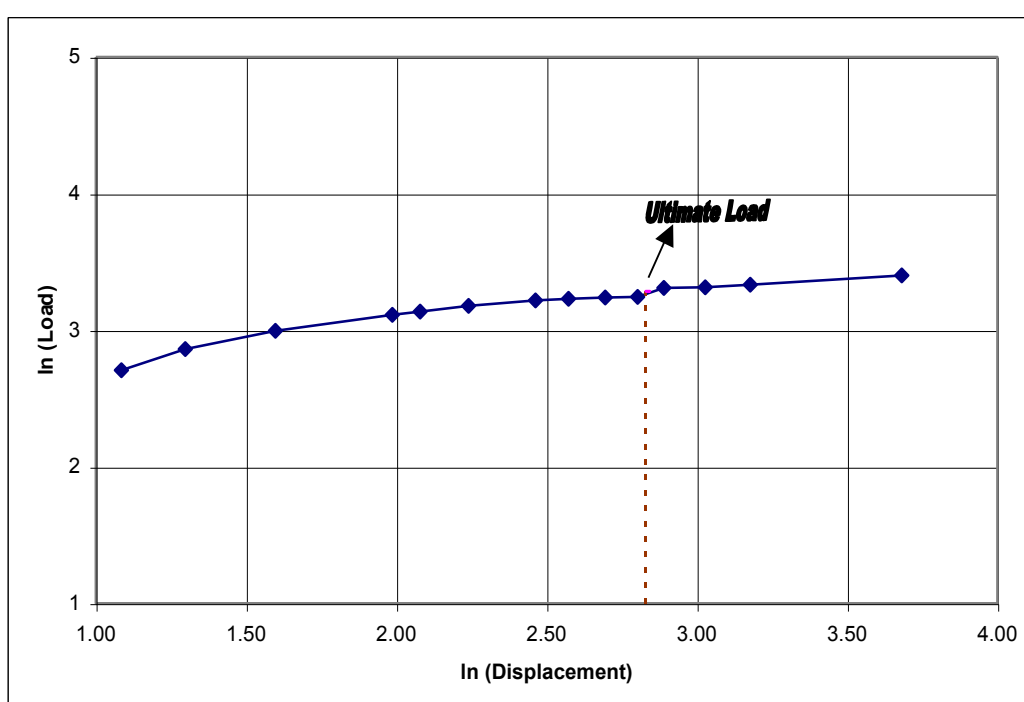


Figure 6.18. Determination of the ultimate load by DeBeer Yield Load method

- Load capacity determination by the Hansen 80-per cent criterion (1963):

This method proposes that the ultimate load capacity is the load which gives four times the displacement of the pile head as obtained for 80 per cent of that load (Fellenius, 2001). By investigating the extrapolated load-displacement curve presented in Figure 6.17, it was estimated that the load of 28500 kN corresponds to the required criteria so selected as the ultimate load.

Table 6.18 presented below shows the comparison of allowable loads determined by the methods presented above with statical analysis results.

Table 6.18. Comparison of allowable load capacity values

	Ultimate Load Capacity (kN)	Safety Factor	Allowable Load Capacity (kN)
Statical Anaysis (DESIGN)	39,700	3,50	10,400
Davisson Offset Method	27,000	2,0	13,500
DeBeer Yield Load Method	26,500	2,0	13,250
Hansen 80 % Criterion	28,500	2,0	14,250

Results showed that all the methods to determine allowable load capacity by extrapolation of load-displacement curve of the conventional static load test present similar values, which are higher than the allowable capacity calculated by statical analysis. However, interpreting the results from a static load test to obtain an ultimate load larger than the maximum load applied to the pile in the test is not recommended due to high amount of assumptions (Fellenius, 2001).

Therefore, more and more frequently, the conventional test arrangement is expanded to include instrumentation to obtain the required information as in Static Test-2. Normally, the instrumentation consists of strain gauges and load cells placed at selected levels to determine the load at that location for each load applied to the pile head. The gages are used to measure strain and the strain is converted to load. The graphs shown below in Figures 6.19, 6.20 and 6.21 present microstrain, calculated load and skin friction value distributions along the pile shaft.

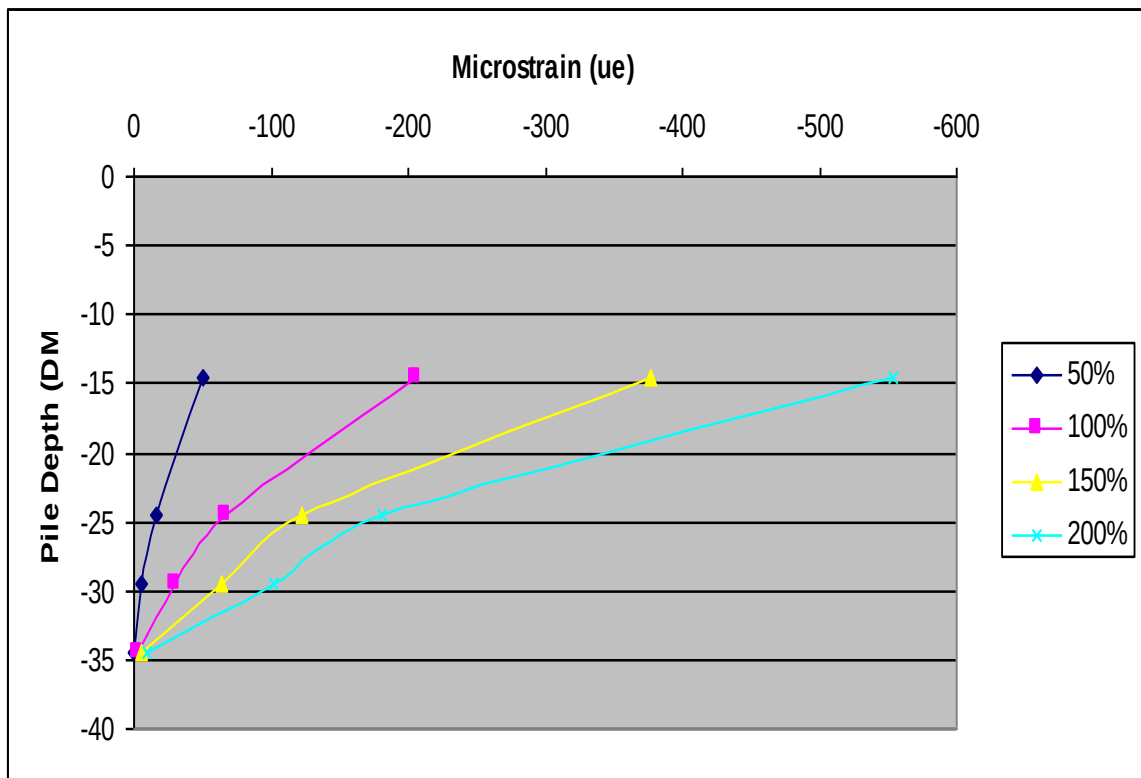


Figure 6.19. Microstrain distribution along the pile shaft

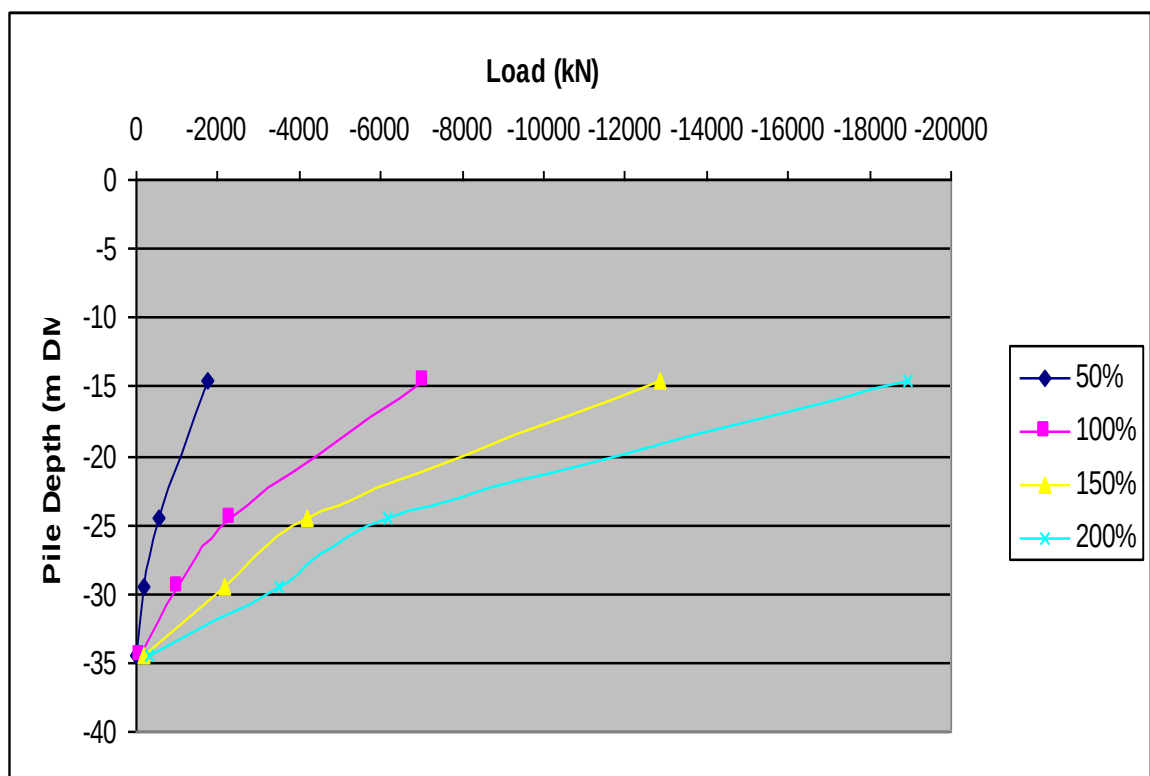


Figure 6.20. Calculated load distribution along the pile shaft

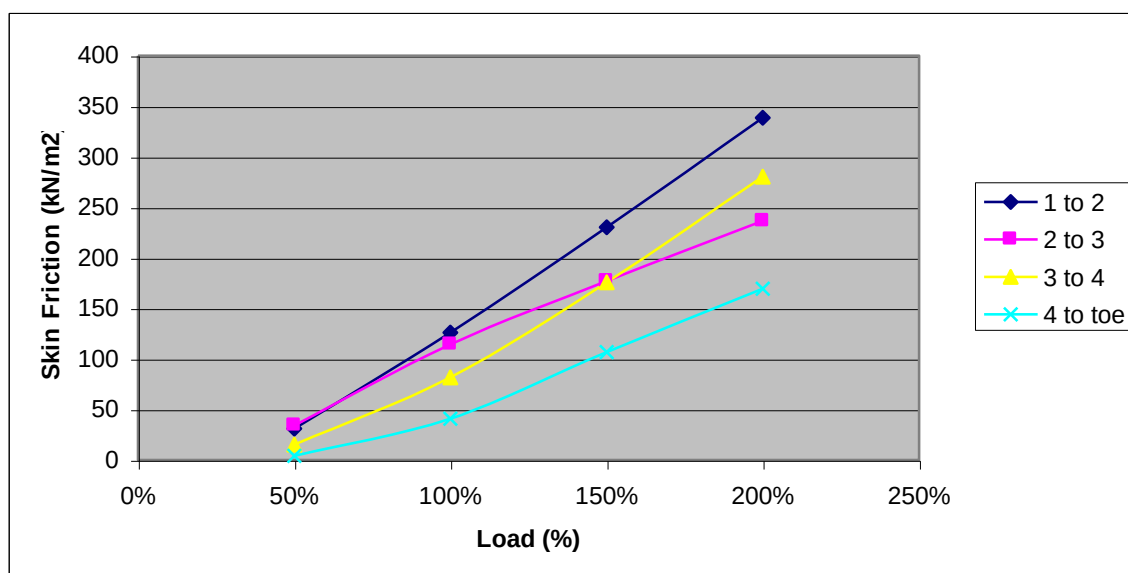


Figure 6.21. Skin friction distribution along the pile shaft

When the results were analysed, it was observed that almost all the load was carried by skin friction and very little of the load had reached to the pile toe, on the other hand in statical analysis a considerable amount of base resistance was calculated to be present in the pile. The ultimate unit skin friction value ranged from 169.4 kN/m^2 to 338.6 kN/m^2 . It should also be considered that before the start of the test, all dial gauges were set to zero, assuming no residual load present in the pile, but for a pile affected by residual load, strains will exist in the pile before the start of the test. Test data was still difficult to interpret the load applied to pile toe in the Static Test-2. Although it is obvious that results of Static Test-2 presented load-transfer mechanism of the pile, the ultimate resistance had not been achieved. Skin and toe resistances were not separately known and again were only assumed. Besides, the modulus of the concrete which was assumed constant in the interpretation can in fact vary within a wide range and is a function of the mobilization of shaft resistance down the pile, causing an error of about 10 per cent to 20 per cent in shaft resistance determined from the measurements. The load-movement on the pile toe and the compression value of the pile could not be clearly determined by conventional static load testing and test results were compared by displacement analysis rather than pile capacity. Table 6.19 presents comparison of displacement values for statical analysis and static load test methods. Total observed displacement values under working loads are smaller than the calculated values in the statical analysis and under serviceability limits (Fellenius, 2001).

Table 6.19. Displacement criteria

Serviceability limit for working load = 10,00 mm.	Static Test-1 Value (mm)	Static Test-2 Value (mm)	Design Value (mm)
Total settlement under 2.0 x W.L. (mm)	5,0 (O.K.)	7,1 (O.K.)	20,8
Residual Settlement (mm)	2,2	2,3	8,4

The higher than expected value of skin friction would lead to more economical pile design. If we had intended to decrease the pile length by four meters, by assuming that the highest calculated unit skin friction of 338,6 kPa was effecting the pile at this level instead of the calculated unit skin values in statical analysis, the new total skin friction at that level would be:

$$Q_{s2} = 338,6 \times 4 \times 90 = 121908 \text{ kN} \quad (6.34)$$

And new allowable load capacity was evaluated, ignoring toe resistance and taking the safety factor as 2.0 due to the reduced assumptions (Table 5.4) as:

$$Q_a = \frac{Q_{s2}}{2} = \frac{121908}{2} = 60954 \text{ kN} \quad (6.35)$$

which was an allowable value since it was higher than the service load of 10000 kN. If we compare the cost of these static tests to the profit by reducing the length of all the 1200 mm diameter piles by four meters on the design in Table 6.20;

Table 6.20. Economic analysis for static load test

	Number of piles	Cost per meter (\$)	Cost per pile (\$)	Total cost/profit (\$)
1200 mm. dia. pile	90	217	4 x 217 x 90 = 78,120	+ 78,180
Static load test	1	-	62,670	- 62,670
Static load test with strain gauges	1	-	99,455	-99,455

As a result, conducting a preliminary static load test alone on this site is uneconomical. The test costed more than its benefit on pile design, besides so much time was wasted.

6.3. Case Study 2: ACC Mixed Use Development Site, Dubai, U.A.E.

The proposed Mixed Use Development Tower for The Office of H. H. Saeed Bin Zayed Al Nahayyan is designed as a 1,500,000 square foot high-rise development divided into two distinctive, slender towers; one 53 stories and one 60 stories with one basement and one ground floors, situated on each side of a multi-story atrium. The Suites Tower houses hotel and luxury service apartments, while the Residential Tower houses rental apartments. The project site is located at Plot No. 345-346, Dubai.

6.3.1. General

Soil investigation was carried by GEOCONSULT and the geotechnical report presents the overall foundation engineering evaluations utilizing the data from six boreholes to a depth 50.00 meters and five boreholes to a depth 40.00 meters. Figure 6.22 illustrates a design overview of the proposed structure.



Figure 6.22. Mixed use development tower design overview-1

Two hydraulic drilling machines; IMT AF220 and Bauer BG28, two crawler cranes of Hitachi KH180 model, two de-sanders and two pumps used to deliver bentonite into de-sanders on bentonite tanks from borehole were used in the construction. Figure 6.23 presents a picture from construction in day shift and Figure 6.24 in night shift.



Figure 6.23. Construction in day shift



Figure 6.24. Construction in night shift

In the project, a total of 274 piles were designed. 217 of these piles were 120 cm, 39 of them were 150 cm. and 18 of them were 90 cm. in diameter. Pile schedule, designed according to the service load values acting from the upper structure, are presented below in Table 6.21. Figure 6.25 shows the pile layout of Suites Tower side and Figure 6.26 the Residential Tower side according to the design. This study concentrated on the comparison of capacity and displacement values determined by static method of analysis and preliminary osterberg load testing with each other and designed pile parameters to check the allowability of the design. Moreover, their effectiveness on pile design was analysed. A P2-1 coded pile type, with a 1200 mm. diameter and 40 m. length was chosen for the assessments. Table 6.22 presents the total testing schedule for designed piles in the project, including both preliminary testing and proof testing after construction for cost comparison.

Table 6.21. Pile schedule

[illegible]

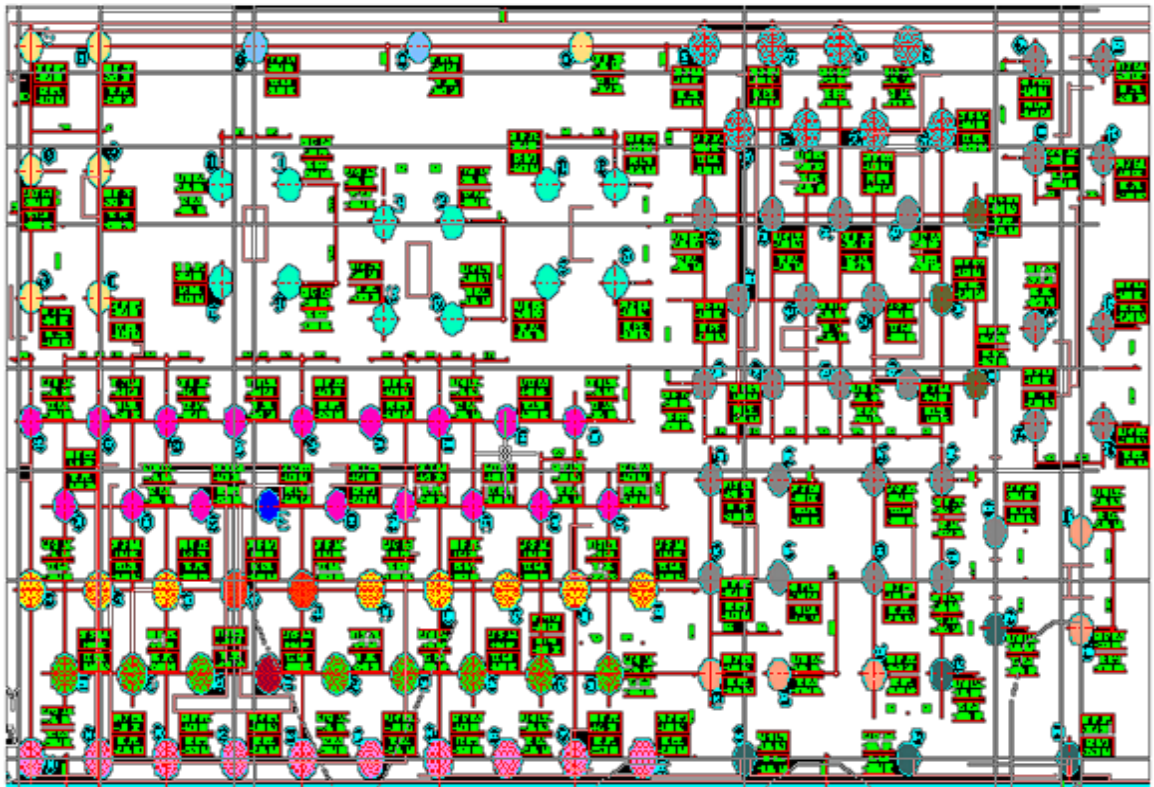


Figure 6.25. Pile layout of Suites tower side

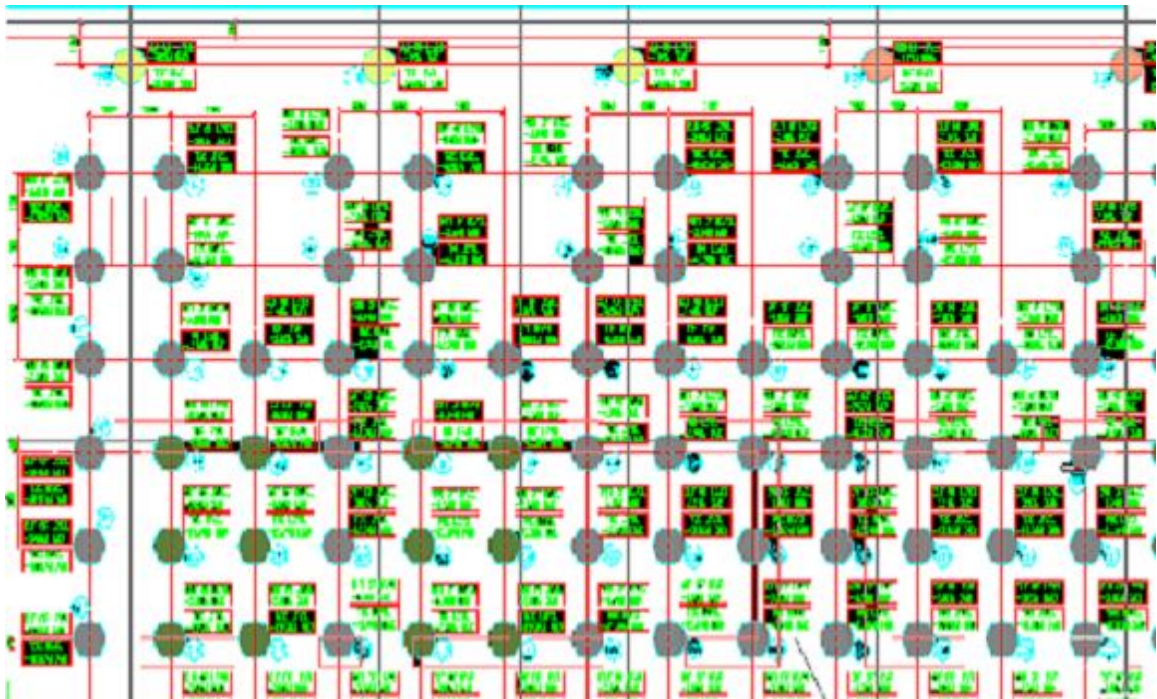


Figure 6.26. Pile layout of Residential tower side

Table 6.22. Pile testing schedule for piles

Pile Diameter (mm)	Number of Tests				Non-working piles
	Working piles				
	Low strain Integrity Test	Sonic logging test	Static Load Test	Dynamic Load Test	
1500	37	4	1	2	1
1200	219	22	2	11	1
900	17	2	1	1	1

According to the table, one per cent of all piles were static, five per cent of them were dynamic load tested. Next, 10 per cent of the piles were cross-hole sonic logging and 100 per cent of the piles were low strain integrity tested with pulse echo method. Also, three non-working piles from each diameter pile was osterberg load tested as preliminary piles. Table 6.23 presents the payment details for the piling program and Table 6.24 for the testing program.

Table 6.23. Payment details for total piling program

Item Ref.	ITEM DESCRIPTION	QUANTITY	Unit	Rate (\$)	Amount \$
PILING					
A	Allow for mobilization and demobilization for piling		Item	111.720	111.720
Boring and stabilisation of borehole					
B	For 900 mm diameter	434	m	159	68.825
C	For 1200 mm diameter	8.075	m	217	1.753.620
D	For 1500 mm diameter	1.583	m	370	585.320
Reinforced concrete					
E	Reinforced concrete as specified	12.200	m³	121	1.476.000
REINFORCEMENT BARS					
F	900mm diameter	1.500	Ton	1.172	1.757.500
G	1200 mm diameter				
H	1500 mm diameter				
TOTAL					5.753.000

Table 6.24. Payment details for total testing program

Item Ref.	ITEM DESCRIPTION	QUANTITY	Unit	Rate (\$)	Amount \$
CUTTING OFF Pileheads					
A	900 mm diameter	17	No.	381,5	6.485
B	1200 mm diameter	219	No.	545	119.350
C	1500 mm diameter	37	No.	845	31.250
TESTING					
Load Test on Non-working Pile					
D	Testing as required of 900 mm pile (OSTERBERG)	1	No.	83.380	83.380
E	Testing as required of 1200 mm pile	1	No.	132.150	132.150
F	(OSTERBERG)	1	No.	166.210	166.210
	Testing as required of 1500 mm pile				
G	(OSTERBERG)	1	No.	40.870	40.870
H	Static Load Test	2	No.	59.945	119.890
J	Testing as required of 900 mm pile	1	No.	107.900	107.900
	Testing as required of 1200 mm pile				
K	Testing as required of 1500 mm pile	1	No.	1.830	1.825
L	Dynamic Testing	11	No.	2.020	22.180
M	Testing as required of 900 mm pile	2	No.	3.050	6.100
	Testing as required of 1200 mm pile				
N	Testing as required of 1500 mm pile	2	No.	1.580	3.160
O	Sonic Cross-hole Testing	22	No.	1.745	38.365
P	Testing as required of 900 mm pile	4	No.	2.260	9.050
	Testing as required of 1200 mm pile				
R	Testing as required of 1500 mm pile	17	No.	15,00	255
S	Low Strain Integrity Testing	219	No.	15,00	3.280
T	Testing as required of 900 mm pile	37	No.	15,00	555
	Testing as required of 1200 mm pile				
	Testing as required of 1500 mm pile				892.266
TOTAL					

As a result, 15 per cent of all pile construction cost corresponds to pile testing cost including pile head preparation.

6.3.2. Soil Conditions and Design Parameters

Existing ground level ranges between 1,1 DMD and 2,4 DMD at borehole locations and it was assumed at +2.3 DMD for the subject project. According to the geotechnical report, top surface layer consists of dense, light grey, silty, fine to medium grained sand with some shell fragments. A very weak to weak, grayish brown to light brown, fine to medium grained shely sandstone was encountered between -3.70 to -9.20 DMD. Fractures are closely to medium spaced fractures in this zone and occasional vertical fractures were encountered. Below this zone a weak, massive to thinly laminated, grayish brown to light brown, fine to medium grained sandstone / gypsiferous sandstone occasionally interbedded with very dense cemented silty sand layers, was found down to -33.0 DMD. All boreholes were ended in a very weak to weak grayish to light brown, fine grained siltstone / gravelly siltstone with very closely to closely fractured unit. At the time of investigation, the ground water level was established at a depth of +0.70 to +1.50 DMD from the ground surface level. However this level may be subject to changes if strong dewatering plants in nearby areas stop operating. Ground water level was assumed to be at +1.00 DMD in the design. UCS design values for each zone are given in Figure 6.27 with idealized soil profile. The variation of UCS values with depth are plotted in Figure 6.28 and soil profile below the foundation level was divided into three zones based on the UCS values from the figure.

<i>Layer No</i>		<i>DMD</i>	<i>Description</i>
		2.30	Natural Ground Level
1	<i>GWT</i>	1.00	SAND
		-8.00	
2		-20.00	SANDSTONE UCS = 1.88 Mpa
3		-35.00	SANDSTONE UCS = 1.48 Mpa
4		-58.00	SILTSTONE UCS = 0.87 Mpa

Figure 6.28. Idealized soil profile and UCS design values for pile design

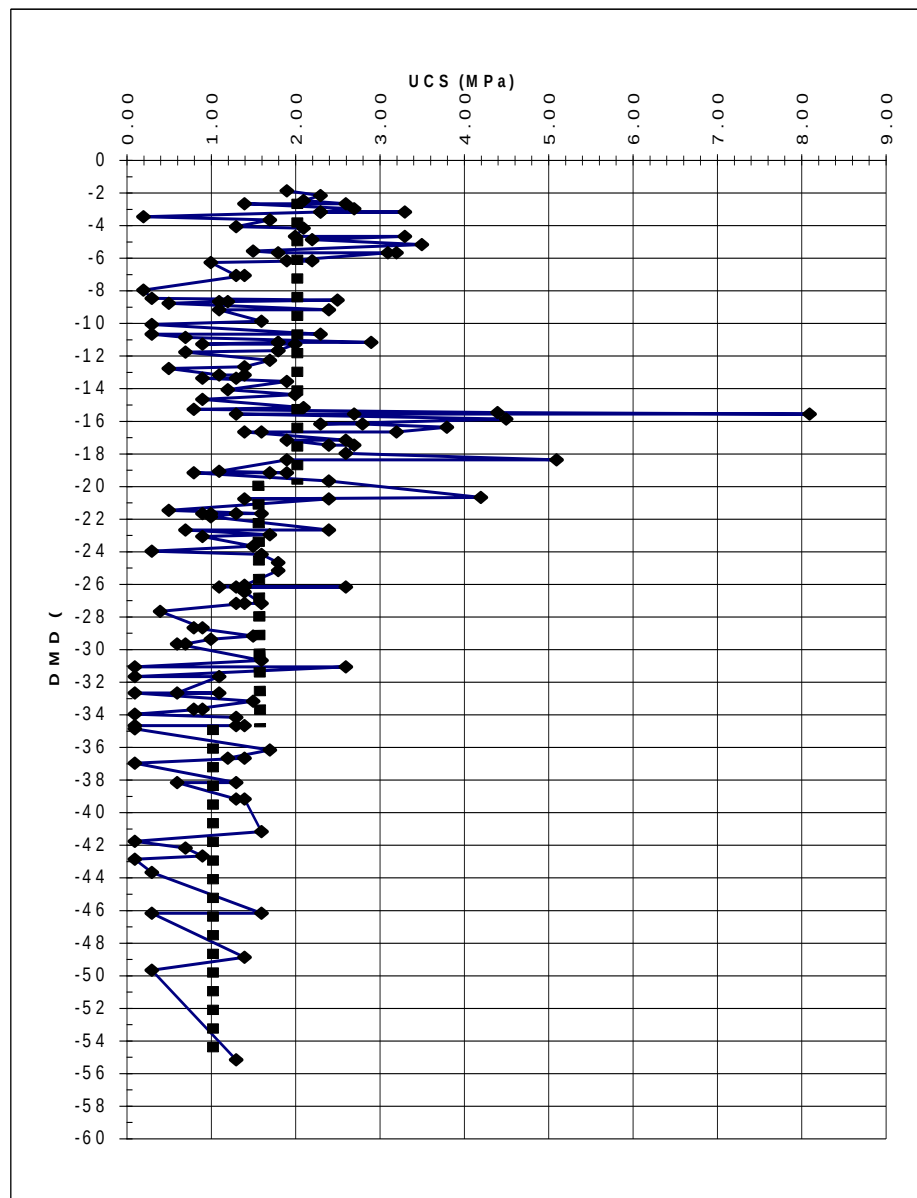


Figure 6.27. Variation of UCS values with depth

6.3.3. Load Capacity and Settlement Calculations by Statical Analysis

Since soil conditions are similar with Case Study-1, same theoretical correlations were used in the assessment of pile capacity and settlements. Again, pile capacities relied only on bearing stresses of the sandstone and siltstone strata, and the skin friction through the upper sand layer was neglected. Another point is that pile group effect was not taken into account. Table 6.25 presents the design unit skin friction values and Table 6.26 the skin friction values evaluated using same precedures as Case Study-1. Table 6.25.

Determination of unit skin friction of rock layers

$$f_s = \alpha q_u^\beta \text{ (Mpa)}$$

Design Method	α	β	Layer 4 Sandstone $q_u = 1.88$ (MPa)	Layer 5 Sandstone $q_u = 1.48$ (MPa)	Layer 6 Sandstone $q_u = 0.87$ (Mpa)
Horvath and Kenney (1979)	0.21	0.50	0.29	0.26	0.20
Carter and Kulhawy (1988)	0.20	0.50	0.27	0.24	0.19
Williams et al. (1980)	0.44	0.36	0.55	0.51	0.42
Rowe and Armitage (1984)	0.40	0.57	0.57	0.50	0.37
Rosenberg and Journeaux (1976)	0.34	0.51	0.47	0.42	0.32
Reynolds and Kaderbek (1980)	0.30	1.0	0.56	0.44	0.26
Gupton and Logan (1984)	0.20	1.0	0.38	0.30	0.17
Reese and O'Neill (1987)	0.15	1.0	0.28	0.22	0.13
Toh et al. (1989)	0.25	1.0	0.47	0.37	0.22
Migh and Wolshi (1979)	0.22	0.50	0.32	0.28	0.20
Horvath (1982)	0.20	0.50	0.27	0.24	0.19
		$f_s(\text{max})$	0.57	0.71	0.42
		$f_s(\text{min})$	0.27	0.22	0.13
		$f_s(\text{avg})$	0.40	0.34	0.24
		$f_s(\text{design})$	0.38	0.32	0.22

Table 6.26. Calculation of skin friction capacity

Layer (from pile cut-off)	Unit skin friction capacity (f_{design}) (kPa)	Pile Diameter (D) (m)	Layer Length (L) (m)	Friction capacity of the layer ($Q_{s, \text{layer}}$) (kN)
2	380	1.2	16.1	23053
3	320	1.2	15.0	18086
4	220	1.2	8.9	7378

Bearing capacity of skin friction (Q_{skin}) = total $Q_{s, \text{layer}}$ = 48517 kN

Next, toe capacity was calculated using same formulas presented in Case Study-1. As a result it was evaluated as,

$$Q_{toe} = 29435 \text{ kN} \quad (6.36)$$

Using a safety factor of 3.5 as stated in Table 5.4, the ultimate capacity was calculated:

$$Q_{ult} = \frac{(Q_s + Q_{toe})}{3.5} = 22272 \text{ kN.} \quad (6.37)$$

As taking the density of steel as 80 kN/m^3 and concrete as 25 kN/m^3 ; for 1200 mm. diameter and 40 m. length, pile weight was calculated as:

$$Q_o = Q_{concrete} + Q_{steel} = 1130.40 + 41.64 = 1172 \text{ kN} \quad (6.38)$$

And the total allowable bearing capacity of the pile is:

$$Q_{allow} = 22272 - 1172 = 21100 \text{ kN} \quad (6.39)$$

By using the calculated capacity in settlement formulas as presented in Case Study-1; the displacement values under working load were determined and shown in Table 6.27. Allowable pile load (Q_{allow}) was greater than pile design working load value of 18991 kN, however, displacement value was higher than performance criteria. Therefore, it was concluded that this pile was not allowable.

Table 6.27. Settlement calculations for P2-1 pile type

P2-1				
E = 29,000,000 kN/m²		Diameter 1200 mm. ; Pile Length = -42.2 m		
	Q_{design}	1.5 x Q_{design}	2.0 x Q_{design}	
Load =	21,100 kN	31,650 kN	42,200 kN	
Side settlement (mm) =	5.2	7.83	10.44	
Tip settlement (mm) =	0.0	0.0	0.1	
Elastic settlement (mm) =	12.9	19.3	25.7	
Total (under full load) (mm) =	18.1	27.1	36.2	
Total (residual) (mm) =	5.2	7.8	10.4	

6.3.4. Osterberg Load Test

A preliminary osterberg load test had been carried out on a non-working bored pile coded as PTP3 at the Dubai Mixed Use Development site, Dubai, U.A.E. The main objective of this load test was to determine if the pile design was allowable and if a more economical design was available or not, by proof – loading the test pile to its maximum test load of 4,748 tonnes which was two and a half times the working load of 1,899 tonnes and comparing the results with the statical analysis of the working pile coded P2-1 which had same design parameters of 1200 mm diameter and approximately 40.00 m length below the cut-off level. A total of three osterberg load tests were conducted on the site for each 1000, 1200 and 1500 mm. diameter piles, however in this study only one of them, the 1200 mm diameter pile, was investigated.

6.3.4.1. Test Instrumentation. One hydraulic jack assembly comprising three 600-tonne capacity bidirectional hydraulic jacks was installed at -29.15 m. DMD (Level J1) and another hydraulic jack assembly also comprising three 600-tonne capacity bidirectional hydraulic jack was installed at -42.90 m. DMD (Level J2). The installation of one of the load cells is shown in Figure 6.29. There were a pair each of telltale extensometers installed at the top and bottom of each hydraulic jack assembly and their movements were measured against a reference frame constructed by the contractor. The pile top movements were also measured against this reference frame. Eleven levels of vibrating wire-type strain gauges (VWSG) comprising four units at each level were installed in the test pile to measure strains at nominated locations. The strain gauges were mounted at designated Level S1 to Level S11, corresponding to -4.40 m, -7.40 m, -10.40 m, -13.40 m, -16.40 m, -19.40 m, -22.40 m, -25.00 m, -32.00 m, -35.00 m. and -38.00 m. DMD respectively. The location and placement of the instrumentation are shown in Figure 6.30. During construction, for the easy access of tremie pipes down from the hydraulic load cells, cutouts of sufficient sizes were provided in the cell steel bearing plates and a funnel was constructed between the opening in the top plate of the cell assembly and the main vertical rebar to guide the tremie pipe through steel bearing plates and to avoid the tremie pipe accidentally hit and damage the load cell or steel cage as shown in Figure 6.31.



Figure 6.29. Installation of the load cell into steel cage



Figure 6.30. Positioning of the hydraulic load cell system



Figure 6.31. Positioning of the cutout and funnel on the load cell with extensometers and hoses

Bored pile excavation was carried out under the bored pile contractor's work plan as approved. The hydraulic jack assemblies and steel cages were jointed, lowered and the pile was concreted according to the specifications described in Chapter 5.3.3 (Figure 6.32). Pressure was applied to the hydraulic jack by using an air-driven hydraulic pump. A high-pressure bourdon gauge as well as a calibrated pressure transducer was used to indicate the pressure. The displacement transducers were supported from the reference frame that was mounted on the ground. It is to be noted that the loads applied by the bidirectional hydraulic jacks acted in two opposite directions, resisted by upper side shear above the jack and by the combined end bearing and lower side shear below the jack. Effectively, the pile is divided into three pile segments; Segment 1 above Level J1, Segment 2 below Level J1 and above Level J2 and Segment 3 which was below Level J2 as illustrated in Figure 6.33.



Figure 6.32. Installation of steel cage carrying the load cell

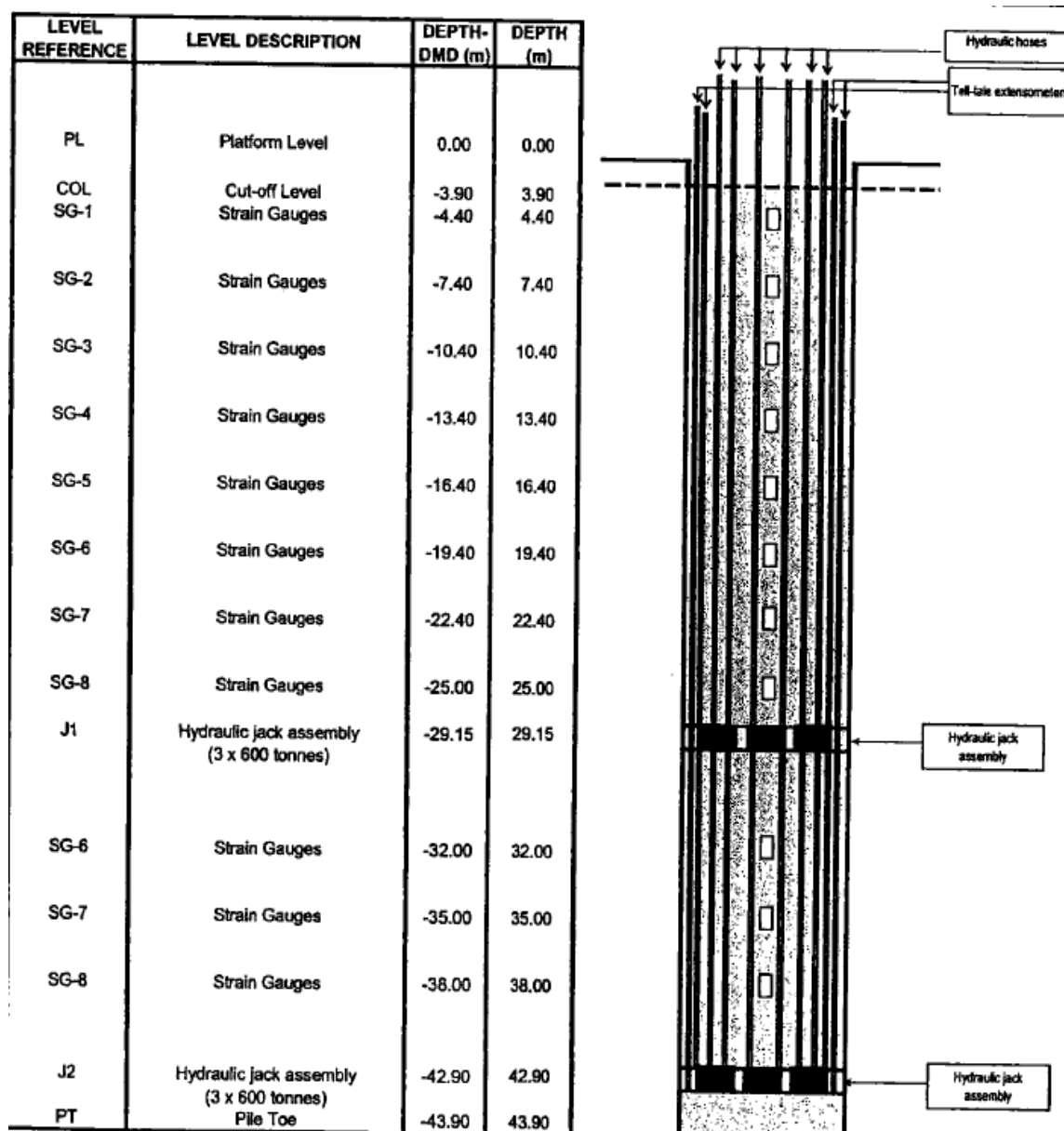


Figure 6.33. Pile instrumentation details for the Osterberg cell test

The load test was conducted in two loading cycles. The first loading cycle, using only the bottom hydraulic jack assembly at Level J2, loaded the base of the pile (Segment-three) using the side shear above the assembly (Segment-two) as reaction. The second cycle, using only the top jack at Level J1 (with the bottom jack pressure off and allowed to drain), loaded the side shear for the pile segments immediate above and below the top jack, first and second segment, using each other as reaction. During this loading, no load was

transferred to end bearing. The two loading cycles are shown in the following Tables 6.28 and 6.29 respectively.

Table 6.28 First loading cycle

Loading Cycle 1	Load Increment / Decrement (% SWL)	Effective Bi-directional Load (tonnes)	Minimum Holding Period
L1-0	0	0	0
L1-1	25	230	10 min
L1-2	50	460	10 min
L1-3	75	689	10 min
L1-4	100	919	1 hour
L1-5	125	1,149	10 min
L1-6	150	1,379	10 min
L1-7	175	1,608	10 min
L1-8	200	1,838	10 min
L1-9	225	2,068	10 min
L1-10	250	2,298	2 hours
U1-1	225	2,068	10 min
U1-2	200	1,838	10 min
U1-3	175	1,608	10 min
U1-4	150	1,379	10 min
U1-5	125	1,149	10 min
U1-6	100	919	10 min
U1-7	75	689	10 min
U1-8	50	460	10 min
U1-9	25	230	10 min
U1-10	0	0	10 min

It is to be noted that the loading was terminated at 125 per cent of working load due to high displacement.

Table 6.29 Second loading cycle

Loading Cycle 1	Load Increment / Decrement (% SWL)	Effective Bi-directional Load (tonnes)	Minimum Holding Period
L2-0	0	0	0
L2-1	25	360	10 min
L2-2	50	720	10 min
L2-3	75	1,080	10 min
L2-4	100	1,440	1 hour
L2-5	125	1,800	10 min
L2-6	150	2,160	10 min
L2-7	175	2,520	10 min
L2-8	200	2,880	10 min
L2-9	225	3,240	10 min
L2-10	250	3,600	2 hours
U2-1	225	3,240	10 min
U2-2	200	2,880	10 min
U2-3	175	2,520	10 min
U2-4	150	2,160	10 min
U2-5	125	1,800	10 min
U2-6	100	1,440	10 min
U2-7	75	1,080	10 min
U2-8	50	720	10 min
U2-9	25	360	10 min
U2-10	0	0	10 min

A millimeter scale was fixed to the reference frame, and direct readings from a dumpy level to this scale to check that there were no gross errors in the displacement transducer readings. The displacement and load data were automatically recorded at one minute intervals. The pile head displacement, top of hydraulic jack assembly displacement for J1 and J2 and bottom of hydraulic jack assembly for J1 and J2 displacement were measured using displacement transducers, which were connected to telltale extensometers. A total of six displacement transducers were used to measure all movements at the selected locations. All the displacement transducers performed well during the duration of the test. All VWSGs installed were found to be in good working order throughout the duration of test.

6.3.4.2. Interpretation of Test Data and Discussion of Results. The load-displacement data was read from the display screen of computerized system during the test by an experienced engineer. Figure 6.34 shows the load movement curves of loading for the first cycle and Figure 6.35 for the second cycle determined by the displacement readings presented in Appendix C. During the first cycle, the maximum pile top movement was 0,4 mm. and maximum upward movement at the jack assembly was 1,3 mm. The maximum downward movement at the jack assembly was 79,0 mm. at the applied bi-directional load of 1,149 tonnes. During the second cycle, the maximum pile top movement was 45,6 mm. and maximum upward movement at the jack assembly was 50,8 mm. The maximum downward movement at the jack assembly was 4,8 mm. at the applied bi-directional load of 3,600 tonnes.

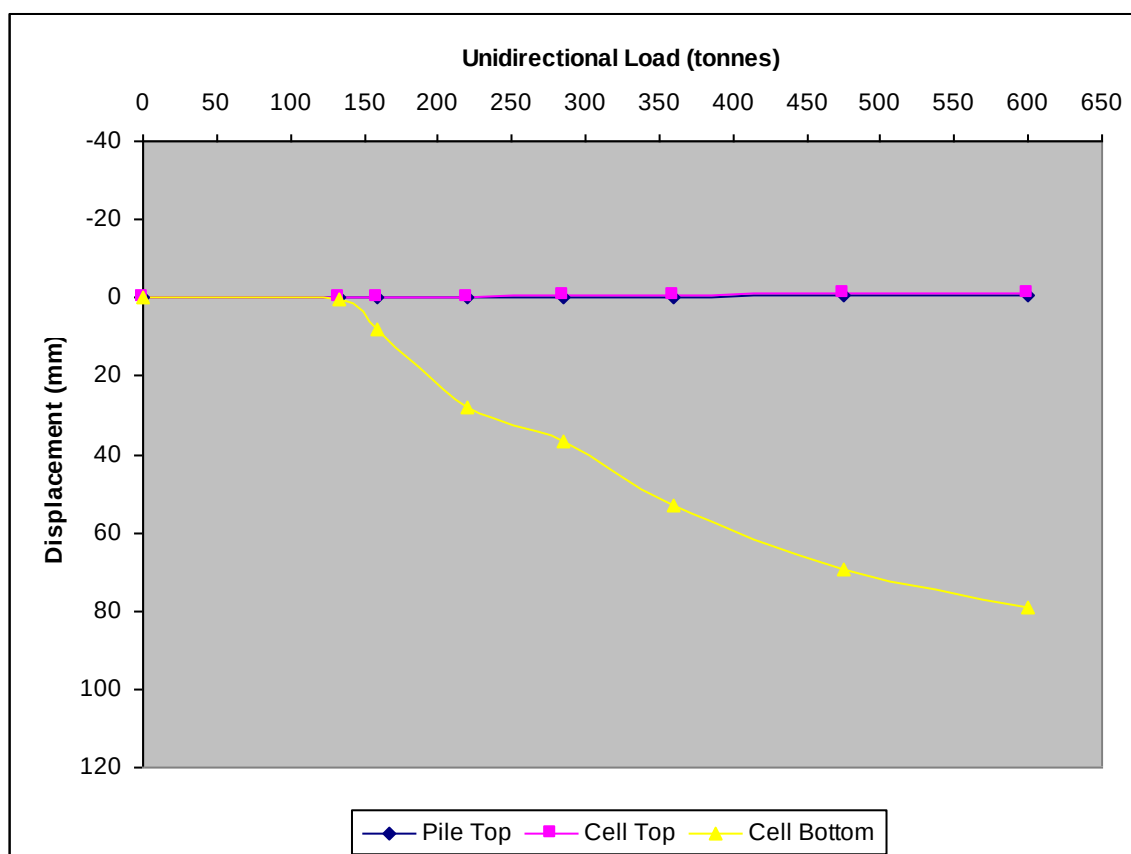


Figure 6.34. Load movement plot for bottom jack (Cycle 1)

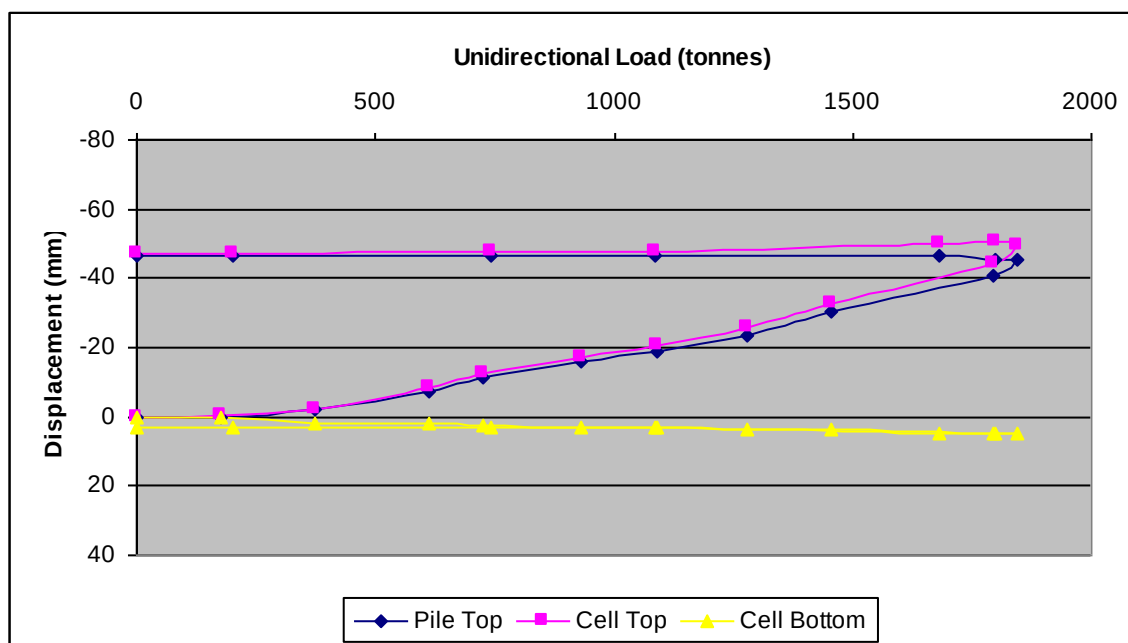


Figure 6.35. Load movement plot for top jack (Cycle 2)

Moreover, strain was read at respective strain gauge levels during the test. These readings from second cycle are presented below at Table 6.30.

Table 6.30. Microstrain ($\varepsilon_z \times 10^6$) at different levels

Level (DMD)	(7200 kN) 50%	(14400 kN) 100%	(21600 kN) 150%	(28800 kN) 200%	(36000 kN) 250%
-4.40	0.5	0.1	0.5	0.9	0.6
-7.40	-3.1	-4.5	-4.4	-4.6	-5.6
-10.40	-5.8	-9.3	-8.4	-11.0	-17.9
-13.40	-8.3	-14.1	-14.4	-23.3	-40.2
-16.40	-12.9	-21.0	-21.8	-40.1	-75.3
-19.40	-18.6	-34.4	-36.4	-73.2	-115.9
-22.40	-22.9	-39.0	-43.9	-98.0	-169.9
-25.00	-27.3	-63.6	-77.0	-144.9	-217.3
-32.00	-46.7	-115.8	-167.0	-233.9	-298.3
-35.00	-14.2	-40.2	-45.2	-84.7	-133.4
-38.00	-6.0	-7.1	-8.1	-23.8	-43.8

These measured changes in strain gauge readings and estimated pile properties (cross-sectional area and modulus of elasticity) were used to evaluate the load transfer distribution of the pile for both loading cycles respectively. Load transfer (P) at each strain gauge level was calculated as follows:

$$P = \Delta_{wsG} AE_p \quad (6.40)$$

where,

Δ_{wsG} = Change in strain gauge readings.

AE_p = Equivalent axial pile stiffness.

In the analysis, the elastic modulus of 37 kN/mm² was used based on the cube test results released. The calculated loads at different levels for the second cycle are presented at Table 6.31 and based on this table, the load distribution curve was plotted in Figure 6.36.

Table 6.31. Calculated loads at different levels (kN)

Level (DMD)	(7200 kN)	(14400 kN)	(21600 kN)	(28800 kN)	(36000 kN)
	50%	100%	150%	200%	250%
-4.40	22	3	24	40	27
-7.40	-141	-201	-198	-209	-253
-10.40	-261	-416	-379	-496	-804
-13.40	-373	-634	-649	-1049	-1807
-16.40	-578	-944	-983	-1804	-3387
-19.40	-839	-1550	-1637	-3294	-5218
-22.40	-1032	-1757	-1976	-4411	-7647
-25.00	-1228	-2863	-3467	-6519	-9781
-32.00	-2102	-5212	-7515	-10525	-13423
-35.00	-638	-1810	-2036	-3811	-6003
-38.00	-272	-322	-367	-1070	-1973

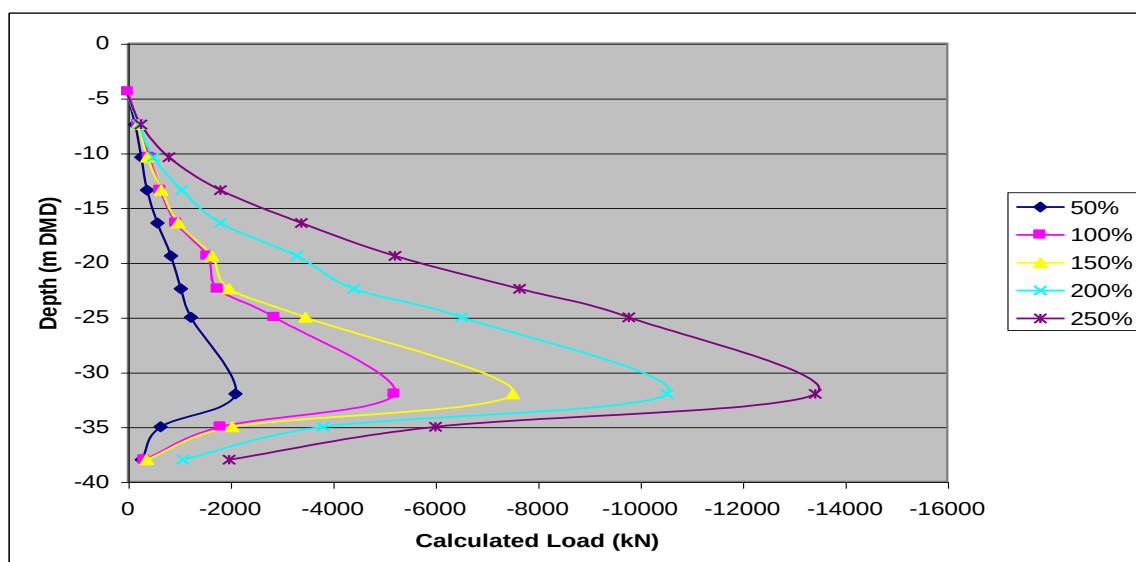


Figure 6.36. Calculated load distribution along the pile shaft

The difference between the loads at any two levels represents the shaft load carried by the portion of pile between the two strain gauge levels. Load transferred between levels in the second cycle are presented in Table 6.32.

Table 6.32. Load transferred between levels (kN)

	7200 kN	14400 kN	21600 kN	28800 kN	36000 kN
Level	50%	100%	150%	200%	250%
$E_c = 37 \text{ kN/mm}^2$					
1 to 2	162	203	222	249	280
2 to 3	120	216	182	287	551
3 to 4	112	217	270	554	1004
4 to 5	206	310	334	755	1580
5 to 6	261	606	654	1490	1830
6 to 7	192	207	339	1117	2429
7 to 8	196	1106	1490	2108	2134
8 & Jack1	2372	4337	7333	7881	8219
Sum above Jack	3622	7202	10824	14440	18027
9 to Jack1	1498	1988	3285	3875	4578
9 to 10	1463	3402	5479	6715	7419
10 to 11	366	1489	1669	2740	4031
Sum below Jack	3328	6878	10433	13330	16027
Measured Load during Cycle 2	6950	14081	21258	27770	34055

Average unit side shear between any two strain gauge levels of the pile in the second cycle was calculated by the change in load divided by the circumferential area between the two strain gauge levels. The maximum mobilized unit side shear between the adjacent strain gauge levels were computed and are shown in the following Table 6.33. Mobilized unit shaft friction curves are presented in Figure 6.37.

Table 6.33. Unit skin friction values (kN/m²)

	7200 kN	14400 kN	21600 kN	28800 kN	36000 kN
Level	50%	100%	150%	200%	250%
$E_c = 37 \text{ kN/mm}^2$					
1 to 2	14.3	18.0	19.6	22.0	24.8
2 to 3	10.6	19.1	16.0	25.3	48.7
3 to 4	9.9	19.2	23.9	49.0	88.8
4 to 5	18.2	27.4	29.5	66.8	139.7
5 to 6	23.1	53.6	57.8	131.7	161.8
6 to 7	17.0	18.3	30.0	98.8	214.8
7 to 8	20.0	112.9	152.0	215.1	217.7
8 & Jack	135.3	247.4	418.3	449.6	468.9
9 & Jack	169.1	224.4	370.8	437.4	516.7
9 & 10	32.4	131.6	147.6	242.3	356.4
10 & 11	24.0	28.4	32.4	94.6	174.4

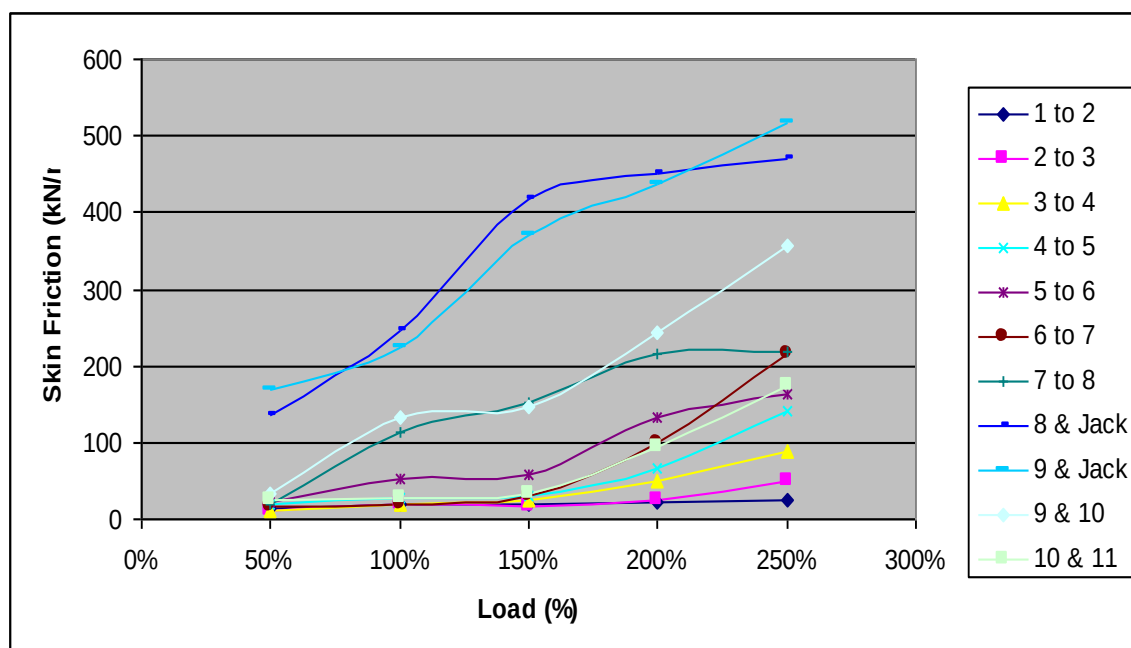


Figure 6.37. Unit skin friction along the pile shaft

The unit skin friction value ranges from 24,80 kN/mm² to 516,70 kN/mm² with maximum between level nine and bottom of jack (J1). The calculated skin friction values increases with applied load indicates that the ultimate resistance had not achieved during the ultimate load. The variation in unit skin friction indicates the change in lithology. The high displacement measured at bottom jack level indicates low end bearing capacity of the pile as wells as the presence of soft material such as siltstone at the pile toe.

To evaluate the load capacity of the pile, the equivalent top loaded load-settlement movement curves were derived from osterberg load test results based on the calculations presented in Section 5.3.5.1. To start with, end bearing load movement curve was derived and presented in Figure 6.38 using the measured displacement data of the bottom of the cell in the first loading cycle, because the pressure loaded to the bottom osterberg cell was resisted by side shear at the cell top and this reaction provided the determination of the base resistance at the cell bottom, so displacement at the bottom of the cell was proportional to the value of toe resistance. Since the cell expands on both directions, half of the applied pressure was taken as the effective load as the downward pressure determined the base resistance.

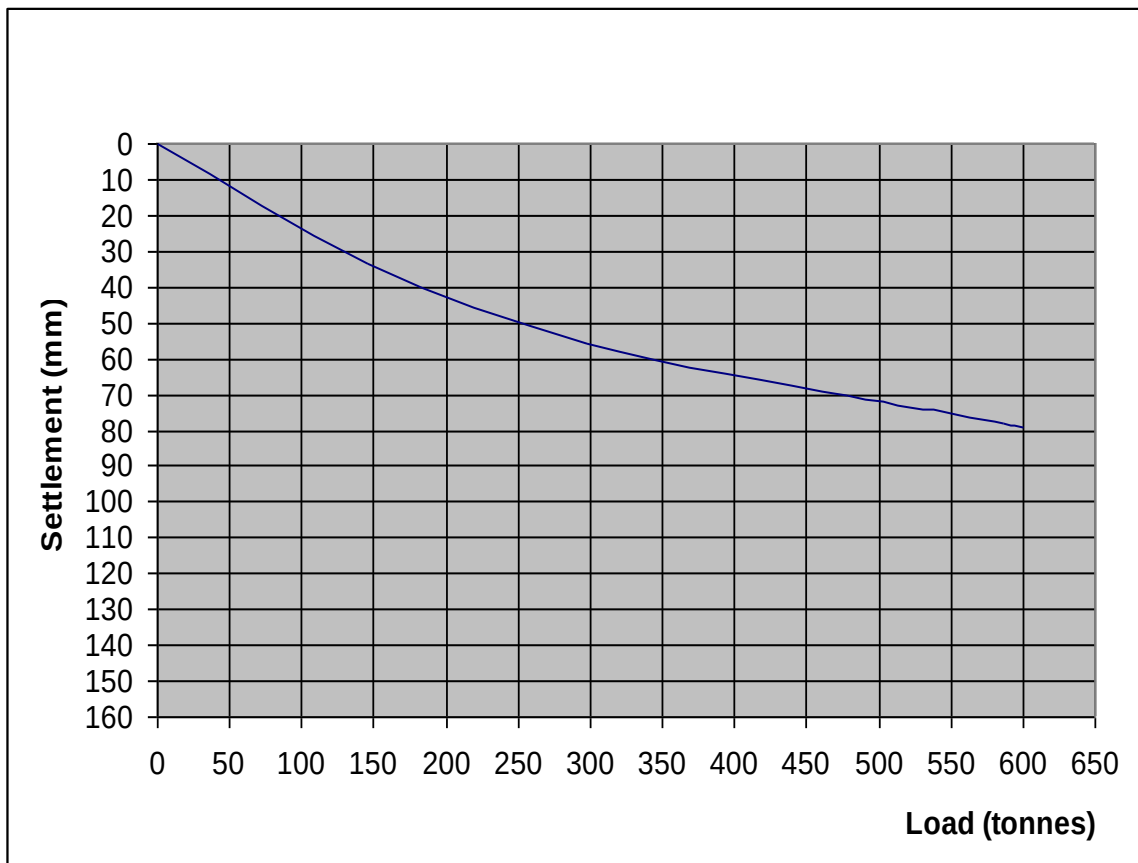


Figure 6.38. End bearing load movement curve during osterberg load cell

Next, both the upper and the lower side-shear load-movement curves were derived using the displacement data of both the lower load cell in the first cycle of loading and the upper load cell in the second cycle of loading as shown in Figure 6.39. When the upper load cell was pressured with the lower cell stable, soil layers at both sides of the upper cell reacted against each other, providing displacements at both sides which were read by the gauges. The displacement values at the bottom of the upper cell were used to plot the lower side shear load movement curve. The upper side shear movement curve was plotted using the displacement data of the cell top of the lower load cell from the first loading cycle, by using the assumption presented in Section 5.3.5.1 that pile behaves as a rigid body and top of pile moves downward the same as the bottom.

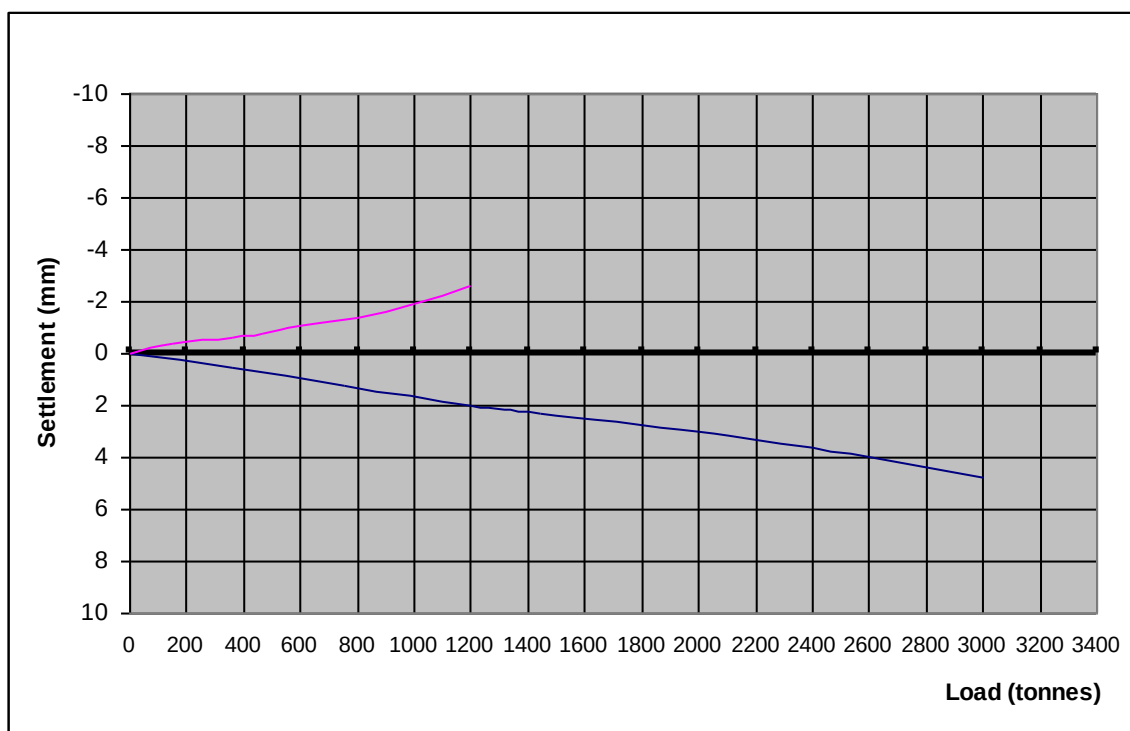


Figure 6.39. Upper and lower side shear load movement curves

Then, to construct the equivalent top loaded settlement curve, the side shear and end bearing curves should be combined in a single load-movement curve. To accomplish this, using the assumption again presented in Section 5.3.5.1 that the top of pile moves downward the same as the bottom, the corresponding load value for a selected displacement value in the end bearing load movement curve was added to the load value of the same displacement value in the side shear movement curve. However, in the first loading cycle displacement values were so high under small loads for end bearing so the loading was terminated at 125 per cent of working load although skin friction settlement values were very little. Therefore, for skin friction settlement values to reach to same values of the end bearing, the upper side shear load movement curve was extrapolated after 1149 tonnes of loading by the suitable hyperbolic curve fitting technique. The maximum displacement was selected to be 30 mm. for the equivalent top loaded settlement curve although the maximum read displacement in end bearing was 79 mm, since above that level the high values could not be allowable. Figure 6.40 shows the extrapolated upper side shear load movement curve. For example, let's say an arbitrary displacement such as 10 mm. in end bearing curve corresponds to 50 tonnes of loading. Next, same 10 mm. displacement corresponds to 3150 tonnes of loading in the extrapolated side shear curve.

So, for a 10 mm. displacement, the sum of the loading values for both end and side shear was calculated to find 3200 tonnes, which was in the end plotted on the equivalent top loaded settlement curve. Same calculations were done for all points and as a result the curve was determined as shown in Figure 6.41.

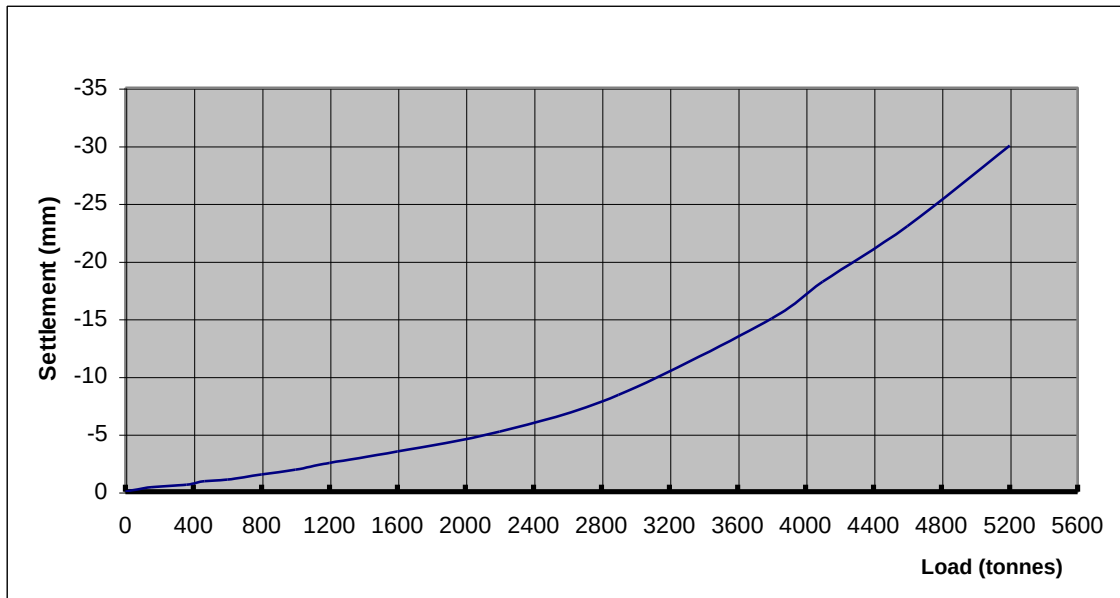


Figure 6.40. Extrapolated side shear load-movement curve

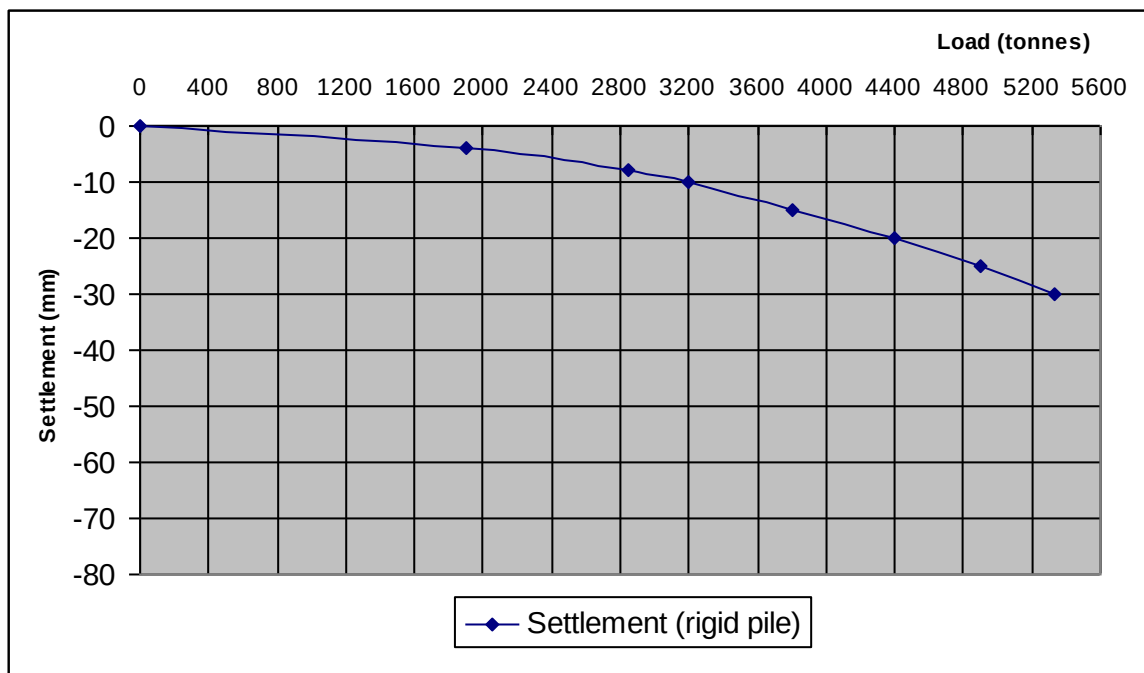


Figure 6.41. Equivalent top load-settlement chart by combination side shear and end bearing movement curve

The adjustment for additional elastic compression was calculated using the correlations presented in Section 5.3.5.1. Since it was a multi level test, following formulas were used:

$$\delta_{BDSL} = \delta_1 + \delta_2 \quad (6.41)$$

$$\delta_1 = C_3 \cdot \frac{Q_1 \cdot l_1}{AE} \quad (6.42)$$

$$\delta_2 = C_2 \cdot \frac{Q_2 \cdot l_2}{AE} \quad (6.43)$$

$$Q_{LT} = Q_0 + \delta_{LT} \quad (6.44)$$

$$\delta_0 = \frac{P \cdot l_0}{AE} \quad (6.45)$$

$$\delta_{LT} = \delta_0 + \delta_1 + \delta_2 \quad (6.46)$$

where,

$$C_1 = 0.441$$

$$C_2 = 0.579$$

$$C_3 = 0.396$$

$$l_0 = 1.00 \text{ m}$$

$$l_1 = 29.15 \text{ m (embedded length of shaft above mid-jack)}$$

$$l_2 = 13.75 \text{ m (embedded length of shaft between hydraulic jack)}$$

$$l_3 = 1.00 \text{ m (neglected and taken as 0.00 m)}$$

$$A = 150.72 \text{ m}^2 \text{ (cross-sectional area of the pile)}$$

$$E = 3700000 \text{ ton/m}^2 \text{ (elastic modulus)}$$

Subtracting the ($\delta_{BDSL T}$) from the ($\delta_{TL T}$) compression gives the desired additional elastic compression at the top of the equivalent top loaded test. The additional elastic compression is then added to the rigid equivalent curve as shown in Figure 6.42 to obtain the final, corrected equivalent load-settlement curve for the top loaded test on the same pile. The calculations of the elastic compression are shown in Table 6.34. Based on Figure 6.42, the results show that the test pile would settle by 8.7 mm. at 1,899 tonnes and 37,4 mm. at 4,748 tonnes respectively.

Table 6.34. Calculation of elastic compression

$\Delta_{BDSL T}$ (mm)	Q_A' (tonnes)	$Q_{B \text{ bottom}}$ (tonnes)	$Q_{B \text{ top}}$ (tonnes)	P (tonnes)	$\delta_{TL T}$ (mm)	$\delta_{BDSL T}$ (mm)	Δ_{δ} (mm)	$\Delta_{BDSL T} + \Delta_{\delta}$ (mm)
0	0	0	0	0	0	0	0	0
4	20	20	1860	1900	8.6	3.9	4.7	8.7
8	30	40	2780	2850	12.9	5.8	7.1	15.1
10	40	60	3100	3200	14.5	6.5	8.0	18.0
15	65	65	3670	3800	17.5	7.7	9.8	24.8
20	80	70	4250	4400	20.0	8.8	11.2	31.2
25	100	80	4720	4800	22.3	9.9	12.4	37.4
30	130	100	5100	5330	23.4	10.6	12.8	42.8

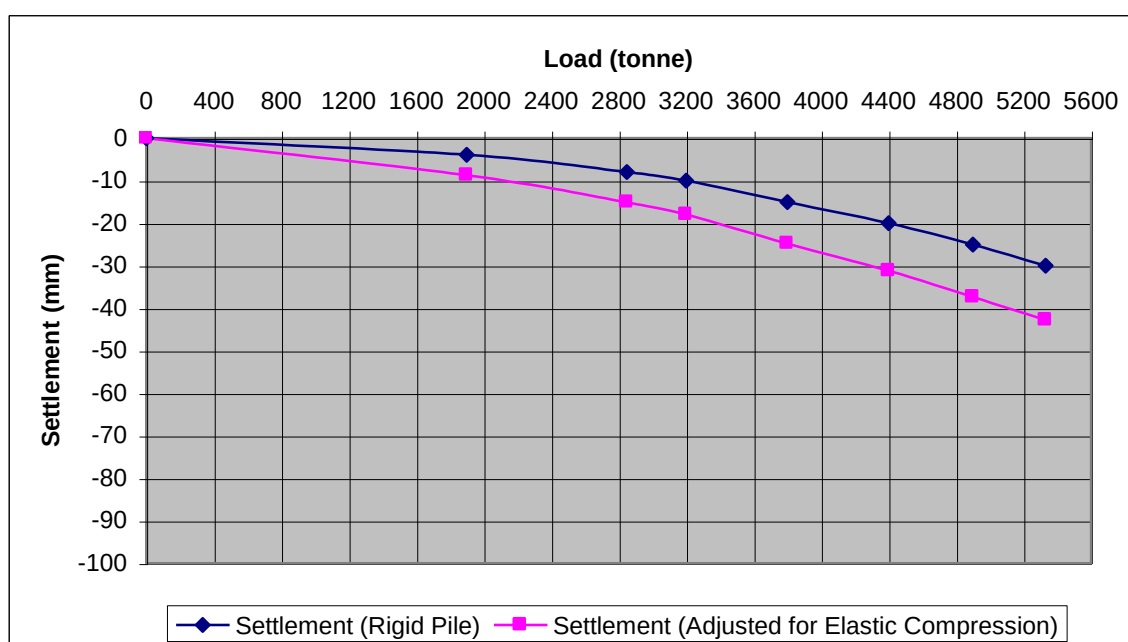


Figure 6.42. Equivalent top load vs. settlement chart

To sum up, the equivalent top-loaded load displacement curve as constructed showed that a top-loaded pile of 40 m. length from cut-off level would be expected to settle by 8,7 mm. at 1,899 tonnes and 37,4 mm. at 4,748 tonnes respectively. The computed unit skin friction would indicate the resistance range of 24,80 kN/m² to 516,70 kN/m².

6.3.5. Results

Osterberg cell test results showed that skin friction was higher and toe resistance was lower than the calculated values in statical analysis, meaning that almost all the load was carried by skin resistance in rock sockets. Results were more informative and detailed than a static load test, since toe and skin resistances were separately observed and the displacement due to the effect of elastic compression was reduced from displacement readings. Besides, results showed that a possible construction defect would be more easily observed by analysing the separate end bearing load-displacement curve, showing the pile bottom. The higher than expected value of skin friction would lead to a more economic pile design. In statical analysis, the skin friction was calculated to be 48517 kN, and by using a safety factor of 3.5 the allowable design load was evaluated to be 21100 kN with 18,1 mm. displacement value. The results of the osterberg cell test showed that under the same displacement value of 18,1 mm., the ultimate load capacity of the pile was approximately 32000 kN (Figure 6.42), and toe resistance was only 80 kN (Figure 6.38), almost all the load was carried by skin friction. As a result, if we had intended to decrease the pile length by five meters, and by assuming that the highest calculated unit skin friction of 517 kPa was effecting the pile at this level to be on the safe side, the skin friction at that level was calculated as:

$$Q_{s, \text{ level}} = 517 \times 1,2 \times \pi \times 5 = 9740 \text{ kN} \quad (6.47)$$

and new ultimate load capacity was evaluated, ignoring toe resistance as:

$$Q_u = 32000 - 9740 \cong 22000 \text{ kN} \quad (6.48)$$

which was allowable. If the cost of this osterberg test is compared to the profit by reducing all the 1200 mm diameter piles with five meters on the design in Table 6.35, it is seen that it would reduce the total estimated piling cost for about 105,680 dollars.

Table 6.35. Economical analysis for osterberg test

	Number of piles	Cost per meter (\$)	Cost per pile (\$)	Total profit (\$)
1200 mm dia. pile	219	217.2	5 x 217 = 1086	+ 237834
Osterberg cell test	1	-	132153	- 132153

6.4. Case Study 3: Dubai Municipality Sports Complex Site, Dubai, U.A.E.

The construction of the Dubai Municipality Aquatic Sports Complex is undertaken in Dubai Land. The works comprise the design and construction of Dubai Sports Complex. The complex comprises of three pools, a 50m competition pool, a 50m multi purpose pool, and a diving pool with spectator seating for 15,000 persons. The complex is designed with the intention of multiple uses and configurations, including international standard aquatic sports arena, international standard dry sports venue, concert venue, and exhibition space. The sports complex comprises of two main areas, the arena and foyer areas. The arena contains the 15,000 seat spectator bowl surrounding the diving and competition pools on three sides. Plant, stores, circulation, hospitality and other areas are provided below the seating in the stands. The foyer area includes the entrance, retail, kitchen and foyer spaces at the entrance level and the warm up/multipurpose pool below.

6.4.1. General

The project also includes infrastructure roads, car parking and a water feature. Figure 6.43 shows a design overview of the sports city.



Figure 6.43. Design overview of sports city including sports complex, Dubai

The total contract value for the piling works was 5,903,460 U.S. dollars and the contract period was 160 days. Total payment cost for piling was 2,895,700 dollars. Cost of total pile testing is given at Table 6.36 below:

Table 6.36. Cost of pile testing

Test type	Number	Unit cost (\$)	Total Cost (\$)
Preliminary static load testing	20	Varies	394,660
Static load testing on working piles	2 no. 900 mm ; 5000 kN / L = 11.0 m	35,695	71,400
	2 no. 1200 mm ; 10000 kN / L = 17.0 m	55,313	110,630
Dynamic Load Testing	17	2,180	37,060
Cross-hole sonic logging testing	35	1,308	45,780
Low strain testing with pulse-echo method	342	15,00	5,125

At site, one per cent of all piles were static load tested. Five per cent of the piles were dynamic load tested. 10 per cent of them were cross-hole sonic and 100 per cent of the piles were integrity tested with pulse-echo method. Total pile testing cost is 23 per cent of all piling works.

6.4.2. Subsurface Conditions and Preliminary Pile Design

The surface strata consist of loose / medium dense / very dense, fine to medium grained calcareous sand, pale brown to brown, fine to medium, silty sand extended to depths between 0,25 m to 2,5 m for all of the zones. Below the sandy deposits, there is pale brown to pale grayish brown, slightly to moderately weathered sandstone with cemented sand layer. However, the loose / medium dense, very dense and pale brown to brown medium silty sand layer extended down to 8.0 m below the working platform level for south of the site. As per the soil investigations report and site measurements, ground water level is recorded to be below the maximum drilling depth except piles number P-421-415. These two piles encountered 1.5-2.0 m. of water at the bottom of pile which corresponds to 31 m. from the platform level.

The unit skin friction was the primary parameter for the pile vertical capacity. Figure 6.44 below plots the available unconfined compressive strength values determined by laboratory tests on core samples for representation of the soil profile.

Piling design parameters were based on data from 10 boreholes having depths up to 60 m. from existing ground surface and fully instrumented preliminary pile static load tests. The non-working piles were tested up to 12000 kN and 20000 kN for diameters 900 mm. and 1200 mm. respectively. Four levels of strain gauges were installed in all preliminary test piles for precise determination of design parameters. Static load test schedule on non-working preliminary test piles is presented in Table 6.37.

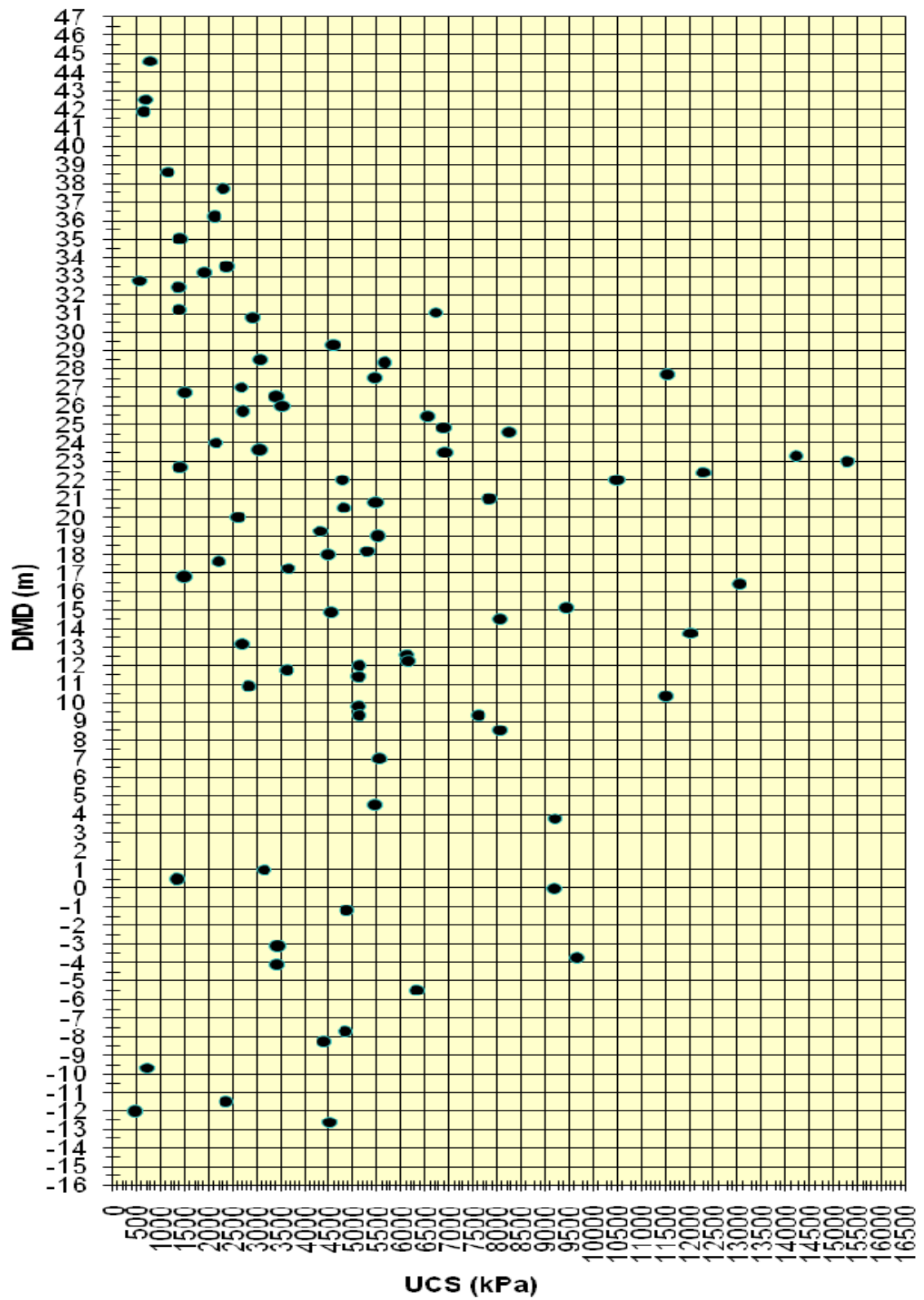


Figure 6.44. Unconfined compressive strength values versus depth

Table 6.37. Preliminary test piles and test details

Pile Type	Pile No.	Test Type	Test Load (KN)	Test Pile Toe Level (DMD)	Tension Load per Reaction Pile (KN)	Reaction Pile Diameter (mm)	Reaction Toe Level (DMD)	Pile Settlement %100 (mm)	Pile Settlement %200 (mm)
900A	TP 13	Compression	12000	31.00	3000	900	29.00	1,8	6,7
900B	TP 08	Compression	12000	32.50	3000	900	29.00	1,7	5,4
1200 A	TP 03	Compression	20000	29.00	5000	900	24.00	2,5	7,1
1200 B	TP 18	Compression	20000	31.00	5000	900	24.00	1,2	5,0

In conclusion, unit skin friction for sandstone had been adopted as 275 kPa and required pile capacities and lengths were calculated based on the interpretation of results from four preliminary static load tests. Generalized soil profile for piling design is stated in Table 6.38.

Table 6.38. Design parameters

Elevation (DMD)	Soil Type	SPT N Value	UCS (MPa)	γ_{sat} (kN/m ³)	F_s (kPa)
47.0 – 41.0	DENSE SAND	>50	-	17	45
41.0 – 31.0	SANDSTONE	-	1.5	18	275
31.0 – 25.0	SANDSTONE	-	3.0	18	275
25.0 – 15.0	CONGLOMERATE	-	4.0	20	420
15.0 – 10.0	LIMESTONE	-	2.0	21	515
10.0 – 0.00	CONGLOMERATE	-	6.0	20	300

The latest revision of working pile design and calculation report were based on data of fully instrumented preliminary pile static load tests and shown in Table 6.39. Piling design layout project is shown in Figure 6.45.

Table 6.39. Working piles schedule

Pile Type	Number of Piles	Diameter(mm)	Pile Length(m)	Working downward load per pile (kN)
P1	18	900	14	4533
P2	18	1200	16,5	7838
P3	24	1200	21,5	11185
P3A	4	1200	21,5	11185
P4	6	1200	15,5	9526
P5	32	900	18	6765
P6	14	1200	21,5	11080
P7	20	900	16,5	5863
P8	5	900	18,5	6539
P9	2	1200	17	8054
P10	2	1200	21,5	11359
P10GL20	2	1200	21,5	11359
P11	24	1200	14	8053
P12	8	1200	16	9873
P12A	6	1200	13	6625
P13	8	1200	14	8063
P14	12	1200	14	8186
P14GL21	8	1200	16,5	10275
P15	4	1200	17	10833
P16	1	900	13	5144
P17	1	1200	20	10347
P18	6	1200	18,5	9584
P19	4	1200	14	6774
P20	2	900	14	6115
C1	22	1200	21,5	11500
C2	16	1200	17,5	11300
C3	12	1200	13	6200
G1	16	1200	15	7220
G2	32	1200	17	8600
G3	4	1200	14,8	5900
G4	12	1200	13	5650

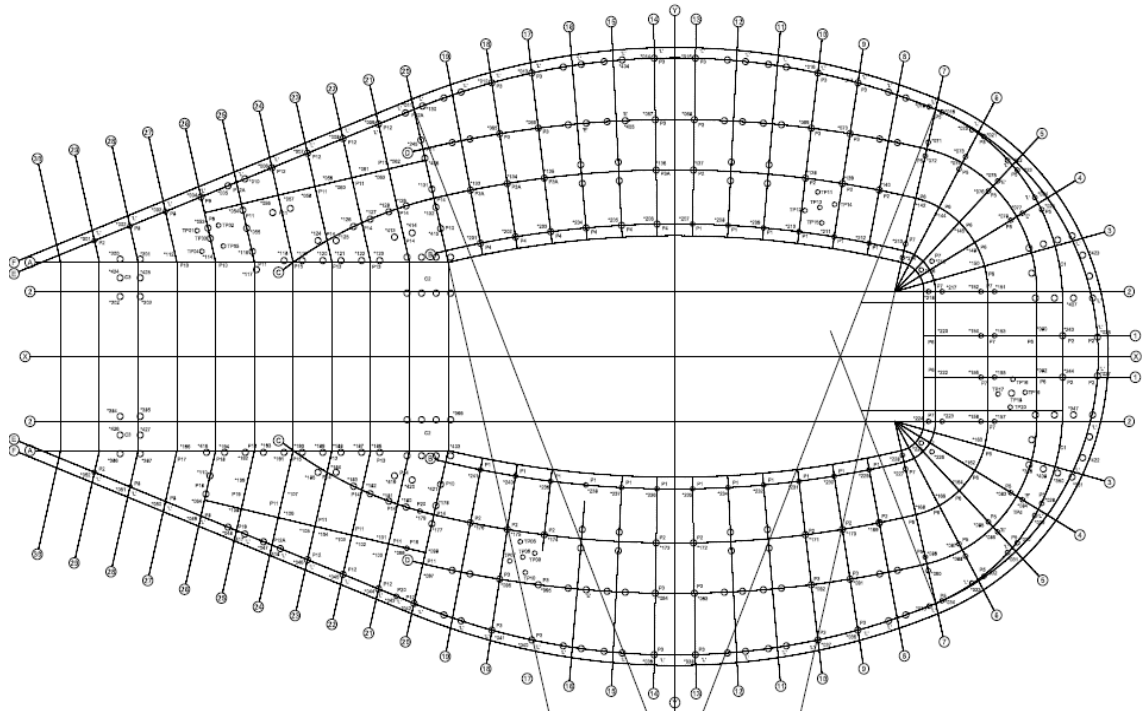


Figure 6.45. Piling layout

Capacity assessment detail for a C-1 type pile is shown below. Skin friction capacity was calculated using the same correlations presented in Case Study-1 and shown in Table 6.40.

Table 6.40. Calculation of skin friction capacity

Layer (from pile cut-off of +45.40 DMD)	Unit skin friction capacity (f_{design}) (kPa)	Pile Diameter (D) (m)	Layer Length (L) (m)	Friction capacity of the layer ($Q_{s, \text{layer}}$) (kN)
1 (Dense Sand)	45	1.2	4.4	746
2 (Sandstone)	275	1.2	10	10362
3 (Sandstone)	275	1.2	6	6217
4 (Conglomerate)	420	1.2	1.1	1741

Bearing capacity of skin friction (Q_{skin}) = total $Q_{s, \text{layer}}$ = 19066 kN

Tip resistance was calculated using the same correlations conducted in Case Study-1, and was determined as:

$$Q_{\text{tip}} = q_b = 2N_{\phi} q_u = 2 \times 2,66 \times 1.74 = 9265 \text{ (kN)} \quad (6.49)$$

Allowable pile capacity was calculated using above parameters and by taking the factor of safety as 2.0 since static load testing was conducted on preliminary piles:

$$Q_{all} = \frac{Q_{skin} + Q_{tip}}{2} = 14166 \text{ kN} \quad (6.50)$$

And by considering the pile's dead weight in the design, the final allowable pile capacity is:

$$Q_{all} = 14166 - 1965 = 12200 \text{ kN} \quad (6.51)$$

Based on the result, the design parameters of the C-1 coded pile was allowable since the allowable load capacity was larger than the required working load of 11500 kN. Below, both static and dynamic load testing was conducted on the working pile of C-1 pile type after the construction of the pile. The aim of this study was to compare the capacity and displacement assessments of both load tests with each other and with the design values of the pile to check if there could be a more economic design for this pile type if below study had been conducted as a preliminary work.

6.4.3. Static Load Testing

A static load test was performed on pile number of P-340 which was a C-1 type pile at DM Sport Complex site in Dubai, U.A.E., as per ASTM D 1143. The working pile was tested up to 150 per cent of its design load. Testing setup was shown in Figure 6.46 as a sample photo from site and as an illustration of test setup in Figure 6.47. Test details are described in Table 6.41 below:

Table 6.41. Test pile details

Test Pile	Diameter (mm)	Cut-off Level (DMD)	Type of Test	Working Load (kN)	Test Load (kN)	Tension Load per Reaction Pile (kN)	Reaction Pile Diameter (mm)	Cut-off Level (DMD)	Reaction Pile Toe Level (DMD)
P-340 (C1)	1200	+45.40	Compression	11500	17250	8625	1200	+45.40	+28.90



Figure 6.46. Static compression load test setup sample photo

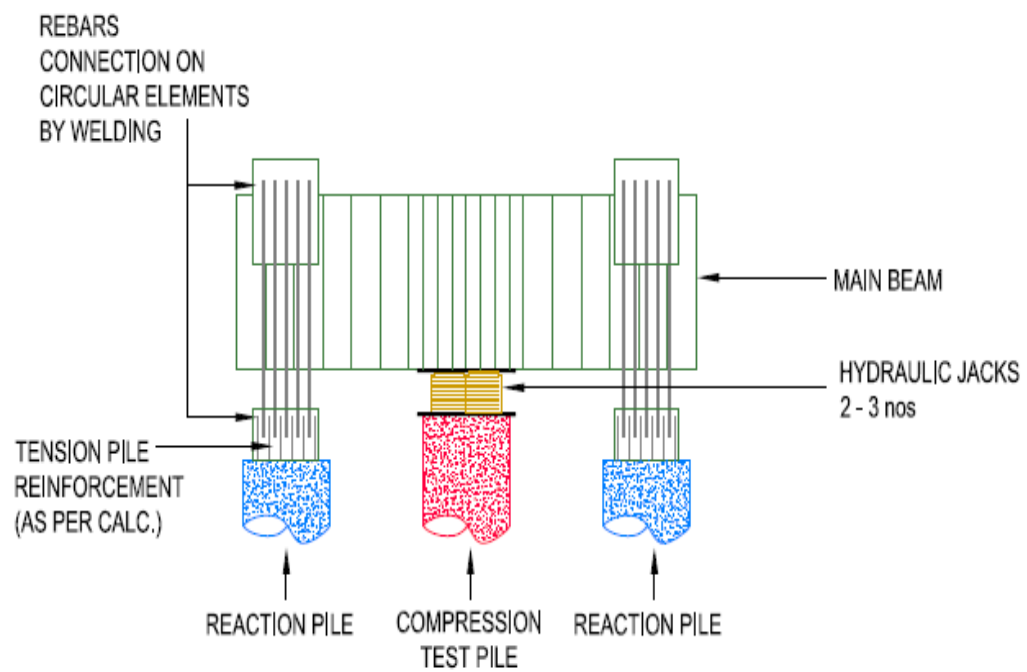


Figure 6.47. Static load test setup illustration

It is to be noted that test was interrupted at the end of the first cycle due to leakage in hydraulic jacks. Loading resumed from zero per cent the following day.

Dial gauge readings are given in Appendix D. Test results are plotted on Figure 6.48 as displacement vs. load graph based on full step readings and Figure 6.49 as displacement vs. time graph. As a result, following results are obtained from the mentioned static load test on pile P-340:

- The maximum settlement up to 100 per cent of the working load (11,500 kN) is 1,6 mm.
- The maximum settlement at 150 per cent of the working load (17,250 kN) is 2,5 mm.
- The residual settlement after unloading to zero is 0,4 mm.

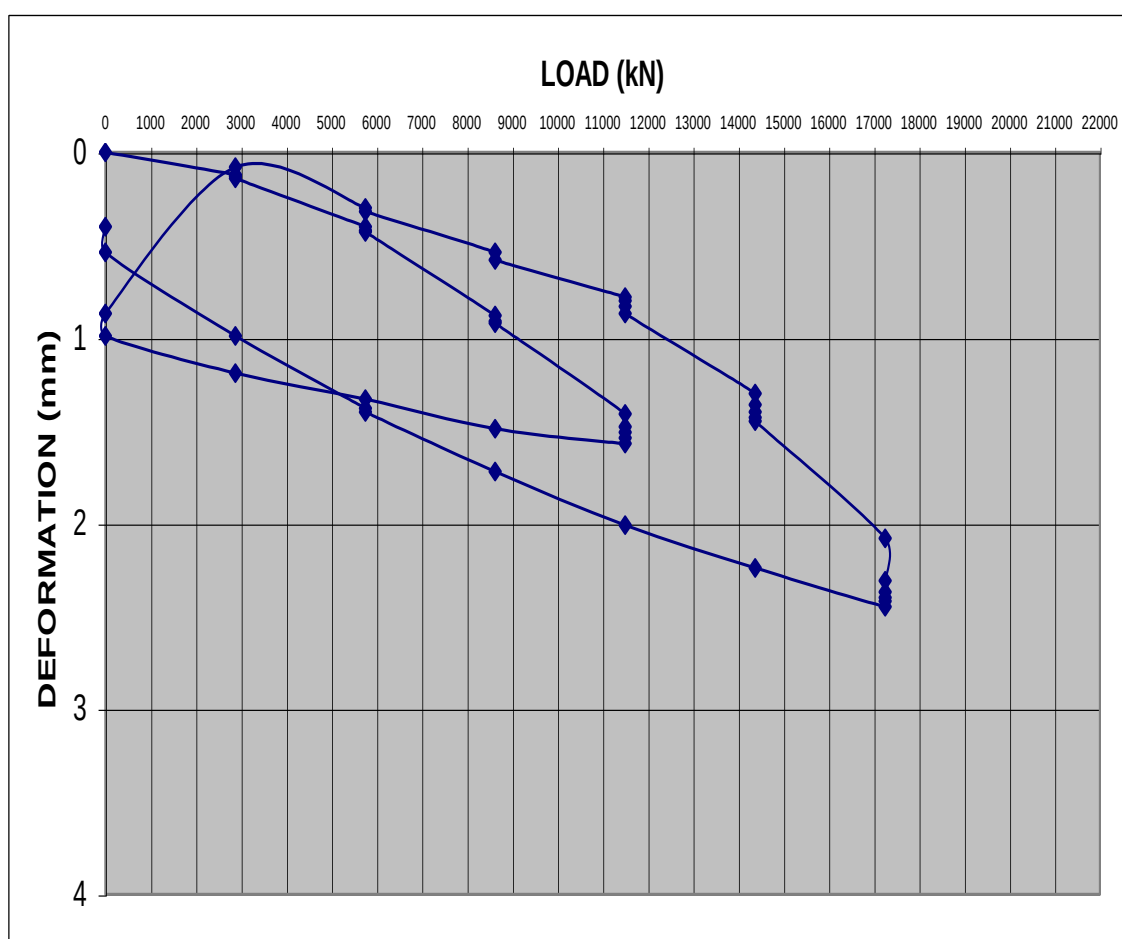


Figure 6.48. Displacement vs. Load graph

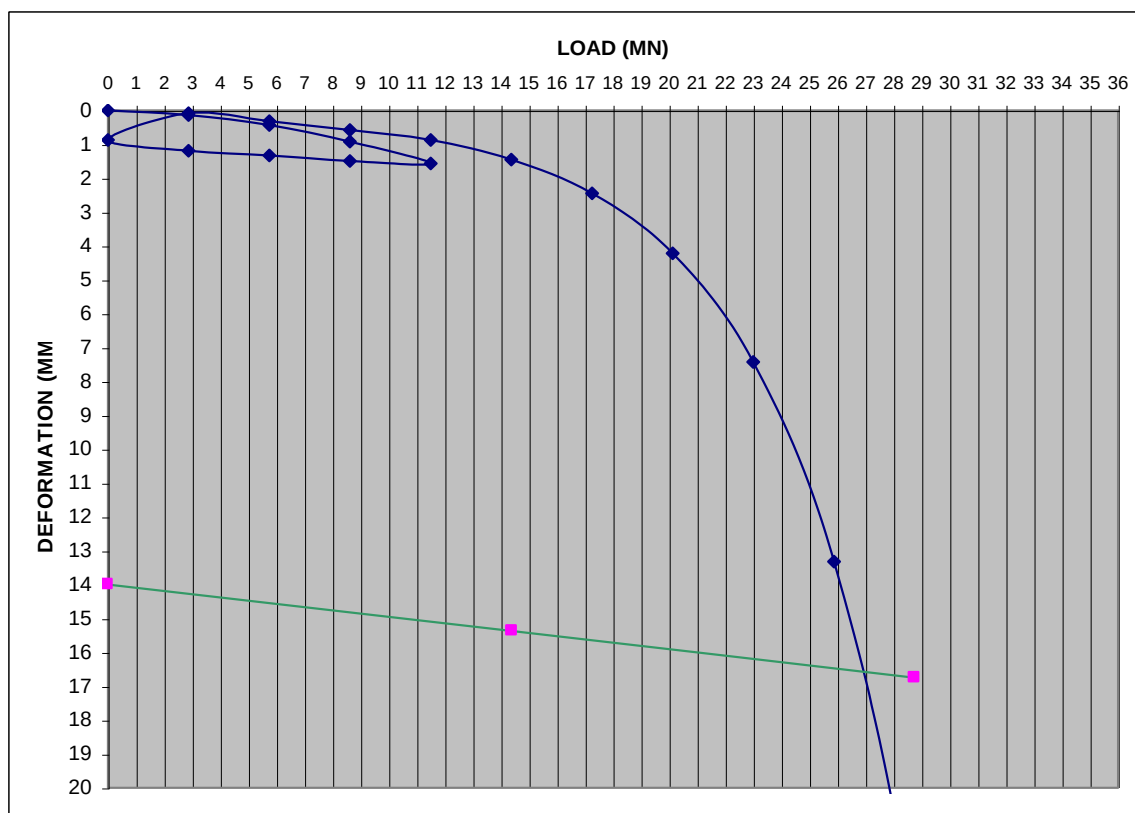


Figure 6.50. Ultimate capacity by Davisson method

6.4.4. Dynamic Load Testing

A dynamic load test was performed on the same working pile named as P-340 in the above mentioned site as shown in Figure 6.51. Field measurement was performed using a Pile Driving Analyser (PDA) – PAK Model, which uses the Case Method and CAPWAP analysis for the numerical computations to better assess the separation between side shear and end bearing. Dynamic measurements were obtained with one pair of accelerometers and two pair of strain transducers attached at a convenient location below the top of the pile (at least 1,5 times the diameter). The PDA processed the records from the hammer blow and calculated the values for the maximum transferred energy, the maximum compressive and tensile stresses and an evaluation of the pile activated static bearing capacity as calculated by the case method assuming a damping factor. For each blow, pile strains were converted to forces and accelerations to velocities as a function of time and evaluated according to the case method solutions by the PDA in real time in the field.



Figure 6.51. Dynamic test preparation

Force and velocity records were viewed on the PDA's graphic screen to evaluate the data quality, the pile integrity and aspects of soil resistance. Stored dynamic data and PDA field results were the basis of the office analysis presented in following tables. Following Table 6.42 shows the pile details. The records of pile force, velocity, displacement and energy for the tested pile are presented in Appendix E.

Table 6.42. Pile details

Pile no.	Diameter (cm)	Length of pile (m)	Working Load (tons)	Test Load (tons)
P – 340 (C1)	120	21.50	1150.0	1725.0

First, the acceleration (a) and strain (e) at the pile head were obtained by accelerometers and strain gauges and recorded by the PDA. Velocity (V) was obtained by the PDA by integrating the acceleration-time history. Force (F) was calculated using the relationship:

$$F = EA\epsilon \quad (6.53)$$

where,

E = Young's modulus of the pile material.

A = Pile cross sectional area.

The testing was accomplished with the use of 20 tons of drop weight hammer, released vertically from a height about 110 cm. The piles were protected with three plywood cushions.

The ram weight (W) and stroke height (H) values were used by PDA to determine the potential energy (PE) per blow:

$$P.E. = m \cdot g \cdot h = W \cdot H \quad (6.54)$$

During the ram drop, the potential energy transformed into kinetic energy (KE). At the instant of impact, this energy is theoretically:

$$K.E. = 0,5 \times m \times v^2 \quad (6.55)$$

where,

m = ram mass.

v = the ram velocity at impact.

The energy reaching the pile head is termed transferred energy. The maximum transferred energy (EMX) is equal to the work done on the pile. However, due to many

factors such as diesel gas pressures, valve timing and mechanical friction, the energy at impact typically ranges between only 50 and 70 per cent of the potential energy (Hussein and Likins, 2001).

Therefore, electronic devices like PDA is the best option to be used to evaluate hammer performance for bearing capacity assessments. EMX is computed by the PDA from measured force and velocity records by integrating the product of force (F) and velocity (V):

$$EMX = \int F \cdot V \cdot dt \quad (6.56)$$

Figure 6.52 illustrates the hammer, driving system and pile with energy transfer concepts.

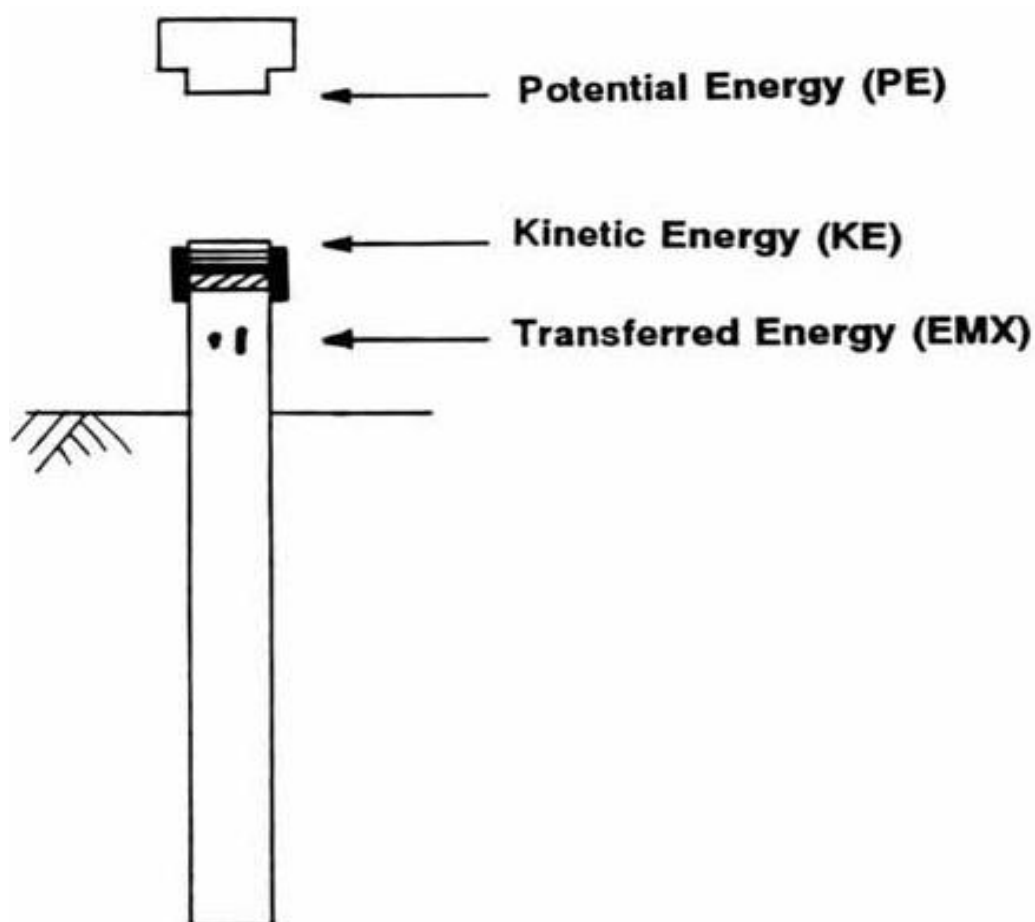


Figure 6.52. Energy transfer from hammer to pile (Hussein and Likins, 2001)

Measurements indicated that after hammer impact, the dynamic axial compressive stress wave speed for the tested piles averaged 4000 m/s for the concrete, using a specific weight of 2.4 tons/m³. The dynamic elastic modulus for the concrete in the tested piles was 392 tons/cm². In the field, the PDA interpreted the measured dynamic data according to the case method equations. Selected output from the PDA typically included values such as the measured force and calculated stress maximum, transferred energy, efficiency of the hammer blow and case method calculation of the mobilised static capacity.

CAPWAP analysis was conducted on the tested piles using measured dynamic pile data and wave equation type analysis as a system identification process employing signal matching techniques. The output is presented in Appendix E and Tables 6.43 and 6.44.

Table 6.43. CAPWAP final results

Soil Segment No.	Distance Below Gages (m)	Depth Below Grade (m)	R _u (tons)	Force in Pile (tons)	Sum of R _u (tons)	Unit Resistance (Depth) tons/m	Unit Resistance (Area) tons/m ²	Smith Damping Factor (s/m)	Quake (mm)
					1789.9				
1	3.1	0.6	30.0	1759.9	30.0	14.63	3.88	1.364	1.000
2	5.1	2.6	89.9	1670.0	119.9	43.92	11.65	1.364	1.241
3	7.2	4.7	119.9	1550.1	239.8	58.55	15.53	1.364	1.334
4	9.2	6.7	140.6	1409.5	380.4	68.68	18.22	1.364	1.334
5	11.3	8.8	144.9	1264.6	525.3	70.75	18.77	1.364	1.334
6	13.3	10.8	157.6	1107.0	682.9	76.98	20.42	1.364	1.334
7	15.4	12.9	170.5	936.5	853.4	83.28	22.09	1.364	1.313
8	17.4	14.9	181.0	755.4	1034.5	88.42	23.45	1.364	1.029
9	19.5	17.0	195.1	560.3	1229.5	95.27	25.27	1.364	0.806
10	21.5	19.0	251.4	309.0	1480.9	122.76	32.56	1.364	0.653
Avg.			148.1			77.94	19.18	1.364	1.097
Skin									
Toe			309.0				273.20	0.391	0.800

Total CAPWAP Capacity was 1789.9 tons; along shaft 1481 tons and at toe 309 tons.

Table 6.44. Extrema table

Pile Segment No.	Distance Below Gages (m)	Max. Force (tons)	Min. Force (tons)	Max. Compressive Stress (tons / cm ²)	Max. Tension Stress (tons / cm ²)	Max. Transferred Energy (tonne - m)	Max. Velocity (m/s)	Max. Displacement (mm)
1	1.0	1837.9	-147.7	0.163	-0.013	12.89	2.4	10.884
2	2.0	1896.9	-162.3	0.168	-0.014	9.13	1.1	6.437
3	3.1	1951.8	-212.5	0.173	-0.019	8.91	1.1	6.114
4	4.1	1940.0	-245.9	0.172	-0.022	8.33	1.0	5.789
5	5.1	1997.1	-282.6	0.177	-0.025	8.10	1.0	5.456
6	6.1	1858.6	-251.7	0.164	-0.022	7.02	0.9	5.137
7	7.2	1912.7	-253.9	0.169	-0.022	6.79	0.9	4.810
8	8.2	1726.9	-191.2	0.153	-0.017	5.66	0.8	4.499
9	9.2	1773.0	-187.8	0.157	-0.017	5.44	0.8	4.174
10	10.2	1576.4	-138.3	0.139	-0.012	4.37	0.7	3.869
11	11.3	1615.4	-152.0	0.143	-0.013	4.16	0.7	3.553
12	12.3	1434.7	-96.3	0.127	-0.009	3.29	0.6	3.265
13	13.3	1496.2	-117.1	0.132	-0.010	3.13	0.6	2.982
14	14.3	1333.2	-46.6	0.118	-0.004	2.44	0.5	2.739
15	15.4	1370.4	-65.5	0.121	-0.006	2.32	0.5	2.504
16	16.4	1168.5	-0.2	0.103	-0.000	1.79	0.4	2.313
17	17.4	1182.7	-0.2	0.105	-0.000	1.72	0.3	2.131
18	18.4	945.3	-0.2	0.084	-0.000	1.29	0.3	1.990
19	19.5	951.2	-0.2	0.084	-0.000	1.25	0.3	1.849
20	20.5	707.3	-0.2	0.063	-0.000	0.88	0.3	1.747
21	21.5	706.2	-0.2	0.062	-0.001	0.42	0.3	1.636
Absolute	5.1			0.177			(T = 22.0 ms)	
	5.1				-0.025		(T = 35.3 ms)	

The total soil resistance, (R) acting during pile driving with the hammer blow was calculated using the expression (C. Ho and H. Lim, 2008):

$$R = \frac{F_{10} + F_{15} + F_{20} + F_{25}}{2} \quad (6.57)$$

where,

$$t_2 = t_1 + 2L / c \quad (6.58)$$

L = Length of pile below gages,

c = Wave speed in pile material

Z = Pile impedance

The static soil resistance (R_s) was computed using the Case method (Rausche et al., 1985). It was assumed that all the damping effects were concentrated at the pile toe and was given by the relationship (C. Ho and H. Lim, 2008):

$$R_d = j_c Z V_{toe} \quad (6.59)$$

where,

j_c = Case damping coefficient

V_{toe} = Velocity at pile toe

Z = Pile impedance

The static component of the soil resistance (R_s) was obtained by subtracting the dynamic component (R_d) from the total resistance, R resulting in the expression (C. Ho and H. Lim, 2008):

$$R_s = \frac{F_{10} + F_{15} + F_{20} + F_{25}}{2} - R_d \quad (6.60)$$

In Table 6.45, resistance values with various damping factors are presented. Time t_1 was chosen to coincide with the peak force (FMX) to give the maximum static resistance (RMX). Table 6.46 presents the pile parameters used in analysis.

Table 6.45. Case Method

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RS1	953.9	931.0	908.0	885.1	862.1	839.1	816.2	793.2	770.3	747.3
RMX	989.8	931.0	908.0	885.1	862.1	839.1	816.2	793.2	770.3	747.3
RSU	1755.4	1812.5	1869.7	1926.9	1984.1	2041.3	2098.4	2155.6	2212.8	2270
RAU= 0.2 (tons) ; RA2 = 953.6 (tons)										

Current CAPWAP $R_u = 1789.9$ (tons) ; Corresponding $J (R_s) = 0.00$; $J (R_x) = 0.00$

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	RLT
m/s	m/s	tons	tons	tons	mm	mm	tonne-m	tons
0.44	0.01	486.5	697.0	1851.5	1769	1.000	1.7	1238.1

Where;

EMX = Maximum value of energy transferred into the pile

FMX = Maximum measured compression force

VT1*Z = velocity times pile impedance = Force

RMX = Maximum static resistance (case method estimate)

J = damping coefficient

Table 6.46. Pile profile and pile model

Depth (m)	Area (cm ²)	E-Modulus (tons/cm ²)	Specific Weight (tons/m ³)	Circumference (m)
0.00	11309.70	392.0	2.403	3.770
21.50	11309.70	392.0	2.403	3.770
Toe Area	1.131 m ²			
Top Segment Length = 1.02 m, Top Impedence = 1108.42 tons/m/s				
Pile Damping = 2.0 %, Time Increment 0.256 ms, Wave Speed 4000.0 m/s				

The activated capacity for this pile are tabulated in Table 6.47.

Table 6.47. Capacity evaluation results for pile P-340 (C1)

Pile no.	Nominal Diameter (cm)	Activated Total Static Capacity (Tons)	Activated Skin Friction Capacity (Tons)	Activated Toe Resistance Capacity (Tons)	Deflection at Activated Capacity (mm)
P – 340 (C1)	120	1789.9	1480.9	309.0	8.0

Utilizing the bottom force and velocity measurements obtained with the dynamic sensors embedded in the pile, bottom force and velocity estimates from the CAPWAP program were matched to measurements by adjusting the magnitude of static resistance, quake and damping parameters for both shaft and toe. Initially the main separation between side shear and end bearing was determined utilizing both peak force and velocities at the pile tip. The side shear distribution and the damping coefficient were modified to obtain a better match utilizing the signal matching capabilities of CAPWAP (C. Alvarez and B. Zuckerman, 2006). The ultimate soil resistance calculated by CAPWAP analysis for the dynamic test was 1790 tons, of which 1481 tons of those were estimated to be in side shear and 309 tons in end bearing. Test results at Appendix E also show a comparison between measured toe force and velocity, the latter multiplied by the pile impedance, and the equivalent readings from the embedded dynamic sensors. The correlations between the pile impedance, wave speed, velocity and force can be interpreted using correlations showed in Chapter 5.4.3.2.

Based on results of the CAPWAP analysis for the pile the displacement at the design working load of the pile, at the maximum achieved load as well as the net displacement after release of the load for the tested piles are tabulated below at Table 6.48.

Table 6.48. Displacement evaluation results for pile P-340 (C1)

Pile no.	Nominal Diameter (cm)	Settlement at working load (mm)	Settlement at Test load (mm)	Net settlement after release of the load (mm)
P – 340 (C1)	120	3.82	6.65	2.71

Based on the results of high strain dynamic testing, it is concluded that the tested piles have no integrity problems and can safely carry the design working loads in

Undoubtfully, by the rapid improvement of electronical technology, soon more sophisticated devices will be available which will show much more detailed results. At site, one per cent of all working piles were static load tested. Five per cent of the working piles were dynamic load tested. Moreover, 20 piles were preliminary static load tested and four piles were both dynamic and static load tested. The high number of testing would reduce the uncertainty so a very low safety factor could be assigned. Therefore, the new safety factor was selected as 1.5. So, new allowable pile capacity was calculated as:

$$Q_{all} = \frac{Q_{ult}}{FS} = \frac{179.4}{1.5} = 119.6 \text{ kN} \quad (6.62)$$

Since the allowable capacity was larger than the design capacity and displacements were lower than required value of 10 mm, this pile design was allowable. Instead of high number of preliminary static load tests on a site, conducting less number of tests with a combination of static and dynamic loading is much preferable, since obtaining same capacity and displacement values is possible with a lower cost.

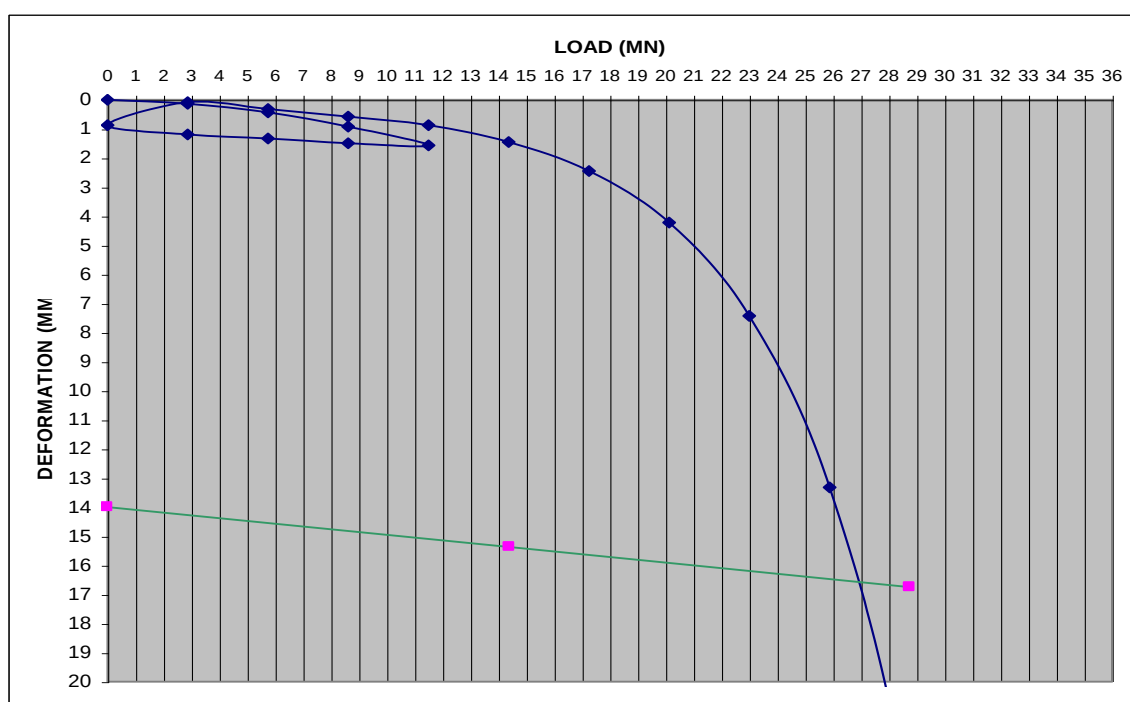


Figure 6.53. Pile capacity assessment by Davisson offset load method

6.5. Integrity Assessment: DM Sport Complex Site, Dubai, U.A.E.

6.5.1. Cross-hole Sonic Logging Test

Cross-hole sonic logging tests were carried out on 35 piles on the site in accordance to the methods presented at Section 5.5 in order to evaluate the integrity of the shaft such as cracks, voids, poor quality concrete, soil and slurry intrusions. In the shaft, the following Table 6.49 presents the data for 14 of the tested pile specifics. For each pile four sonic access tubes were installed, although three was enough for 90 mm. diameter piles as stated in Section 5.5.3.

Table 6.49. Cross-hole sonic logging test schedule

SI. No.	Pile No.	Tube Length (m)	Diameter (cm)	No. of Tubes
1	P5-29	17.50	90	4
2	P5-85	16.30	90	4
3	P5-90	17.00	90	4
4	P7-224	16.30	90	4
5	P7-228	17.30	90	4
6	P7-217	17.00	90	4
7	P5-020	17.00	90	4
8	P-075	17.00	90	4
9	P5-025	17.00	90	4
10	P-077	17.00	90	4
11	P7-156	16.00	90	4
12	P7-153	15.50	90	4
13	P-337	21.70	120	4
14	P-407	21.80	120	4

Sonic transmitter generating sonic pulse at a minimum frequency of 50,000 Hz. and sonic receiver (Piezoelectric accelerometer) capable of picking the signals of same frequency were used as an equipment in the test. Cross-hole analyzer (CHA) compatible PC, Pentium processor with Windows operating system capable of data acquisition with a minimum 12 bit A/D converter with a sampling frequency at least 50,000 Hz was used for recording, storage, built in data interpretation and retrieval of storage data including an output to printers and optional printer for field as shown in Figure 6.54. It provided

immediate on-screen field measurements and results with clear graphical presentation of signal time, velocity and or energy in either metric or English units. Connection cables for both transmitter and receiver with PC were up to 60 meters.



Figure 6.54. Cross-hole analyser compatible PC

In the cross logging system the source (transmitter) is lowered to the bottom of the tube and the receiver is lowered to the bottom of another tube (Figure 6.55). The source (transmitter) and the receiver were pulled simultaneously to allow the horizontal ultrasonic pulse velocity to be measured, offering excellent results in pinpointing the locations and assessing the magnitude of a defect both horizontally and vertically. In equidistant tubes, uniform good quality concrete yields uniform travels with reasonable pulse wave speed and signal strength. Non uniformities such as contamination, soft concrete, honeycombs, voids, or inclusions exhibit delayed arrival. Furthermore a depth wheel controls the resolution of the collected data. Typically the source (transmitter) and the receiver are pulled to the top of each shaft, thus giving a complete assessment of the concrete quality between the two tubes. All possible pathways were checked and an additional opposing diagonal CSL tests were performed to check the integrity of the inner core of the shaft. Orientation of testing is shown in Figure 6.56.



Figure 6.55. Installing transmitter cables down the access tubes

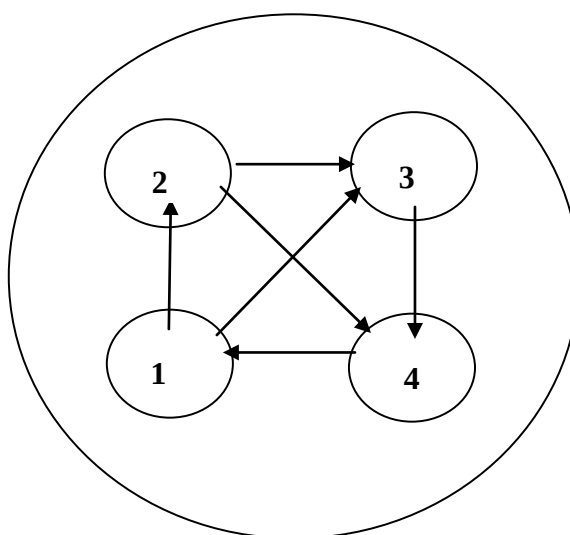


Figure 6.56. Orientation of testing

A graphical presentation of the ultrasonic profile for each of the tubes of the tested piles along with the relative energy & the arrival time are attached in Appendix F. The interpretation of test data shown in the PC is shown below at Figure 6.57. The comments for the test results are described below in Table 6.50.

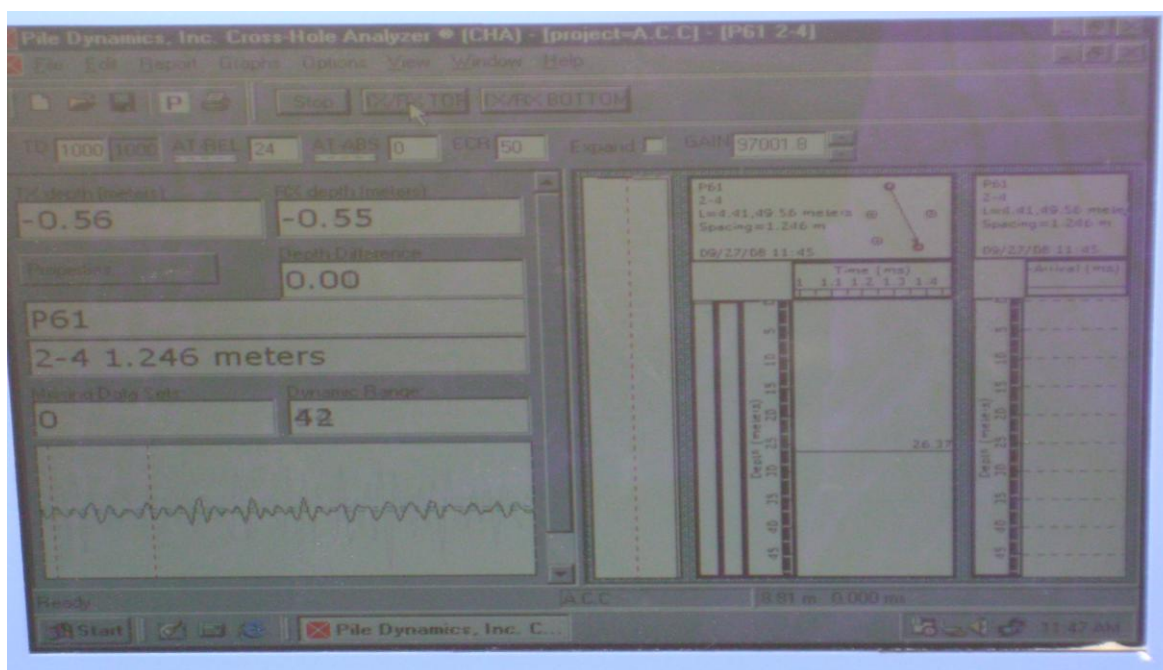


Figure 6.57. Test data displayed from PC screen during testing

Table 6.50. Comments for the tested piles

Pile No.	Comments
P5-29	Tube #4 was blocked at -11.50 m
P5-85	No Comment (OK)
P5-90	No Comment (OK)
P7-224	No Comment (OK)
P7-228	Tube #4 was blocked at -3.00 m
P7-217	No Comment (OK)
P5-020	No Comment (OK)
P-075	No Comment (OK)
P5-025	No Comment (OK)
P-077	No Comment (OK)
P7-156	No Comment (OK)
P7-153	Tube #1 was blocked at -14.00 m & Tube #4 was blocked from the top
P-337	No Comment (OK)
P-407	No Comment (OK)

According to the test data graphics presented in Appendix F, the interpretation and conclusion for defected piles are shown below:

- Pile No. P5-29 : The pile was 90 cm. in diameter. It is to be noted that one of the pipes (fourth pipe) was blocked at -11.50 m. from the top of the pile; therefore the testing was performed through the use of the remaining three tubes. According to the graphs, the sonic readings showed no significant defects at the tested sections; hence the pile is concluded to be ok for the depth that was tested.
- Pile No. P7-228 : The pile was 90 cm in diameter. It is to be noted that one of the pipes (fourth pipe) was blocked at -3.00 m. from the top of the pile; therefore the testing was performed through the use of the remaining three tubes. According to the graphs, the sonic readings showed no significant defects at the tested sections; hence the pile is concluded to be ok for the depth that was tested.
- Pile No. P7-153 : The pile was 90 cm in diameter. It is to be noted that two of the pipes were blocked; first pipe was blocked at -14.00 m. and fourth pipe was blocked from the top of the pile. Therefore the testing was performed through the use of the remaining three tubes, excluding the fourth pipe. According to the graphs, the sonic readings showed no significant defects at the tested sections; hence the pile is concluded to be ok.

6.5.2. Low Strain Integrity Testing

Low strain integrity testing of 30 piles were performed at the above referenced site. The tested piles were 90 cm. and 120 cm. diameter cast in place concrete piles with each pile designated by its identify number. According to design project, the 90 cm. dia. piles had 17.0 m. length and 120 cm. dia. piles had 21.80 m. length. The purpose of the testing was to determine if the piles contained any flaws involving inclusions of foreign material, necking, bulging and also piles that were too short. The tests were accomplished with the use of the PDI Pile Integrity Tester, P.I.T. Other hardware used was an accelerometer, power supply, a portable computer capable of converting analogue data into digital form traceable to international standards. The equipment and instrumentation details are explained in Chapter 5.6. The stress wave speed for all piles was assumed to be four (m/ms). This magnitude was generally representative for concrete and related best for the

majority of the piles. Depending on various factors however, the actual wave speed would differ by 10 per cent, sometimes even more (M. Amir, 2008).

The records of all the tested piles as enclosed in Appendix G except P-243 and P-234 showed no flaws and therefore have been classified as Class-one. However, the piles of P-243 and P-234 had less actual pile depths than design values so declared as "too short" and classified as Class-three (Chapter 5.6.4). The extra length ratios (ELR) of the piles, which is the amount by which the measured pile length deviates from the planned length, were presented in Table 6.51. Figure 6.58 shows the cumulative distribution of ELR for all piles by using the data taken from the results of the test. It was observed from figure that 80 per cent of the tested piles were at least five per cent longer than design lengths and the majority of the piles were between five per cent and 10 per cent longer than design lengths. During construction usually boreholes are drilled one or two meters more than needed due to the fact that easy penetration of soil at these small depths makes it hard for the operator to finish drilling on the exact design depth. Whatever, during drilling the borehole, usually operators drill more than design length because a possible collapse in the pile after finishing of drilling may cause the steel cage not properly installed into the pile shaft and delays in construction schedule might happen. The formula of ELR is (I. Amir, 2008):

$$ELR = \frac{measured - plan}{plan} \quad (6.63)$$

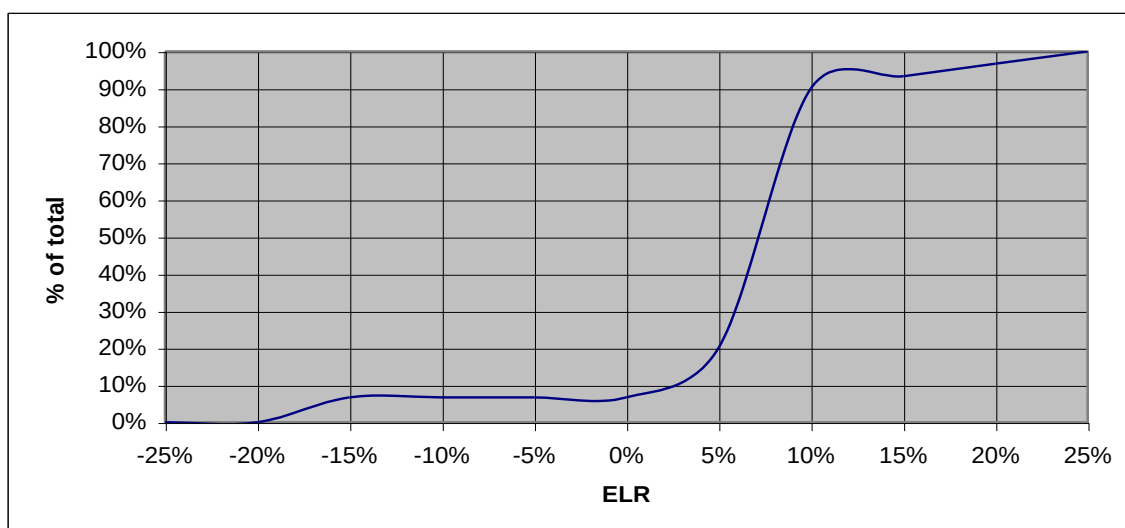


Figure 6.58. Cumulative ELR distribution

Table 6.51. Pulse-echo testing results

Pile No.	Pile Diameter (cm)	Design Pile Length (m)	Actual (measured) Pile Length (m)	Slenderness Ratio (L / D)	ELR
P-21	90	17.0	18.53	21	0.09
P-23	90	17.0	21.24	24	0.249
P-25	90	17.0	19.32	21	0.136
P-74	90	17.0	18.56	21	0.092
P-76	90	17.0	19.58	22	0.152
P-78	90	17.0	18.52	21	0.089
P-80	120	21.80	22.19	18	0.018
P-82	120	21.80	22.00	18	0.009
P-151	90	17.0	17.59	20	0.035
P-153	90	17.0	17.86	20	0.051
P-216	90	17.0	17.00	19	0.000
P-217	90	17.0	17.37	19	0.022
P-218	90	17.0	18.70	21	0.100
P-220	120	21.80	22.05	18	0.011
P-222	120	21.80	22.11	18	0.014
P-223	90	17.0	17.26	19	0.015
P-224	90	17.0	17.26	19	0.015
P-226	90	17.0	17.08	19	0.005
P-243	120	21.80	17.78	15	-0.184
P-244	120	21.80	17.53	15	-0.196
P-334	120	21.80	23.16	19	0.062
P-335	120	21.80	23.31	19	0.069
P-337	120	21.80	23.00	19	0.055
P-338	120	21.80	22.90	19	0.050
P-339	120	21.80	23.30	19	0.069
P-341	120	21.80	23.58	20	0.082
P-342	120	21.80	22.46	19	0.030
P-344	120	21.80	22.71	19	0.042
P-346	120	21.80	22.97	19	0.054
P-407	120	21.80	22.57	19	0.035

6.5.3. Results

Both cross-hole sonic and low strain integrity tests generally are used for quality assessment of bored piles since their results not only show the presence of discontinuities, flaws and cracks in the pile but also help the engineer to check the actual vs. planned pile lengths. Integrity testing methods provide length-checking of a pile, but no capacity assessment is possible.

Comparing the records of the test data presented in Appendix F and G, it can be said that sonic pulse echo method is more preferable than sonic logging method in determining the length of the tested piles. First, the piezoelectric probes in access tubes in the sonic logging test are sensitive so the blocking of the tubes can not be directly related to pile shortness. A flaw may be present at that location and although the pile is deeper, the signals may not be properly taken to show it. However, in pulse echo method the shock waves are introduced to the pile head and impulses reach until the toe, so the exact location of the toe of the pile is determined by the length of the echo digitalized by the computer, and the length of the pile is directly dependent upon the value of the propagation velocity so the extra length ratios can be computed from records.

Sonic testing method is more costly; it needs more time and qualified workmanship to conduct. However, since probes themselves lowered down to pile shaft during installation, the records give more detailed and realistic results and the chance to encounter a flaw discontinuity in the pile is more. According to M. Amir and I. Amir (2008), testing using sonic access tubes have a higher (typically three times) flaw detection probability than low integrity testing with pulse echo method. Moreover, improving technology now also makes it possible to digitalize the pile body in 3D tomographic, which is an excellent way to deeply investigate the pile quality. Besides, with longer piles, the higher amplification also magnifies noise, making toe reflection of the wave impulses less obvious for pulse echo method thus more blows are needed to achieve a proper reflectogram and intrusive methods. Therefore, longer and larger piles are preferred to be subjected to cross-hole testing. Recent studies conducted by J. M. Amir and I. Amir to detect flaws by cross-hole sonic testing showed that the anomaly created by a flaw depends not only on the flaw's size, but also its location.

As a result, the best way to assess the quality of a pile is to conduct both cross-hole sonic and low integrity pulse-echo testing methods on a site; the latter one with more numbers. As mentioned in Chapter 5.6.4, at least 50 per cent of the piles should be subjected to integrity testing on a site consisting of more than 40 piles. Due to the facts that pulse echo method is more cost-effective, requires less testing time and have a less possibility of detecting flaws and discontinuities; more number of piles were conducted to a low integrity test on the above mentioned site (14 cross hole logging against 30 pulse echo method testing).

7. CONCLUSIONS

There are several methods for the assessment of the load bearing capacity and quality of bored piles. Some of these methods such as static or dynamic analysis are based on theoretical correlations derived from soil or rock parameters obtained by soil investigation. This methodology include many assumptions and uncertainties since is too general and do not depend on construction practice. On the other hand, some methods are instrumental and require conduction of field tests on system or trial bored piles on field. These methods give more accurate datas on soil or rock behavior against design working loads acting downward from the pre-designed structure, therefore reduce the risk of a potential failure. Besides, some of them such as sonic logging and pulse-echo testing check the construction quality which is also a crucial parameter in the determination of load capacity.

Keeping the risk of foundation failure below an allowable level is the goal for any foundation. To accomplish this, safety factors are applied to the ultimate pile capacities derived from methods stated above to calculate an allowable design or working load for the piles. Several deep foundation codes has been published in recent years recommending safety factors for each method; higher values for theoretical methods and lower values for instrumental methods. Moreover, codes recommend the lowest safety factors when the number of tested piles increase on the site. In addition to that, some tests are more sophisticated and use more technological devices than others, giving more idealized results so that safety factors can be decreased. The selection of the capacity assessment method for bearing capacity determination is crucial in piling work, due to limited time and cost restrictions. Methods such as osterberg cell loading not only saves a lot of time compared with a typical static loading test, but also gives more detailed and reliable results for skin and toe resistances and also for load-displacement values. However, osterberg cell method is the most expensive method. Therefore a detailed comparison of the benefit of each test to pile design with its cost disadvantage should be conducted for a more economic design.

In addition, in espacially prestigious structures consisting of high service loads, assessment of vertical load bearing capacity is not sufficient for a proper design. Since piles are structural members and are designed as a part of the whole structure, performance

criteria governs the design methodology and serviceability limit state is accepted to be the performance limit state. Therefore, displacement based design should also be applied to analysis and maximum vertical displacement values obtained during the test under working and ultimate loads should also be compared with the design values for an allowable design. According to the performance criteria, below equation shall be ensured:

$$\delta_v \leq \delta_{cr} \quad (7.1)$$

where,

δ_v = Displacement under service load

δ_{cr} = Allowable displacement for serviceability limit state

The effects of both vertical displacement and differential settlement to pile rafts are much more amplified on described mega-structures in these case studies due to their large weights, heights and imposed loads. When height of the structure increases, inclination angle will proportionally increase. Therefore, maximum horizontal displacement is proportional to height of structure as shown below at Figure 7.1.

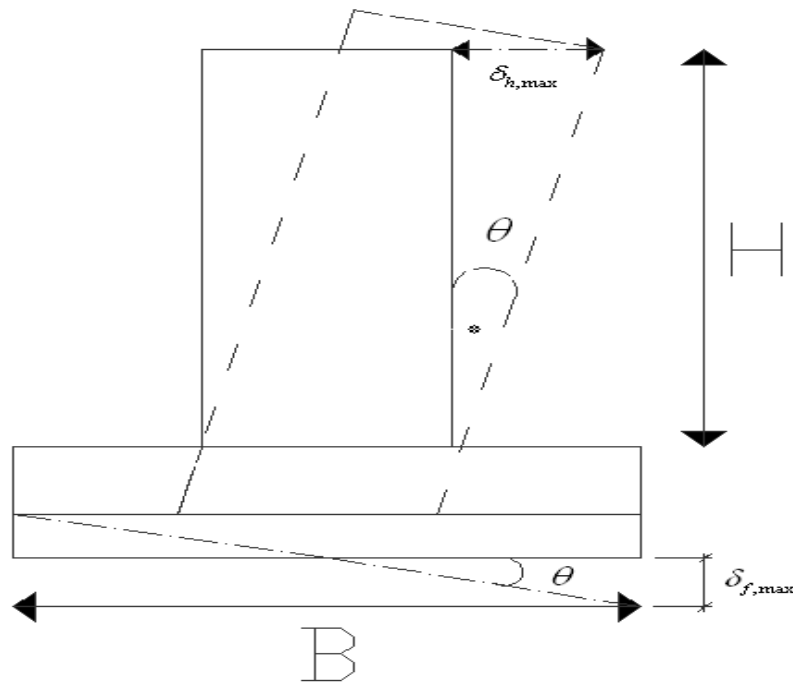


Figure 7.1. Determination of maximum displacement

Considering the displacement based design methodology of piled rafts,

$$\delta_{h,max} = H \left(\frac{\delta_{f,max}}{B} \right) \quad (7.2)$$

where,

H = Height of the structure.

B = Width of structure.

$\delta_{f,max}$ = Maximum differential displacement.

$\delta_{h,max}$ = Maximum horizontal displacement at the top of the structure.

θ = Inclination angle.

Since in Case Study-1 and Case Study-2, height of the structures are approximately 195 meters and width is 60 meters, according to formula presented above, maximum displacement value is calculated to be 6,5 times amplified than displacement measured at the base. The serviceability limit was requested to be 10 mm. for all the case studies by the structural designer, meaning that maximum allowable displacement was 65 mm for the main structure.

As a consequence, in this thesis three case studies were presented, including both the pile design by static method of analysis and the effect of preliminary pile tests to the design, all in rock sockets. In Case Study-1, first the pile schedule determined by static analysis using the theoretical pile design correlations were presented, and then results of two static load tests, one conventional and the other with strain gauges inserted, and which were conducted on preliminary piles, were compared with each other and with the pile design parameters. Both static load tests presented lower displacement values than serviceability limit values, displaying the effectiveness of the tests. Although using strain gauges on a static load test gave more detailed results than conventional top down static loading test and enabled the engineer to design more economical piles, due to the total cost and time loss and also the high amount of assumptions it was concluded that static load testing alone would not be enough for a proper design. Besides, the undetermined separate

behavior of skin and toe resistances prevented a more economical design, since probably the toe resistance was overvalued.

In Case Study-2, under similar soil conditions and design parameters with the first case, design of a selected pile with static method of analysis was compared with a design of the same pile type on the site using the results of an osterberg cell loading test. Although displacement values calculated by static method of analysis were higher than performance limit, osterberg test results presented lower values than performance limit for the same designed pile parameters so it was determined that osterberg cell test method is an effective method for preliminary design. The results provided a clear separation of end bearing and skin resistances and indicated that skin friction was much more higher than predicted in static analysis, therefore it was concluded that toe resistance can be ignored in the analysis, leading to a more economical design. The profit by the reduction in pile lengths using osterberg cell results of all 1200 mm. diameter piles was more than the test cost, so osterberg testing was decided to be economical for this site. However, it should be noted that for large projects consisting of high design working loads and number of piles this test may be economical, whereas for small projects might be uneconomical due to its high test cost. Amongst all the tests, osterberg cell test more accurately explained the low than predicted value of toe resistance and that in rock sockets skin resistance was more than expected.

In Case Study-3, pile design was based on both static analysis and preliminary static load tests. At this time, by conducting both static and dynamic load test on the same working pile, not only capacity but also quality of the pile was tested. The main aim of this study was to compare the capacity and displacement of the tested pile under design working load with the design capacity and displacement values of the preliminary static load test results, to estimate the effectiveness of the dynamic load test. Results showed that although both capacity and displacement values were allowable for each test, the speed of the dynamic load test and its effect on the decrease of the safety factor according to codes would make this test a preferable option as a preliminary load test. Also by CAPWAP analysis toe and skin resistances were separately estimated, however the results were more poor and less accurate when compared with the osterberg load test.

Case Study-3 also evaluated the quality assessment of bored piles based on integrity tests. Despite the low price of the tests, they played an important role by effectively displaying the actual pile length as in pulse-echo method and pile quality as in cross-hole sonic logging method. These tests showed that since quality of piles directly effect pile capacity, construction sequence is as important as design in bored piling.

Modern testing methods using electronic devices are more effective for both capacity and quality assessments and are widely used. Since the development of nanotechnology and electronic sciences offer faster and easy-to-conduct testing methods with more accurate results, there would be a growing increase in the investment of further research on pile testing methods.

APPENDIX A: CASE STUDY 1 – STATIC LOAD TESTING-1

ZEMIN TEKNOLOJISI A.Ş.				PRELIMINARY STATIC LOAD TEST 2000 kN							Consultant		ARUP			
Place			Test no.		TestLoad		20,000 kN			Main Contractor						
Duration			23:50 h													
Extra Load			0 kN		Pile no.		PTP18			Supervisor		AL HAI & AL MUKADDAM				
Max. load			20,000 kN		Pile Dia.		1200									
time			load		dial gauge readings (compression)					total displacement (dial gauges) mm		dial gauge readings (tension)				
real	test	step	kN	bar	1	2	3	4	average			1	2	3	4	
12:35	00:00:00	00:00	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
25.0% P.W.L	12:35	00:00:00	00:00	2,500	75	0.09	0.00	0.01	0.14	0.06	0.06	0.21	0.00	0.11	0.23	
	12:45	00:10:00	00:10	2,500	75	0.09	0.00	0.01	0.14	0.06	0.06	0.22	0.00	0.13	0.24	
	12:55	00:20:00	00:10	2,500	75	0.09	0.00	0.01	0.14	0.06	0.06	0.22	0.00	0.13	0.24	
	13:05	00:30:00	00:10	2,500	75	0.09	0.00	0.01	0.14	0.06	0.06	0.22	0.00	0.13	0.24	
50.0% P.W.L	13:05	00:30:00	00:00	5,000	150	0.27	0.36	0.23	0.35	0.30	0.30	0.61	0.00	0.52	0.77	
	13:15	00:40:00	00:10	5,000	150	0.27	0.37	0.25	0.36	0.31	0.31	0.61	0.00	0.52	0.78	
	13:25	00:50:00	00:10	5,000	150	0.27	0.38	0.26	0.36	0.32	0.32	0.61	0.00	0.53	0.78	
	13:35	01:00:00	00:10	5,000	150	0.27	0.38	0.26	0.36	0.32	0.32	0.61	0.00	0.53	0.78	
0.75%P.W.L	13:35	01:00:00	00:00	7,500	226	0.52	0.66	0.57	0.63	0.60	0.60	2.04	0.08	1.08	2.23	
	13:45	01:10:00	00:10	7,500	226	0.57	0.69	0.62	0.67	0.64	0.64	2.17	0.10	1.11	2.45	
	13:55	01:20:00	00:10	7,500	226	0.57	0.69	0.65	0.69	0.65	0.65	2.32	0.11	1.29	2.46	
	14:05	01:30:00	00:10	7,500	226	0.57	0.69	0.66	0.69	0.65	0.65	2.35	0.11	1.31	2.48	
100.0% P.W.L	14:05	01:30:00	00:00	10,000	301	0.84	0.90	1.06	1.04	0.96	0.96	3.52	0.64	2.47	3.80	
	14:35	02:00:00	00:30	10,000	301	0.89	1.15	1.15	1.12	1.08	1.08	3.75	0.67	2.54	3.89	
	15:05	02:30:00	00:30	10,000	301	0.90	1.20	1.19	1.15	1.11	1.11	3.78	0.68	2.56	3.89	
	15:35	03:00:00	00:30	10,000	301	0.94	1.22	1.21	1.18	1.14	1.14	3.80	0.68	2.58	3.72	
	16:05	03:30:00	00:30	10,000	301	0.95	1.25	1.23	1.20	1.16	1.16	3.95	0.69	2.59	3.75	
	16:35	04:00:00	00:30	10,000	301	0.95	1.27	1.25	1.21	1.17	1.17	3.96	0.69	2.59	3.82	
	17:05	04:30:00	00:30	10,000	301	0.95	1.28	1.26	1.21	1.18	1.18	3.97	0.69	2.59	3.83	
	17:35	05:00:00	00:30	10,000	301	0.95	1.29	1.27	1.22	1.18	1.18	3.98	0.69	2.59	3.84	
	18:05	05:30:00	00:30	10,000	301	0.95	1.29	1.29	1.22	1.19	1.19	4.00	0.69	2.59	3.86	
	18:35	06:00:00	00:30	10,000	301	0.95	1.29	1.29	1.22	1.19	1.19	4.00	0.69	2.59	3.86	
	19:05	06:30:00	00:30	10,000	301	0.96	1.30	1.31	1.22	1.20	1.20	4.00	0.69	2.60	3.86	
	19:35	07:00:00	00:30	10,000	301	0.96	1.30	1.31	1.22	1.20	1.20	4.01	0.69	2.60	3.86	
75.0% P.W.L	20:05	07:30:00	00:30	10,000	301	0.96	1.30	1.31	1.22	1.20	1.20	4.01	0.69	2.60	3.86	
	20:05	07:30:00	00:00	7,500	226	0.95	1.02	1.09	1.20	1.07	1.07	3.30	0.60	2.05	3.02	
	20:10	07:35:00	00:05	7,500	226	0.94	1.01	1.09	1.20	1.06	1.06	3.30	0.59	2.03	3.00	
50.0% P.W.L	20:15	07:40:00	00:05	7,500	226	0.94	1.01	1.08	1.20	1.06	1.06	3.29	0.59	2.02	3.00	
	20:15	07:40:00	00:00	5,000	150	0.78	0.90	0.95	1.05	0.92	0.92	2.59	0.47	1.57	2.27	
	20:20	07:45:00	00:05	5,000	150	0.77	0.90	0.94	1.04	0.91	0.91	2.57	0.45	1.56	2.25	
25.0% P.W.L	20:25	07:50:00	00:05	5,000	150	0.76	0.90	0.94	1.03	0.91	0.91	2.56	0.44	1.56	2.23	
	20:25	07:50:00	00:00	2,500	75	0.55	0.70	0.81	0.83	0.72	0.72	1.51	0.38	0.99	1.26	
	20:30	07:55:00	00:05	2,500	75	0.55	0.70	0.80	0.81	0.72	0.72	1.50	0.37	0.98	1.25	
0.0% P.W.L	20:35	08:00:00	00:05	2,500	75	0.54	0.70	0.79	0.80	0.71	0.71	1.50	0.37	0.98	1.25	
	20:35	08:00:00	00:00	0	0	0.31	0.45	0.51	0.48	0.44	0.44	0.77	0.35	0.55	0.63	
	20:50	08:15:00	00:15	0	0	0.30	0.44	0.50	0.47	0.43	0.43	0.77	0.35	0.53	0.62	
	21:05	08:30:00	00:15	0	0	0.28	0.43	0.50	0.45	0.42	0.42	0.41	0.76	0.34	0.51	
	21:20	08:45:00	00:15	0	0	0.28	0.42	0.50	0.45	0.41	0.41	0.40	0.75	0.34	0.50	
	21:35	09:00:00	00:15	0	0	0.28	0.42	0.49	0.45	0.41	0.41	0.40	0.75	0.34	0.50	

PRELIMINARY STATIC LOAD TEST 2000 kN																
Place			Test no.		Test Load 20,000 kN					Consultant		ARUP				
Duration		23:50 h									Main Contractor					
Extra Load		0 kN		Pile no.		PTP18					Supervisor		AL HAI & AL MUKADAM			
Max. load		20,000 kN		Pile Dia.		1200										
time			load		dia gauge readings (compression)					total displacement (dia gauges) mm		dia gauge readings (tension)				
real	test	step	kN	bar	1	2	3	4	average			1	2	3	4	
100.0% PNL																
21:35	09:00:00	00:00	10,000	301	1.13	1.30	1.32	1.48	1.31	1.31	4.05	0.87	2.56	3.87		
21:50	09:15:00	00:15	10,000	301	1.15	1.31	1.33	1.49	1.32	1.32	4.07	0.89	2.58	3.89		
22:05	09:30:00	00:15	10,000	301	1.16	1.33	1.34	1.50	1.33	1.33	4.07	0.89	2.60	3.90		
22:20	09:45:00	00:15	10,000	301	1.17	1.34	1.36	1.50	1.34	1.34	4.08	0.90	2.60	3.91		
22:35	10:00:00	00:15	10,000	301	1.17	1.34	1.37	1.50	1.35	1.35	4.08	0.90	2.61	3.91		
125.0% PNL																
22:35	10:00:00	00:00	12,500	376	1.42	1.72	1.76	1.95	1.71	1.71	5.23	1.24	3.31	4.90		
22:50	10:15:00	00:15	12,500	376	1.44	1.74	1.77	1.96	1.73	1.73	5.25	1.26	3.33	4.93		
23:05	10:30:00	00:15	12,500	376	1.45	1.75	1.79	1.98	1.74	1.74	5.26	1.28	3.34	4.94		
23:20	10:45:00	00:15	12,500	376	1.45	1.76	1.79	1.99	1.75	1.75	5.27	1.30	3.34	4.95		
23:35	11:00:00	00:15	12,500	376	1.45	1.76	1.79	1.99	1.75	1.75	5.27	1.30	3.34	4.95		
150.0% PNL																
23:35	11:00:00	00:00	15,000	451	2.24	2.35	2.66	3.14	2.60	2.60	6.72	2.00	4.06	5.93		
00:35	12:00:00	01:00	15,000	451	2.28	2.39	2.71	3.17	2.64	2.64	6.75	2.03	4.10	5.96		
01:35	13:00:00	01:00	15,000	451	2.33	2.43	2.75	3.22	2.68	2.68	6.78	2.07	4.12	5.99		
02:35	14:00:00	01:00	15,000	451	2.36	2.45	2.77	3.25	2.71	2.71	6.80	2.10	4.14	6.00		
03:35	15:00:00	01:00	15,000	451	2.38	2.46	2.78	3.26	2.72	2.72	6.81	2.11	4.15	6.02		
04:35	16:00:00	01:00	15,000	451	2.38	2.47	2.78	3.26	2.72	2.72	6.81	2.11	4.15	6.03		
05:35	17:00:00	01:00	15,000	451	2.38	2.47	2.78	3.26	2.72	2.72	6.81	2.11	4.15	6.03		
125.0% PNL																
05:35	17:00:00	00:05	12,500	376	2.35	2.37	2.70	3.22	2.66	2.66	6.54	1.85	3.82	5.37		
05:40	17:05:00	00:05	12,500	376	2.35	2.37	2.70	3.22	2.66	2.66	6.53	1.85	3.81	5.35		
05:45	17:10:00	00:05	12,500	376	2.34	2.36	2.68	3.21	2.65	2.65	6.53	1.85	3.80	5.35		
100.0% PNL																
05:45	17:10:00	00:00	10,000	301	2.12	1.95	2.45	3.00	2.18	2.18	5.92	1.57	3.37	4.62		
05:50	17:15:00	00:05	10,000	301	2.11	1.94	2.44	2.98	2.17	2.17	5.90	1.57	3.36	4.61		
05:55	17:20:00	00:05	10,000	301	2.10	1.94	2.43	2.98	2.16	2.16	5.90	1.57	3.35	4.60		
75.0% PNL																
05:55	17:20:00	00:00	7,500	226	1.85	1.91	2.15	2.80	1.96	1.96	4.90	1.26	2.72	3.65		
06:00	17:25:00	00:05	7,500	226	1.84	1.91	2.14	2.78	1.94	1.94	4.95	1.26	2.71	3.64		
06:05	17:30:00	00:05	7,500	226	1.84	1.91	2.13	2.77	1.94	1.94	4.95	1.26	2.70	3.64		
50.0% PNL																
06:05	17:30:00	00:00	5,000	150	1.61	0.75	1.88	2.34	1.65	1.65	3.76	1.06	2.09	2.68		
06:10	17:35:00	00:05	5,000	150	1.60	0.74	1.86	2.33	1.63	1.63	3.74	1.07	2.09	2.67		
06:15	17:40:00	00:05	5,000	150	1.60	0.74	1.85	2.33	1.63	1.63	3.74	1.07	2.09	2.67		
25.0% PNL																
06:15	17:40:00	00:00	2,500	75	1.33	0.40	1.57	1.91	1.30	1.30	2.42	0.91	1.42	1.64		
06:20	17:45:00	00:05	2,500	75	1.32	0.40	1.56	1.90	1.30	1.30	2.40	0.90	1.41	1.63		
06:25	17:50:00	00:05	2,500	75	1.31	0.39	1.55	1.90	1.29	1.29	2.40	0.90	1.40	1.62		
0.0% PNL																
06:25	17:50:00	00:00	0	0	0.86	0.10	1.07	1.23	0.82	0.82	1.23	0.82	0.80	0.76		
06:40	18:05:00	00:15	0	0	0.85	0.09	1.05	1.23	0.81	0.81	1.20	0.81	0.80	0.75		
06:55	18:20:00	00:15	0	0	0.83	0.09	1.04	1.21	0.79	0.79	1.20	0.80	0.79	0.75		
07:10	18:35:00	00:15	0	0	0.83	0.09	1.04	1.21	0.79	0.79	1.20	0.79	0.79	0.74		
07:25	18:50:00	00:15	0	0	0.83	0.09	1.04	1.21	0.79	0.79	1.19	0.79	0.78	0.74		

TEST DATA A-1			PRELIMINARY STATIC LOAD TEST 2000 kN								Consultant		ARUP				
Place			Test no.		Test Load		20,000 kN				Main Contractor						
Duration			23:50 h									Supervisor		AL HAI & AL MUKADDAM			
Extra Load			0 kN		Pile no.		PTP18										
Max. load			20,000 kN		Pile Dia.		1200										
time			load		dial gauge readings (compression)					total displacement (dial gauges) mm		dial gauge readings (tension)					
real	test	stop	kN	bar	1	2	3	4	average			1	2	3	4		
100.0% P.W.L.																	
07:25	18:50:00	00:00	10,000	301	2.01	1.41	2.28	2.75	2.11	2.11	5.65	1.65	3.23	4.48			
07:40	19:05:00	00:15	10,000	301	2.03	1.42	2.28	2.76	2.12	2.12	5.68	1.65	3.24	4.47			
07:55	19:20:00	00:15	10,000	301	2.04	1.43	2.28	2.77	2.13	2.13	5.69	1.66	3.25	4.48			
08:10	19:35:00	00:15	10,000	301	2.04	1.43	2.28	2.77	2.13	2.13	5.69	1.66	3.25	4.48			
08:25	19:50:00	00:15	10,000	301	2.04	1.43	2.28	2.77	2.13	2.13	5.69	1.67	3.25	4.48			
150.0% P.W.L.																	
08:25	19:50:00	00:00	15,000	451	2.58	2.47	3.12	3.65	2.96	2.96	7.35	2.33	4.36	6.17			
08:40	20:05:00	00:15	15,000	451	2.60	2.48	3.13	3.67	2.97	2.97	7.35	2.34	4.37	6.19			
08:55	20:20:00	00:15	15,000	451	2.61	2.49	3.13	3.68	2.98	2.98	7.36	2.35	4.37	6.20			
09:10	20:35:00	00:15	15,000	451	2.61	2.50	3.13	3.69	2.98	2.98	7.37	2.36	4.37	6.21			
09:25	20:50:00	00:15	15,000	451	2.61	2.50	3.13	3.69	2.98	2.98	7.37	2.36	4.37	6.21			
175.0% P.W.L.																	
09:25	20:50:00	00:00	17,500	526	3.25	3.34	3.74	4.30	3.66	3.66	8.42	2.73	4.93	7.01			
09:40	21:05:00	00:15	17,500	526	3.28	3.37	3.75	4.32	3.69	3.69	8.46	2.75	4.96	7.03			
09:55	21:20:00	00:15	17,500	526	3.30	3.38	3.79	4.33	3.70	3.70	8.47	2.76	4.97	7.05			
10:10	21:35:00	00:15	17,500	526	3.31	3.38	3.80	4.35	3.71	3.71	8.48	2.77	4.98	7.06			
10:25	21:50:00	00:15	17,500	526	3.31	3.38	3.80	4.35	3.71	3.71	8.48	2.77	4.98	7.06			
200.0% P.W.L.																	
10:25	21:50:00	00:00	20,000	602	4.14	4.02	4.78	5.82	4.94	4.94	9.41	3.29	5.57	7.83			
10:40	22:05:00	00:15	20,000	602	4.16	4.05	4.80	5.84	4.96	4.96	9.42	3.32	5.60	7.84			
10:55	22:20:00	00:15	20,000	602	4.20	4.08	4.82	5.87	4.99	4.99	9.43	3.35	5.62	7.85			
11:10	22:35:00	00:15	20,000	602	4.22	4.10	4.83	5.89	5.01	5.01	9.45	3.36	5.63	7.85			
11:25	22:50:00	00:15	20,000	602	4.22	4.10	4.83	5.89	5.01	5.01	9.45	3.36	5.63	7.85			
0.0% P.W.L.																	
11:25	22:50:00	00:00	0	0	1.69	1.65	1.82	2.72	2.22	2.22	1.34	1.12	0.90	0.92			
11:40	23:05:00	00:15	0	0	1.65	1.63	1.80	2.70	2.20	2.20	1.30	1.10	0.91	0.90			
11:55	23:20:00	00:15	0	0	1.64	1.61	1.77	2.68	2.18	2.18	1.30	1.09	0.90	0.90			
12:10	23:35:00	00:15	0	0	1.64	1.61	1.77	2.68	2.18	2.18	1.30	1.09	0.90	0.90			
12:25	23:50:00	00:15	0	0	1.64	1.61	1.77	2.68	2.18	2.18	1.30	1.09	0.90	0.90			

APPENDIX B: CASE STUDY 1 – STATIC LOAD TESTING-2

MAIN CONTRACTOR		CONSULTANT		FILE NO		DATE OF CAST		TOP LEVEL		DIAL GAUGE READINGS										REMARKS	
CONTRACTOR		HARRO GULF		P.T.P. 03																	
DATE	TIME	LOAD %	PRESSURE GAUGE READING kg	EQUIVALENT T LOAD (Ks)	A (mm)	B (mm)	C (mm)	D (mm)	GAUGE SETTLEMENT	AVERAGE	E (mm)	F (mm)	G (mm)	H (mm)	REMARKS						
18/09/08	14:00	0%	00	00	0.10	0.10	0.10	0.10	0.00	0.10	0.10	0.10	0.10	0.10							
"	14:00	25%	75	250	0.32	0.32	0.20	0.15	0.25	0.25	0.03	0.10	0.04	0.07							
"	14:10	"	"	"	0.32	0.32	0.20	0.15	0.25	0.25	0.03	0.10	0.04	0.07							
"	14:20	"	"	"	0.32	0.32	0.20	0.15	0.25	0.25	0.03	0.10	0.04	0.07							
"	14:30	"	"	"	0.32	0.32	0.20	0.15	0.25	0.25	0.03	0.10	0.04	0.07							
"	14:30	50%	150	500	0.90	0.67	0.54	0.54	0.66	0.66	0.10	0.23	0.04	0.11							
"	14:40	"	"	"	0.92	0.67	0.56	0.56	0.68	0.68	0.10	0.24	0.04	0.15							
"	14:50	"	"	"	0.94	0.67	0.57	0.57	0.69	0.69	0.10	0.24	0.04	0.17							
"	15:00	"	"	"	0.95	0.85	0.57	0.58	0.74	0.74	0.10	0.24	0.04	0.17							
"	15:00	75%	206	750	1.62	1.33	1.03	1.01	1.25	1.25	0.21	0.69	0.84	0.60							
"	15:00	"	"	"	1.67	1.33	1.09	1.06	1.29	1.29	0.23	0.72	0.84	0.68							
"	15:20	"	"	"	1.72	1.49	1.12	1.09	1.35	1.35	0.23	0.74	0.84	0.54							
"	15:30	"	"	"	1.73	1.49	1.13	1.09	1.36	1.36	0.23	0.74	0.84	0.54							
For consultant:		Date: 18-09-08		Date: 18/09/08		Date: 18/09/08		Date: 18/09/08		Date: 18/09/08		Date: 18/09/08		Date: 18/09/08							
Sign:		Sign:		Sign:		Sign:		Sign:		Sign:		Sign:		Sign:							

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FILE NO. _____ DATE OF CAST _____ P.T.P. 03														
TOP LEVEL _____														
DIAL GAUGE READINGS														
TIME	LOAD %	PRESSURE READING bar	EQUIVALENT LOAD/TONS (Kc)	DIAL GAUGE READINGS										
				A (mm)	B (mm)	C (mm)	D (mm)	QUA SETTLEMENT	AVERAGE	E (mm)	F (mm)	G (mm)	H (mm)	INCHES
20	100	301	1000	2.85	2.31	1.90	1.97		2.26	1.50	3.00	0.97	2.17	
00	"	"	"	2.95	2.48	1.98	2.04		2.36	1.70	3.07	0.97	2.30	
30	"	"	"	3.03	2.48	2.01	2.06		2.39	1.72	3.08	0.97	2.35	
00	"	"	"	3.05	2.48	2.02	2.07		2.40	1.81	3.30	0.97	2.40	
30	"	"	"	3.09	2.48	2.05	2.09		2.43	1.87	3.32	1.00	2.43	
00	"	"	"	3.10	2.48	2.06	2.10		2.43	1.88	3.33	1.00	2.44	
30	"	"	"	3.11	2.48	2.06	2.10		2.44	1.89	3.33	1.00	2.45	
00	"	"	"	3.11	2.48	2.07	2.10		2.44	1.90	3.33	1.00	2.45	
30	"	"	"	3.12	2.51	2.10	2.10		2.46	1.90	3.35	1.00	2.51	
00	"	"	"	3.12	2.51	2.11	2.10		2.46	1.90	3.35	1.00	2.51	
30	"	"	"	3.12	2.51	2.12	2.10		2.46	1.90	3.35	1.00	2.51	
00	"	"	"	3.13	2.51	2.12	2.10		2.46	1.90	3.35	1.00	2.51	
30	"	"	"	3.13	2.51	2.12	2.10		2.46	1.90	3.35	1.00	2.51	
00	"	"	"	3.13	2.51	2.12	2.10		2.46	1.90	3.35	1.00	2.51	
AL Hail & Al Muballadani Under: 18/09/08 Sign: _____														

FILE NO		DATE OF CAST		TOP LEVEL									
PT-2-23		12/2/03											
DIAL GUAGE READINGS													
LOAD %	PRESSURE GUAGE READING bar	EQUIVALEN T LOAD Tons (Kil)	A (mm)	B (mm)	C (mm)	D (mm)	GUAGE SETTLEMENT	AVERAGE	E (mm)	F (mm)	G (mm)	H (mm)	REMARKS
			2.78	2.18	1.77	1.76		2.12	1.70	2.78	0.85	2.12	
5%	226	250	2.77	2.18	1.77	1.76		2.12	1.70	2.78	0.85	2.12	
"	"	"	2.77	2.17	1.75	1.75		2.11	1.68	2.77	0.84	2.10	
20%	150	500	2.17	1.70	1.46	1.37		1.67	1.31	2.03	0.75	1.56	
"	"	"	2.17	1.70	1.46	1.37		1.67	1.30	2.03	0.74	1.55	
"	"	"	2.17	1.70	1.46	1.37		1.67	1.30	2.03	0.74	1.55	
5%	75	250	1.60	1.28	1.12	1.04		1.26	0.89	1.34	0.69	1.06	
"	"	"	1.59	1.26	1.12	1.04		1.25	0.88	1.33	0.69	1.05	
"	"	"	1.59	1.26	1.12	1.04		1.25	0.88	1.33	0.69	1.05	
20%	0.0	0.0	0.79	0.63	0.80	0.64		0.71	0.45	0.73	0.63	0.68	
"	"	"	0.75	0.60	0.78	0.61		0.68	0.43	0.69	0.62	0.66	
"	"	"	0.73	0.59	0.77	0.60		0.67	0.43	0.69	0.62	0.65	
"	"	"	0.72	0.58	0.77	0.60		0.67	0.43	0.69	0.62	0.65	
			All Hat & Al Mohaddineh Date: 12/09/03 Sign: <i>Al Hat & Al Mohaddineh</i>										

PILE NO. 1 DATE OF CAST 20.12.03													
TIDE LEVEL													
DIAL GAUGE READINGS													
LOAD %	PRESSURE GUAGE READING bar	EQUIVALENT LOAD TONS (Kil)	A (mm)	B (mm)	C (mm)	D (mm)	GUAGE SETTLEMENT	AVERAGE	E (mm)	F (mm)	G (mm)	H (mm)	REMARKS
50%	6.51	1500	5.34	4.02	4.80	3.73		4.47	6.20	6.31	2.37	4.98	
"	"	"	5.38	4.03	4.84	3.76		4.50	6.22	6.31	2.41	5.07	
"	"	"	5.40	4.05	4.90	3.77		4.53	6.24	6.31	2.41	5.08	
"	"	"	5.42	4.06	4.94	3.78		4.54	6.25	6.32	2.42	5.10	
"	"	"	5.43	4.07	4.96	3.79		4.55	6.25	6.33	2.44	5.13	
35%	3.76	1250	5.12	3.66	4.55	3.44		4.19	5.77	5.83	2.13	4.60	
"	"	"	5.12	3.66	4.54	3.44		4.19	5.75	5.63	2.12	4.58	
"	"	"	5.11	3.65	4.54	3.43		4.18	5.75	5.63	2.12	4.58	
20%	2.01	800	4.62	3.29	4.15	3.01		3.77	5.10	4.91	1.87	4.00	
"	"	"	4.61	3.28	4.14	3.00		3.76	5.08	4.90	1.86	3.98	
"	"	"	4.60	3.28	4.13	3.00		3.75	5.07	4.90	1.86	3.98	
"	"	"	4.59	3.28	4.12	2.99		3.75	5.07	4.90	1.86	3.98	
"	"	"	4.58	3.27	4.11	2.98		3.74	5.06	4.89	1.85	3.97	

At Hol & N. M

MAIN CONTRACTOR			CONSULTANT			CONTRACTOR			FILE NO.			DATE OF CAST			TOP LEVEL		
ARRAP GULF			ARRAP GULF			ARRAP GULF			P.E.P-02								
DATE	TIME	LOAD %	PRESSURE GAUGE READING bar	EQUIVALENT LOAD Tonn (Kca)	DIAL GAUGE READINGS										REMARKS		
					A (mm)	B (mm)	C (mm)	D (mm)	CHARGE SETTLEMENT	AVERAGE	E (mm)	F (mm)	G (mm)	H (mm)			
19/08/08	07:30	75%	2.26	250	4.04	2.82	3.69	2.35			3.27	4.21	3.85	1.65	3.25		
"	07:30	50%	1.50	500	3.39	2.35	3.24	2.11			2.77	3.19	2.90	1.55	2.46		
"	07:35	"	"	"	3.38	2.34	3.22	2.10			2.76	3.17	2.88	1.55	2.45		
"	07:40	"	"	"	3.37	2.34	3.21	2.10			2.75	3.15	2.87	1.55	2.45		
"	07:40	25%	75	250	2.65	1.80	2.68	1.62			2.19	2.20	1.97	1.28	1.81		
"	07:45	"	"	"	2.63	1.79	2.66	1.61			2.17	2.18	1.95	1.26	1.80		
"	07:50	"	"	"	2.62	1.78	2.64	1.60			2.16	2.15	1.94	1.26	1.80		
"	07:50	0%	0.0	0.0	2.05	1.19	2.13	1.15			1.63	1.45	1.30	1.19	1.40		
"	08:05	"	"	"	2.00	1.17	2.07	1.12			1.59	1.41	1.26	1.13	1.36		
"	08:10	"	"	"	1.96	1.14	2.04	1.10			1.56	1.37	1.21	1.09	1.31		
"	08:30	"	"	"	1.94	1.14	2.01	1.09			1.54	1.36	1.21	1.09	1.31		
"	08:50	"	"	"	1.82	1.13	1.99	1.08			1.53	1.35	1.21	1.08	1.30		
"	08:50	"	"	"	1.80	1.12	1.97	1.07			1.53	1.35	1.21	1.08	1.30		
For completion of Date: 19/09/08 Sign: [Signature]																	

MAIN CONTRACTOR		CONSULTANT		CONTRACTOR		FILE NO.		DATE OF CAST		TOP LEVEL						
BRICK P. G. W. F.		P. C. E. T. A.		P. C. E. T. A.		P. C. E. T. A.		P. C. E. T. A.		P. C. E. T. A.						
DATE	TIME	LOAD %	PRESSURE GUAGE READING bar	EQUIVALENT LOAD (KPa)	DIAL GUAGE READINGS											
					A (mm)	B (mm)	C (mm)	D (mm)	GUAGE SETTLEMENT	AVERAGE	E (mm)	F (mm)	G (mm)	H (mm)	REMARKS	
19/01/80	09:05	100%	301	1000	4.55	3.14	4.14	2.78			3.65	4.65	4.57	1.86	3.93	
"	09:10	"	"	"	4.57	3.16	4.15	2.79			3.67	4.66	4.56	1.86	3.94	
"	09:15	"	"	"	4.60	3.18	4.16	2.80			3.68	4.68	4.56	1.87	3.95	
"	09:20	"	"	"	4.60	3.18	4.16	2.80			3.68	4.68	4.56	1.87	3.95	
"	09:50	150%	451	1500	5.88	4.23	5.41	3.84			4.84	6.63	6.48	2.45	5.51	
"	10:05	"	"	"	5.96	4.29	5.48	3.89			4.90	6.80	6.53	2.46	5.56	
"	10:20	"	"	"	6.00	4.31	5.55	3.92			4.94	6.85	6.57	2.49	5.58	
"	10:35	"	"	"	6.01	4.31	5.58	3.94			4.96	6.88	6.59	2.50	5.59	
"	10:50	"	"	"	6.01	4.31	5.58	3.94			4.96	6.89	6.60	2.50	5.59	
"	10:55	150%	451	1500	6.76	5.01	6.35	4.57			5.87	8.00	7.65	2.67	6.47	
"	11:05	"	"	"	6.76	5.01	6.35	4.57			5.87	8.00	7.65	2.67	6.47	
"	11:20	"	"	"	6.87	5.23	6.64	4.71			5.83	8.15	7.84	2.70	6.55	
"	11:25	"	"	"	6.87	5.23	6.64	4.71			5.83	8.15	7.84	2.70	6.55	
"	11:35	"	"	"	6.87	5.23	6.64	4.71			5.90	8.28	7.85	2.70	6.57	
For completion:					At 11:35 & 11:50											
Date: 19.01.80					At 11:35 & 11:50											
Signature:					At 11:35 & 11:50											

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APPENDIX C: CASE STUDY 2 – OSTERBERG LOAD TESTING

BIDIRECTIONAL STATIC LOAD TEST AT

Dubai Mixed Use Development, Dubai, U.A.E

1,200 mm dia Bored Pile (Pile # PTP3)

Tested on 21st to 22nd July 2008 (1st Cycle)

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 03:00 PM	0	0	0	0.0	0.0	0.0
21/07/08 03:01 PM	920	333	167	0.0	0.0	0.2
21/07/08 03:02 PM	762	276	138	0.0	0.0	0.3
21/07/08 03:03 PM	753	273	136	0.0	0.0	0.3
21/07/08 03:04 PM	743	269	135	0.0	0.0	0.3
21/07/08 03:05 PM	746	270	135	0.0	0.0	0.3
21/07/08 03:06 PM	736	267	133	0.0	0.0	0.3
21/07/08 03:07 PM	739	268	134	0.0	0.0	0.3
21/07/08 03:08 PM	737	267	133	0.0	0.0	0.3
21/07/08 03:09 PM	737	267	133	0.0	0.0	0.3
21/07/08 03:10 PM	736	267	133	0.0	0.0	0.4
21/07/08 03:11 PM	737	267	133	0.0	0.0	0.4
21/07/08 03:12 PM	735	266	133	0.0	0.0	0.4
21/07/08 03:13 PM	732	265	133	0.0	0.0	0.4
21/07/08 03:14 PM	1064	385	193	0.0	0.0	1.2
21/07/08 03:15 PM	966	350	175	0.0	0.0	3.3
21/07/08 03:16 PM	1119	405	203	0.0	0.0	5.8
21/07/08 03:17 PM	880	319	159	0.0	0.0	8.3
21/07/08 03:18 PM	859	311	156	0.0	-0.1	11.1
21/07/08 03:19 PM	801	290	145	0.0	-0.1	13.9
21/07/08 03:20 PM	895	324	162	0.0	-0.1	16.5
21/07/08 03:21 PM	975	353	177	0.0	-0.2	19.0
21/07/08 03:22 PM	1056	382	191	0.0	-0.2	21.3
21/07/08 03:23 PM	1119	405	203	0.0	-0.2	23.3
21/07/08 03:24 PM	1179	427	214	0.0	-0.2	25.2
21/07/08 03:25 PM	1236	448	224	0.0	-0.3	26.9
21/07/08 03:26 PM	1216	440	220	0.0	-0.3	27.9
21/07/08 03:27 PM	1246	451	226	0.0	-0.3	28.8
21/07/08 03:28 PM	1301	471	236	0.0	-0.3	29.8
21/07/08 03:29 PM	1309	474	237	0.0	-0.3	30.6
21/07/08 03:30 PM	1321	478	239	-0.1	-0.3	31.3
21/07/08 03:31 PM	1336	484	242	-0.1	-0.3	31.9
21/07/08 03:32 PM	1326	480	240	0.0	-0.4	32.4
21/07/08 03:33 PM	1334	483	242	-0.1	-0.4	32.8
21/07/08 03:34 PM	1340	485	243	-0.1	-0.4	33.1
21/07/08 03:35 PM	1174	425	213	-0.1	-0.3	33.2
21/07/08 03:36 PM	1136	411	206	-0.1	-0.4	33.2
21/07/08 03:37 PM	1134	411	205	0.0	-0.3	33.2
21/07/08 03:38 PM	1131	410	205	0.0	-0.4	33.2
21/07/08 03:39 PM	1193	432	216	0.0	-0.4	33.3
21/07/08 03:40 PM	1219	442	221	-0.1	-0.4	33.3
21/07/08 03:41 PM	1227	444	222	-0.1	-0.4	33.4
21/07/08 03:42 PM	1230	446	223	0.0	-0.3	33.4
21/07/08 03:43 PM	1230	446	223	-0.1	-0.4	33.5
21/07/08 03:44 PM	1234	447	223	-0.1	-0.4	33.5
21/07/08 03:45 PM	1236	448	224	0.0	-0.3	33.6
21/07/08 03:46 PM	1241	449	225	-0.1	-0.4	33.6
21/07/08 03:47 PM	1016	368	184	-0.1	-0.4	33.7
21/07/08 03:48 PM	893	323	162	-0.1	-0.4	33.7
21/07/08 03:49 PM	806	292	146	-0.1	-0.4	33.6
21/07/08 03:50 PM	1258	456	228	-0.1	-0.4	33.6
21/07/08 03:51 PM	1484	538	269	-0.1	-0.4	35.3
21/07/08 03:52 PM	1575	570	285	-0.1	-0.4	36.8

Dubai Mixed Use Development, Dubai, U.A.E
Pile # PTP3

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Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 03:53 PM	1657	600	300	-0.1	-0.4	38.8
21/07/08 03:54 PM	1720	623	311	-0.1	-0.5	40.5
21/07/08 03:55 PM	1785	647	323	-0.1	-0.5	42.2
21/07/08 03:56 PM	1849	670	335	-0.1	-0.5	43.9
21/07/08 03:57 PM	1899	688	344	-0.1	-0.5	45.4
21/07/08 03:58 PM	1887	683	342	-0.1	-0.5	46.5
21/07/08 03:59 PM	1936	701	351	-0.1	-0.6	47.4
21/07/08 04:00 PM	1970	714	357	-0.1	-0.6	48.3
21/07/08 04:01 PM	1960	710	355	-0.1	-0.6	49.0
21/07/08 04:02 PM	1960	710	355	-0.1	-0.6	49.5
21/07/08 04:03 PM	1972	714	357	-0.1	-0.6	50.0
21/07/08 04:04 PM	1986	719	360	-0.1	-0.6	50.4
21/07/08 04:05 PM	1990	721	360	-0.1	-0.6	50.8
21/07/08 04:06 PM	2006	727	363	-0.1	-0.6	51.2
21/07/08 04:07 PM	2015	730	365	-0.1	-0.6	51.6
21/07/08 04:08 PM	1931	699	350	-0.1	-0.6	51.8
21/07/08 04:09 PM	1908	691	346	-0.1	-0.7	51.8
21/07/08 04:10 PM	1957	709	354	-0.1	-0.6	52.0
21/07/08 04:11 PM	1963	711	355	-0.1	-0.7	52.1
21/07/08 04:12 PM	1969	713	357	-0.2	-0.7	52.2
21/07/08 04:13 PM	1972	714	357	-0.1	-0.7	52.4
21/07/08 04:14 PM	1975	715	358	-0.1	-0.7	52.5
21/07/08 04:15 PM	1980	717	359	-0.2	-0.7	52.6
21/07/08 04:16 PM	1982	718	359	-0.2	-0.7	52.8
21/07/08 04:17 PM	1987	720	360	-0.2	-0.7	52.9
21/07/08 04:18 PM	1988	720	360	-0.2	-0.7	53.0
21/07/08 04:19 PM	1981	718	359	-0.2	-0.7	53.1
21/07/08 04:20 PM	1987	720	360	-0.2	-0.7	53.2
21/07/08 04:21 PM	1989	720	360	-0.2	-0.7	53.3
21/07/08 04:22 PM	1989	720	360	-0.2	-0.7	53.4
21/07/08 04:23 PM	1994	722	361	-0.2	-0.7	53.5
21/07/08 04:24 PM	1999	724	362	-0.2	-0.7	53.6
21/07/08 04:25 PM	2002	725	363	-0.2	-0.7	53.7
21/07/08 04:26 PM	2006	727	363	-0.2	-0.7	53.8
21/07/08 04:27 PM	2006	727	363	-0.2	-0.7	53.9
21/07/08 04:28 PM	2005	726	363	-0.2	-0.7	54.0
21/07/08 04:29 PM	2009	728	364	-0.2	-0.7	54.1
21/07/08 04:30 PM	2010	728	364	-0.2	-0.7	54.2
21/07/08 04:31 PM	2012	729	364	-0.2	-0.8	54.2
21/07/08 04:32 PM	2014	729	365	-0.2	-0.7	54.3
21/07/08 04:33 PM	2014	729	365	-0.2	-0.7	54.4
21/07/08 04:34 PM	2016	730	365	-0.2	-0.7	54.5
21/07/08 04:35 PM	2019	731	366	-0.2	-0.7	54.5
21/07/08 04:36 PM	2021	732	366	-0.2	-0.7	54.6
21/07/08 04:37 PM	2022	732	366	-0.2	-0.8	54.6
21/07/08 04:38 PM	2024	733	367	-0.2	-0.7	54.7
21/07/08 04:39 PM	2027	734	367	-0.2	-0.7	54.8
21/07/08 04:40 PM	2031	736	368	-0.2	-0.7	54.9
21/07/08 04:41 PM	2031	736	368	-0.2	-0.7	54.9
21/07/08 04:42 PM	2030	735	368	-0.2	-0.7	55.0
21/07/08 04:43 PM	2033	736	368	-0.2	-0.8	55.1
21/07/08 04:44 PM	2030	735	368	-0.2	-0.7	55.1
21/07/08 04:45 PM	2035	737	369	-0.2	-0.8	55.2
21/07/08 04:46 PM	2035	737	369	-0.2	-0.8	55.2
21/07/08 04:47 PM	2033	736	368	-0.2	-0.8	55.3
21/07/08 04:48 PM	2035	737	369	-0.2	-0.8	55.3
21/07/08 04:49 PM	2036	737	369	-0.2	-0.7	55.4
21/07/08 04:50 PM	2035	737	369	-0.2	-0.8	55.4
21/07/08 04:51 PM	2032	736	368	-0.2	-0.8	55.5

Dubai Mixed Use Development, Dubai, U.A.E
Pile # PTP3

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Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 04:52 PM	2036	737	369	-0.3	-0.8	55.5
21/07/08 04:53 PM	1993	722	361	-0.2	-0.8	55.5
21/07/08 04:54 PM	1642	595	297	-0.3	-0.8	55.6
21/07/08 04:55 PM	1470	532	266	-0.2	-0.8	55.5
21/07/08 04:56 PM	1358	492	246	-0.2	-0.8	55.5
21/07/08 04:57 PM	1265	458	229	-0.2	-0.7	55.6
21/07/08 04:58 PM	2170	786	393	-0.3	-0.8	55.6
21/07/08 04:59 PM	2426	879	439	-0.3	-0.9	57.8
21/07/08 05:00 PM	2532	917	459	-0.3	-0.9	59.9
21/07/08 05:01 PM	2536	919	459	-0.3	-0.9	61.4
21/07/08 05:02 PM	2572	932	466	-0.3	-0.9	62.4
21/07/08 05:03 PM	2572	932	466	-0.3	-1.0	63.1
21/07/08 05:04 PM	2580	934	467	-0.3	-1.0	63.7
21/07/08 05:05 PM	2601	942	471	-0.3	-1.0	64.2
21/07/08 05:06 PM	2612	946	473	-0.3	-1.0	64.7
21/07/08 05:07 PM	2628	952	476	-0.3	-1.0	65.2
21/07/08 05:08 PM	2642	957	478	-0.3	-1.0	65.6
21/07/08 05:09 PM	2626	951	476	-0.3	-1.0	66.0
21/07/08 05:10 PM	2595	940	470	-0.3	-1.0	66.2
21/07/08 05:11 PM	2592	939	469	-0.3	-1.0	66.3
21/07/08 05:12 PM	2588	937	469	-0.3	-1.0	66.5
21/07/08 05:13 PM	2593	939	470	-0.3	-1.0	66.6
21/07/08 05:14 PM	2594	940	470	-0.3	-1.0	66.8
21/07/08 05:15 PM	2595	940	470	-0.3	-1.0	66.9
21/07/08 05:16 PM	2598	941	470	-0.3	-1.0	67.1
21/07/08 05:17 PM	2602	942	471	-0.3	-1.1	67.2
21/07/08 05:18 PM	2599	941	471	-0.3	-1.0	67.3
21/07/08 05:19 PM	2599	941	471	-0.3	-1.0	67.4
21/07/08 05:20 PM	2602	942	471	-0.3	-1.0	67.5
21/07/08 05:21 PM	2605	944	472	-0.3	-1.0	67.6
21/07/08 05:22 PM	2606	944	472	-0.3	-1.0	67.7
21/07/08 05:23 PM	2612	946	473	-0.3	-1.0	67.8
21/07/08 05:24 PM	2612	946	473	-0.3	-1.0	67.9
21/07/08 05:25 PM	2617	948	474	-0.4	-1.1	68.0
21/07/08 05:26 PM	2620	949	474	-0.4	-1.1	68.1
21/07/08 05:27 PM	2618	948	474	-0.4	-1.1	68.2
21/07/08 05:28 PM	2616	948	474	-0.4	-1.1	68.3
21/07/08 05:29 PM	2610	945	473	-0.4	-1.1	68.3
21/07/08 05:30 PM	2614	947	473	-0.4	-1.1	68.4
21/07/08 05:31 PM	2616	948	474	-0.4	-1.1	68.5
21/07/08 05:32 PM	2614	947	473	-0.3	-1.1	68.6
21/07/08 05:33 PM	2618	948	474	-0.3	-1.1	68.6
21/07/08 05:34 PM	2621	949	475	-0.4	-1.1	68.7
21/07/08 05:35 PM	2623	950	475	-0.4	-1.1	68.8
21/07/08 05:36 PM	2625	951	475	-0.4	-1.1	68.8
21/07/08 05:37 PM	2626	951	476	-0.4	-1.1	68.9
21/07/08 05:38 PM	2625	951	475	-0.3	-1.1	68.9
21/07/08 05:39 PM	2627	951	476	-0.4	-1.1	69.0
21/07/08 05:40 PM	2624	950	475	-0.3	-1.1	69.1
21/07/08 05:41 PM	2626	951	476	-0.4	-1.1	69.1
21/07/08 05:42 PM	2629	952	476	-0.3	-1.1	69.2
21/07/08 05:43 PM	2628	952	476	-0.4	-1.1	69.2
21/07/08 05:44 PM	2625	951	475	-0.3	-1.1	69.3
21/07/08 05:45 PM	2625	951	475	-0.4	-1.1	69.3
21/07/08 05:46 PM	2622	950	475	-0.4	-1.1	69.3
21/07/08 05:47 PM	2620	949	474	-0.4	-1.1	69.4
21/07/08 05:48 PM	2622	950	475	-0.4	-1.1	69.4
21/07/08 05:49 PM	2624	950	475	-0.4	-1.1	69.5
21/07/08 05:50 PM	2625	951	475	-0.4	-1.1	69.5

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				File Top	Cell Top	Cell Bottom
21/07/08 05:51 PM	2626	951	476	-0.4	-1.1	69.5
21/07/08 05:52 PM	2624	950	475	-0.4	-1.1	69.6
21/07/08 05:53 PM	2627	951	476	-0.4	-1.1	69.6
21/07/08 05:54 PM	2633	954	477	-0.4	-1.1	69.6
21/07/08 05:55 PM	2636	955	477	-0.4	-1.1	69.7
21/07/08 05:56 PM	2639	956	478	-0.4	-1.1	69.8
21/07/08 05:57 PM	2640	956	478	-0.4	-1.1	69.8
21/07/08 05:58 PM	2646	958	479	-0.4	-1.1	69.8
21/07/08 05:59 PM	2644	958	479	-0.4	-1.1	69.9
21/07/08 06:00 PM	2642	957	478	-0.4	-1.1	69.9
21/07/08 06:01 PM	2642	957	478	-0.4	-1.1	70.0
21/07/08 06:02 PM	2642	957	478	-0.4	-1.1	70.0
21/07/08 06:03 PM	2638	955	478	-0.4	-1.1	70.0
21/07/08 06:04 PM	2640	956	478	-0.4	-1.1	70.1
21/07/08 06:05 PM	2640	956	478	-0.4	-1.1	70.1
21/07/08 06:06 PM	2642	957	478	-0.4	-1.1	70.1
21/07/08 06:07 PM	2642	957	478	-0.4	-1.1	70.1
21/07/08 06:08 PM	2647	959	479	-0.4	-1.1	70.2
21/07/08 06:09 PM	2647	959	479	-0.4	-1.1	70.2
21/07/08 06:10 PM	2649	959	480	-0.4	-1.1	70.3
21/07/08 06:11 PM	2649	959	480	-0.4	-1.1	70.3
21/07/08 06:12 PM	2646	958	479	-0.4	-1.1	70.3
21/07/08 06:13 PM	2645	958	479	-0.4	-1.1	70.3
21/07/08 06:14 PM	2647	959	479	-0.4	-1.1	70.4
21/07/08 06:15 PM	2649	959	480	-0.4	-1.1	70.4
21/07/08 06:16 PM	2651	960	480	-0.4	-1.1	70.4
21/07/08 06:17 PM	2650	960	480	-0.4	-1.1	70.5
21/07/08 06:18 PM	2651	960	480	-0.4	-1.1	70.5
21/07/08 06:19 PM	2651	960	480	-0.4	-1.1	70.5
21/07/08 06:20 PM	2650	960	480	-0.4	-1.1	70.5
21/07/08 06:21 PM	2652	961	480	-0.4	-1.1	70.6
21/07/08 06:22 PM	2652	961	480	-0.4	-1.1	70.6
21/07/08 06:23 PM	2652	961	480	-0.4	-1.1	70.6
21/07/08 06:24 PM	2651	960	480	-0.4	-1.1	70.6
21/07/08 06:25 PM	2648	959	480	-0.4	-1.1	70.7
21/07/08 06:26 PM	2647	959	479	-0.4	-1.1	70.7
21/07/08 06:27 PM	2650	960	480	-0.4	-1.1	70.7
21/07/08 06:28 PM	2653	961	480	-0.4	-1.1	70.8
21/07/08 06:29 PM	2651	960	480	-0.4	-1.1	70.7
21/07/08 06:30 PM	2515	911	455	-0.4	-1.1	70.8
21/07/08 06:31 PM	2182	790	395	-0.4	-1.1	70.8
21/07/08 06:32 PM	1928	698	349	-0.4	-1.1	70.8
21/07/08 06:33 PM	2399	869	434	-0.4	-1.1	70.8
21/07/08 06:34 PM	2650	960	480	-0.4	-1.1	70.8
21/07/08 06:35 PM	2674	969	484	-0.4	-1.1	70.8
21/07/08 06:36 PM	2669	967	483	-0.4	-1.1	70.8
21/07/08 06:37 PM	2661	964	482	-0.4	-1.1	70.9
21/07/08 06:38 PM	2663	965	482	-0.4	-1.1	70.8
21/07/08 06:39 PM	2666	966	483	-0.4	-1.1	70.9
21/07/08 06:40 PM	2665	965	483	-0.4	-1.1	70.9
21/07/08 06:41 PM	2665	965	483	-0.4	-1.1	70.9
21/07/08 06:42 PM	2665	965	483	-0.4	-1.1	70.9
21/07/08 06:43 PM	2665	965	483	-0.4	-1.1	70.9
21/07/08 06:44 PM	2666	966	483	-0.4	-1.1	70.9
21/07/08 06:45 PM	2665	965	483	-0.4	-1.1	71.0
21/07/08 06:46 PM	2665	965	483	-0.4	-1.1	71.0
21/07/08 06:47 PM	2665	965	483	-0.4	-1.1	71.0
21/07/08 06:48 PM	2662	964	482	-0.4	-1.1	71.0
21/07/08 06:49 PM	2656	962	481	-0.4	-1.1	71.0

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 06:50 PM	2654	961	481	-0.4	-1.1	71.0
21/07/08 06:51 PM	2555	925	463	-0.4	-1.2	71.0
21/07/08 06:52 PM	3061	1109	554	-0.4	-1.2	72.1
21/07/08 06:53 PM	3219	1166	583	-0.4	-1.2	73.6
21/07/08 06:54 PM	3238	1173	586	-0.4	-1.2	75.0
21/07/08 06:55 PM	3294	1193	597	-0.4	-1.3	75.9
21/07/08 06:56 PM	3242	1174	587	-0.4	-1.3	76.4
21/07/08 06:57 PM	3249	1177	588	-0.4	-1.3	76.7
21/07/08 06:58 PM	3256	1179	590	-0.4	-1.3	77.1
21/07/08 06:59 PM	3268	1184	592	-0.4	-1.3	77.4
21/07/08 07:00 PM	3280	1188	594	-0.4	-1.3	77.7
21/07/08 07:01 PM	3288	1191	595	-0.4	-1.3	77.9
21/07/08 07:02 PM	3295	1193	597	-0.4	-1.3	78.2
21/07/08 07:03 PM	3300	1195	598	-0.4	-1.3	78.4
21/07/08 07:04 PM	3305	1197	599	-0.4	-1.3	78.6
21/07/08 07:05 PM	3307	1198	599	-0.4	-1.3	78.8
21/07/08 07:06 PM	3314	1200	600	-0.4	-1.3	79.0

BIDIRECTIONAL STATIC LOAD TEST AT
Dubai Mixed Use Development, Dubai, U.A.E
1,200 mm dia Bored Pile (Pile # PTP3)
Tested on 21st to 22nd July 2008 (2nd Cycle)

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 07:37 PM	0	0	0	0.0	0.0	0.0
21/07/08 07:38 PM	240	87	43	0.0	0.0	0.0
21/07/08 07:39 PM	280	101	50	0.0	0.0	0.0
21/07/08 07:40 PM	584	211	105	0.0	0.0	0.0
21/07/08 07:41 PM	639	230	115	0.0	0.0	0.0
21/07/08 07:42 PM	636	229	115	0.0	0.0	0.0
21/07/08 07:43 PM	1172	423	211	0.0	-0.1	0.0
21/07/08 07:44 PM	997	360	180	0.0	-0.1	0.0
21/07/08 07:45 PM	983	354	177	0.0	-0.1	0.0
21/07/08 07:46 PM	980	353	177	0.0	-0.1	0.0
21/07/08 07:47 PM	980	353	177	-0.1	-0.1	-0.1
21/07/08 07:48 PM	976	352	176	0.0	-0.1	0.0
21/07/08 07:49 PM	976	352	176	0.0	-0.1	0.0
21/07/08 07:50 PM	977	352	176	0.0	-0.1	0.0
21/07/08 07:51 PM	976	352	176	-0.1	-0.1	0.0
21/07/08 07:52 PM	978	353	176	0.0	-0.1	0.0
21/07/08 07:53 PM	978	353	176	0.0	-0.1	0.0
21/07/08 07:54 PM	1978	713	357	-0.4	-0.6	0.8
21/07/08 07:55 PM	2084	751	376	-0.9	-1.2	1.5
21/07/08 07:56 PM	2056	741	371	-0.9	-1.3	1.6
21/07/08 07:57 PM	2051	740	370	-1.0	-1.3	1.6
21/07/08 07:58 PM	2051	740	370	-1.1	-1.4	1.6
21/07/08 07:59 PM	2049	739	369	-1.2	-1.5	1.6
21/07/08 08:00 PM	2051	740	370	-1.2	-1.6	1.6
21/07/08 08:01 PM	2055	741	371	-1.3	-1.6	1.6
21/07/08 08:02 PM	2054	741	370	-1.4	-1.7	1.6
21/07/08 08:03 PM	2058	742	371	-1.4	-1.8	1.6
21/07/08 08:04 PM	2062	744	372	-1.4	-1.8	1.7
21/07/08 08:05 PM	2064	744	372	-1.5	-1.8	1.7
21/07/08 08:06 PM	2069	746	373	-1.5	-1.9	1.7
21/07/08 08:07 PM	2064	744	372	-1.6	-2.0	1.6
21/07/08 08:08 PM	2068*	746	373	-1.6	-2.0	1.7
21/07/08 08:09 PM	2070	746	373	-1.7	-2.1	1.6
21/07/08 08:10 PM	2074	748	374	-1.7	-2.1	1.7
21/07/08 08:11 PM	2072	747	374	-1.8	-2.1	1.7
21/07/08 08:12 PM	2073	748	374	-1.8	-2.1	1.7
21/07/08 08:13 PM	2071	747	373	-1.8	-2.2	1.7
21/07/08 08:14 PM	2072	747	374	-1.9	-2.2	1.7
21/07/08 08:15 PM	2074	748	374	-1.9	-2.2	1.7
21/07/08 08:16 PM	2668	962	481	-3.0	-3.5	1.8
21/07/08 08:17 PM	3062	1104	552	-4.4	-5.0	1.9
21/07/08 08:18 PM	3165	1141	571	-5.4	-6.0	1.9
21/07/08 08:19 PM	3238	1168	584	-6.1	-6.7	2.0
21/07/08 08:20 PM	3302	1191	595	-6.6	-7.2	2.0
21/07/08 08:21 PM	3343	1205	603	-6.8	-7.5	2.1
21/07/08 08:22 PM	3362	1212	606	-7.1	-7.8	2.1
21/07/08 08:23 PM	3379	1218	609	-7.3	-8.0	2.1
21/07/08 08:24 PM	3385	1221	610	-7.4	-8.1	2.1
21/07/08 08:25 PM	3394	1224	612	-7.5	-8.2	2.1

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 08:26 PM	3393	1224	612	-7.7	-8.4	2.1
21/07/08 08:27 PM	3396	1225	612	-7.7	-8.4	2.2
21/07/08 08:28 PM	3408	1229	614	-7.8	-8.6	2.1
21/07/08 08:29 PM	3410	1230	615	-7.9	-8.6	2.2
21/07/08 08:30 PM	3416	1232	616	-8.0	-8.7	2.2
21/07/08 08:31 PM	3417	1232	616	-8.1	-8.8	2.2
21/07/08 08:32 PM	3417	1232	616	-8.1	-8.9	2.2
21/07/08 08:33 PM	3418	1233	616	-8.2	-8.9	2.2
21/07/08 08:34 PM	3430	1237	618	-8.2	-9.0	2.2
21/07/08 08:35 PM	3434	1238	619	-8.3	-9.0	2.2
21/07/08 08:36 PM	3435	1239	619	-8.4	-9.1	2.2
21/07/08 08:37 PM	3447	1243	621	-8.4	-9.2	2.2
21/07/08 08:38 PM	3830	1381	691	-9.0	-9.8	2.3
21/07/08 08:39 PM	3925	1415	708	-9.5	-10.3	2.3
21/07/08 08:40 PM	3969	1431	716	-9.8	-10.7	2.3
21/07/08 08:41 PM	4002	1443	722	-10.1	-10.9	2.3
21/07/08 08:42 PM	3983	1436	718	-10.2	-11.1	2.3
21/07/08 08:43 PM	3982	1436	718	-10.3	-11.2	2.4
21/07/08 08:44 PM	3982	1436	718	-10.4	-11.3	2.4
21/07/08 08:45 PM	3986	1437	719	-10.5	-11.4	2.4
21/07/08 08:46 PM	3983	1436	718	-10.6	-11.5	2.4
21/07/08 08:47 PM	3993	1440	720	-10.7	-11.6	2.4
21/07/08 08:48 PM	4019	1449	725	-10.7	-11.6	2.4
21/07/08 08:49 PM	4026	1452	726	-10.7	-11.7	2.4
21/07/08 08:50 PM	4029	1453	726	-10.8	-11.7	2.4
21/07/08 08:51 PM	4026	1452	726	-10.8	-11.7	2.4
21/07/08 08:52 PM	4027	1452	726	-10.8	-11.7	2.4
21/07/08 08:53 PM	4035	1455	728	-10.8	-11.7	2.4
21/07/08 08:54 PM	4040	1457	728	-10.9	-11.8	2.4
21/07/08 08:55 PM	4040	1457	728	-10.9	-11.8	2.4
21/07/08 08:56 PM	4035	1455	728	-10.9	-11.8	2.4
21/07/08 08:57 PM	4036	1455	728	-10.9	-11.8	2.4
21/07/08 08:58 PM	4031	1454	727	-10.9	-11.8	2.4
21/07/08 08:59 PM	4039	1456	728	-10.9	-11.9	2.4
21/07/08 09:00 PM	4032	1454	727	-10.9	-11.8	2.4
21/07/08 09:01 PM	4042	1458	729	-10.9	-11.8	2.4
21/07/08 09:02 PM	4041	1457	729	-10.9	-11.9	2.4
21/07/08 09:03 PM	4023	1451	725	-10.9	-11.9	2.4
21/07/08 09:04 PM	4027	1452	726	-10.9	-11.9	2.4
21/07/08 09:05 PM	3474	1253	626	-11.0	-11.9	2.4
21/07/08 09:06 PM	3959	1428	714	-11.0	-11.9	2.5
21/07/08 09:07 PM	4012	1447	723	-11.1	-12.1	2.5
21/07/08 09:08 PM	4016	1448	724	-11.2	-12.2	2.5
21/07/08 09:09 PM	4021	1450	725	-11.3	-12.2	2.5
21/07/08 09:10 PM	4022	1450	725	-11.3	-12.3	2.5
21/07/08 09:11 PM	4027	1452	726	-11.4	-12.3	2.5
21/07/08 09:12 PM	4033	1454	727	-11.4	-12.4	2.5
21/07/08 09:13 PM	4036	1455	728	-11.4	-12.4	2.5
21/07/08 09:14 PM	4041	1457	729	-11.5	-12.5	2.5
21/07/08 09:15 PM	4043	1458	729	-11.5	-12.5	2.5
21/07/08 09:16 PM	4038	1456	728	-11.6	-12.5	2.5
21/07/08 09:17 PM	4034	1455	727	-11.6	-12.5	2.5
21/07/08 09:18 PM	4024	1451	726	-11.6	-12.6	2.5
21/07/08 09:19 PM	4023	1451	725	-11.6	-12.6	2.5
21/07/08 09:20 PM	4022	1450	725	-11.6	-12.6	2.5

Dubai Mixed Use Development, Dubai, U.A.E
 DT., 4 0702

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 09:21 PM	4020	1450	755	-11.7	-12.6	2.5
21/07/08 09:22 PM	4018	1449	754	-11.7	-12.6	2.5
21/07/08 09:23 PM	4017	1449	754	-11.7	-12.7	2.5
21/07/08 09:24 PM	4014	1447	754	-11.7	-12.7	2.5
21/07/08 09:25 PM	4017	1449	754	-11.7	-12.7	2.5
21/07/08 09:26 PM	4019	1449	755	-11.8	-12.7	2.5
21/07/08 09:27 PM	4022	1450	755	-11.8	-12.7	2.5
21/07/08 09:28 PM	4024	1451	756	-11.8	-12.8	2.5
21/07/08 09:29 PM	4024	1451	756	-11.8	-12.8	2.5
21/07/08 09:30 PM	4020	1450	755	-11.8	-12.8	2.5
21/07/08 09:31 PM	4022	1450	755	-11.8	-12.8	2.5
21/07/08 09:32 PM	4032	1454	757	-11.8	-12.8	2.5
21/07/08 09:33 PM	4042	1458	759	-11.9	-12.8	2.5
21/07/08 09:34 PM	4046	1459	759	-11.9	-12.9	2.5
21/07/08 09:35 PM	4042	1458	759	-11.9	-12.9	2.5
21/07/08 09:36 PM	4026	1452	756	-11.9	-12.9	2.5
21/07/08 09:37 PM	4024	1451	756	-11.9	-12.9	2.5
21/07/08 09:38 PM	4024	1451	756	-11.9	-12.9	2.5
21/07/08 09:39 PM	4019	1449	755	-12.0	-13.0	2.5
21/07/08 09:40 PM	4026	1452	756	-12.0	-12.9	2.5
21/07/08 09:41 PM	4023	1451	755	-12.0	-13.0	2.5
21/07/08 09:42 PM	4023	1451	755	-12.0	-13.0	2.5
21/07/08 09:43 PM	4023	1451	755	-12.0	-13.0	2.5
21/07/08 09:44 PM	4022	1450	755	-12.0	-13.0	2.5
21/07/08 09:45 PM	4026	1452	756	-12.0	-13.0	2.5
21/07/08 09:46 PM	4026	1452	756	-12.0	-13.0	2.5
21/07/08 09:47 PM	4030	1453	757	-12.0	-13.0	2.5
21/07/08 09:48 PM	4025	1451	756	-12.0	-13.0	2.5
21/07/08 09:49 PM	4032	1454	757	-12.1	-13.0	2.5
21/07/08 09:50 PM	4038	1456	758	-12.0	-13.0	2.5
21/07/08 09:51 PM	4040	1457	758	-12.0	-13.0	2.5
21/07/08 09:52 PM	4041	1457	759	-12.1	-13.1	2.5
21/07/08 09:53 PM	4039	1456	758	-12.1	-13.1	2.5
21/07/08 09:54 PM	4029	1453	756	-12.1	-13.1	2.5
21/07/08 09:55 PM	4033	1454	757	-12.1	-13.1	2.5
21/07/08 09:56 PM	4033	1454	757	-12.1	-13.1	2.5
21/07/08 09:57 PM	4030	1453	757	-12.1	-13.1	2.5
21/07/08 09:58 PM	4028	1452	756	-12.1	-13.1	2.5
21/07/08 09:59 PM	4026	1452	756	-12.1	-13.1	2.5
21/07/08 10:00 PM	4024	1451	756	-12.1	-13.1	2.5
21/07/08 10:01 PM	4023	1451	755	-12.1	-13.1	2.5
21/07/08 10:02 PM	4022	1450	755	-12.1	-13.1	2.5
21/07/08 10:03 PM	4021	1450	755	-12.1	-13.0	2.6
21/07/08 10:04 PM	4022	1450	755	-12.1	-13.0	2.5
21/07/08 10:05 PM	4019	1449	755	-12.1	-13.0	2.5
21/07/08 10:06 PM	4016	1448	754	-12.1	-13.0	2.5
21/07/08 10:07 PM	4014	1447	754	-12.1	-12.9	2.5
21/07/08 10:08 PM	4016	1448	754	-12.1	-12.9	2.5
21/07/08 10:09 PM	4014	1447	754	-12.1	-12.9	2.4
21/07/08 10:10 PM	4013	1447	754	-12.1	-12.9	2.4
21/07/08 10:11 PM	4012	1447	753	-12.1	-12.8	2.4
21/07/08 10:12 PM	4011	1446	753	-12.0	-12.8	2.4
21/07/08 10:13 PM	4011	1446	753	-12.0	-12.8	2.3
21/07/08 10:14 PM	4009	1446	753	-12.0	-12.7	2.4
21/07/08 10:15 PM	4009	1446	753	-12.0	-12.7	2.3

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 10:16 PM	4714	1700	850	-12.9	-14.2	2.6
21/07/08 10:17 PM	5247	1892	946	-14.3	-15.7	2.7
21/07/08 10:18 PM	5092	1836	918	-15.2	-16.6	2.8
21/07/08 10:19 PM	5067	1827	914	-15.3	-16.8	2.8
21/07/08 10:20 PM	5064	1826	913	-15.3	-16.8	2.9
21/07/08 10:21 PM	5070	1828	914	-15.4	-16.9	2.8
21/07/08 10:22 PM	5066	1827	913	-15.5	-17.0	2.9
21/07/08 10:23 PM	4887	1762	881	-15.6	-17.0	2.9
21/07/08 10:24 PM	4953	1786	893	-15.5	-17.0	2.9
21/07/08 10:25 PM	4950	1785	892	-15.6	-17.1	2.9
21/07/08 10:26 PM	4950	1785	892	-15.6	-17.1	2.9
21/07/08 10:27 PM	4949	1785	892	-15.6	-17.1	2.9
21/07/08 10:28 PM	4949	1785	892	-15.6	-17.1	2.9
21/07/08 10:29 PM	4949	1785	892	-15.6	-17.1	2.9
21/07/08 10:30 PM	4950	1785	892	-15.6	-17.0	2.9
21/07/08 10:31 PM	5156	1859	990	-15.7	-17.1	2.9
21/07/08 10:32 PM	5794	2089	1045	-16.9	-18.5	3.0
21/07/08 10:33 PM	6043	2179	1090	-17.9	-19.6	3.0
21/07/08 10:34 PM	6150	2218	1109	-18.6	-20.4	3.1
21/07/08 10:35 PM	6129	2210	1105	-18.9	-20.7	3.1
21/07/08 10:36 PM	6113	2204	1102	-18.9	-20.7	3.2
21/07/08 10:37 PM	6099	2199	1100	-18.9	-20.7	3.1
21/07/08 10:38 PM	6088	2195	1098	-18.9	-20.7	3.1
21/07/08 10:39 PM	6077	2191	1096	-18.9	-20.6	3.2
21/07/08 10:40 PM	6068	2188	1094	-18.9	-20.6	3.1
21/07/08 10:41 PM	6060	2185	1093	-18.9	-20.5	3.1
21/07/08 10:42 PM	6052	2182	1091	-18.9	-20.4	3.1
21/07/08 10:43 PM	6044	2179	1090	-18.9	-20.4	3.0
21/07/08 10:44 PM	6038	2177	1089	-18.9	-20.3	3.0
21/07/08 10:45 PM	6030	2174	1087	-19.9	-21.9	3.2
21/07/08 10:46 PM	6972	2514	1257	-21.5	-23.6	3.3
21/07/08 10:47 PM	7139	2574	1287	-22.7	-24.9	3.4
21/07/08 10:48 PM	6573	2370	1185	-23.1	-25.3	3.5
21/07/08 10:49 PM	7097	2559	1280	-23.6	-25.8	3.6
21/07/08 10:50 PM	6486	2339	1169	-23.7	-25.9	3.6
21/07/08 10:51 PM	7055	2544	1272	-24.1	-26.3	3.6
21/07/08 10:52 PM	7040	2539	1269	-24.2	-26.4	3.6
21/07/08 10:53 PM	7030	2535	1268	-24.2	-26.4	3.6
21/07/08 10:54 PM	7025	2533	1267	-24.2	-26.5	3.5
21/07/08 10:55 PM	7020	2531	1266	-24.2	-26.4	3.6
21/07/08 10:56 PM	7015	2530	1265	-24.2	-26.4	3.5
21/07/08 10:57 PM	7012	2529	1264	-24.2	-26.4	3.6
21/07/08 10:58 PM	7006	2526	1263	-24.2	-26.3	3.6
21/07/08 10:59 PM	6999	2524	1262	-24.2	-26.3	3.5
21/07/08 11:00 PM	6999	2524	1262	-24.2	-26.2	3.5
21/07/08 11:01 PM	6768	2441	1220	-24.3	-26.4	3.5
21/07/08 11:02 PM	7521	2712	1356	-25.3	-27.7	3.6
21/07/08 11:03 PM	7807	2815	1408	-26.6	-29.1	3.7
21/07/08 11:04 PM	8001	2885	1443	-27.8	-30.3	3.8
21/07/08 11:05 PM	8043	2900	1450	-28.6	-31.2	3.9
21/07/08 11:06 PM	7961	2871	1435	-29.3	-31.9	3.9
21/07/08 11:07 PM	7207	2599	1299	-29.3	-32.0	3.9
21/07/08 11:08 PM	8045	2901	1451	-29.5	-32.2	3.9
21/07/08 11:09 PM	8093	2918	1459	-30.0	-32.6	4.0
21/07/08 11:10 PM	8077	2913	1456	-30.1	-32.7	3.9

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Pile # PTP3

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Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
21/07/08 11:11 PM	8064	2908	1454	-30.1	-32.7	3.9
21/07/08 11:12 PM	8054	2904	1452	-30.1	-32.7	3.9
21/07/08 11:13 PM	8044	2901	1450	-30.1	-32.7	3.9
21/07/08 11:14 PM	8038	2899	1449	-30.1	-32.6	3.9
21/07/08 11:15 PM	8033	2897	1448	-30.1	-32.5	3.9
21/07/08 11:16 PM	8031	2896	1448	-30.1	-32.5	3.9
21/07/08 11:17 PM	8028	2895	1447	-30.1	-32.4	3.8
21/07/08 11:18 PM	8023	2893	1447	-30.1	-32.6	3.9
21/07/08 11:19 PM	8356	3013	1507	-30.7	-33.4	4.0
21/07/08 11:20 PM	8617	3107	1554	-31.9	-34.7	4.1
21/07/08 11:21 PM	8759	3158	1579	-32.9	-35.8	4.1
21/07/08 11:22 PM	8990	3242	1621	-34.0	-36.9	4.2
21/07/08 11:23 PM	9148	3299	1649	-35.0	-38.1	4.3
21/07/08 11:24 PM	9128	3292	1646	-35.5	-38.5	4.3
21/07/08 11:25 PM	9116	3287	1644	-35.5	-38.5	4.3
21/07/08 11:26 PM	9103	3283	1641	-35.5	-38.5	4.3
21/07/08 11:27 PM	9092	3279	1639	-35.5	-38.4	4.2
21/07/08 11:28 PM	9082	3275	1637	-35.5	-38.3	4.2
21/07/08 11:29 PM	9070	3271	1635	-35.5	-38.2	4.2
21/07/08 11:30 PM	9058	3266	1633	-35.5	-38.1	4.1
21/07/08 11:31 PM	9047	3262	1631	-35.5	-38.1	4.1
21/07/08 11:32 PM	9036	3258	1629	-35.5	-38.0	4.0
21/07/08 11:33 PM	9027	3255	1628	-35.5	-38.1	4.0
21/07/08 11:34 PM	9017	3252	1626	-35.6	-38.6	4.2
21/07/08 11:35 PM	9280	3346	1673	-36.6	-39.7	4.4
21/07/08 11:36 PM	9444	3406	1703	-37.7	-40.9	4.4
21/07/08 11:37 PM	9621	3469	1735	-38.6	-41.9	4.5
21/07/08 11:38 PM	9816	3540	1770	-39.7	-43.1	4.6
21/07/08 11:39 PM	9962	3592	1796	-40.8	-44.3	4.6
21/07/08 11:40 PM	10050	3624	1812	-41.8	-45.3	4.7
21/07/08 11:41 PM	10112	3646	1823	-42.6	-46.1	4.7
21/07/08 11:42 PM	10182	3672	1836	-43.3	-46.9	4.7
21/07/08 11:43 PM	10112	3646	1823	-43.8	-47.4	4.8
21/07/08 11:44 PM	10050	3624	1812	-44.1	-47.7	4.8
21/07/08 11:45 PM	10051	3624	1812	-44.4	-48.0	4.8
21/07/08 11:46 PM	10072	3632	1816	-44.6	-48.2	4.8
21/07/08 11:47 PM	10079	3634	1817	-44.8	-48.4	4.8
21/07/08 11:48 PM	10173	3668	1834	-45.0	-48.7	4.8
21/07/08 11:49 PM	10241	3693	1846	-45.4	-49.0	4.8
21/07/08 11:50 PM	10237	3691	1846	-45.6	-49.3	4.8
21/07/08 11:51 PM	10228	3688	1844	-45.6	-49.3	4.8
21/07/08 11:52 PM	10219	3685	1842	-45.6	-49.3	4.8
21/07/08 11:53 PM	10213	3683	1841	-45.6	-49.2	4.8
21/07/08 11:54 PM	10204	3680	1840	-45.6	-49.0	4.8
21/07/08 11:55 PM	10196	3677	1838	-45.6	-49.6	4.8
21/07/08 11:56 PM	10187	3673	1837	-45.6	-49.6	4.8
21/07/08 11:57 PM	10180	3671	1835	-45.6	-49.6	4.8
21/07/08 11:58 PM	10172	3668	1834	-45.6	-49.6	4.8
21/07/08 11:59 PM	10165	3665	1833	-45.6	-49.6	4.8
22/07/08 12:00 AM	10158	3663	1831	-45.6	-49.6	4.8
22/07/08 12:01 AM	10152	3661	1830	-45.6	-49.6	4.8
22/07/08 12:02 AM	10145	3658	1829	-45.6	-49.9	4.8
22/07/08 12:03 AM	10139	3656	1828	-45.6	-49.9	4.8
22/07/08 12:04 AM	10131	3653	1827	-45.6	-49.9	4.8
22/07/08 12:05 AM	10125	3651	1826	-45.6	-49.9	4.8

Dubai Mixed Use Development, Dubai, U.A.E

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				File Top	Cell Top	Cell Bottom
22/07/08 12:06 AM	10119	3649	1824	-45.6	-49.9	4.8
22/07/08 12:07 AM	10112	3646	1823	-45.6	-50.3	4.8
22/07/08 12:08 AM	10107	3645	1822	-45.6	-50.3	4.8
22/07/08 12:09 AM	10102	3643	1821	-45.6	-50.3	4.8
22/07/08 12:10 AM	10096	3641	1820	-45.6	-50.3	4.8
22/07/08 12:11 AM	10091	3639	1819	-45.6	-50.3	4.8
22/07/08 12:12 AM	10087	3637	1819	-45.6	-50.3	4.8
22/07/08 12:13 AM	10082	3636	1818	-45.6	-50.3	4.8
22/07/08 12:14 AM	10077	3634	1817	-45.6	-50.3	4.8
22/07/08 12:15 AM	10072	3632	1816	-45.6	-50.3	4.8
22/07/08 12:16 AM	10068	3631	1815	-45.6	-50.6	4.8
22/07/08 12:17 AM	10064	3629	1815	-45.6	-50.6	4.8
22/07/08 12:18 AM	10060	3628	1814	-45.6	-50.6	4.8
22/07/08 12:19 AM	10056	3626	1813	-45.6	-50.6	4.8
22/07/08 12:20 AM	10052	3625	1812	-45.6	-50.6	4.8
22/07/08 12:21 AM	10050	3624	1812	-45.6	-50.6	4.8
22/07/08 12:22 AM	10045	3622	1811	-45.6	-50.6	4.8
22/07/08 12:23 AM	10041	3621	1810	-45.6	-50.6	4.8
22/07/08 12:24 AM	10038	3620	1810	-45.6	-50.6	4.8
22/07/08 12:25 AM	10032	3618	1809	-45.6	-50.8	4.8
22/07/08 12:26 AM	10029	3616	1808	-45.6	-50.8	4.8
22/07/08 12:27 AM	10026	3615	1808	-45.6	-50.8	4.8
22/07/08 12:28 AM	10022	3614	1807	-45.6	-50.8	4.8
22/07/08 12:29 AM	10020	3613	1807	-45.6	-50.8	4.8
22/07/08 12:30 AM	10017	3612	1806	-45.6	-50.8	4.8
22/07/08 12:31 AM	10014	3611	1806	-45.6	-50.8	4.8
22/07/08 12:32 AM	10010	3610	1805	-45.6	-50.8	4.8
22/07/08 12:33 AM	10008	3609	1804	-45.6	-50.8	4.8
22/07/08 12:34 AM	10007	3609	1804	-45.6	-50.8	4.8
22/07/08 12:35 AM	10003	3607	1804	-45.6	-50.8	4.8
22/07/08 12:36 AM	10000	3606	1803	-45.6	-50.8	4.8
22/07/08 12:37 AM	9997	3605	1802	-45.6	-50.8	4.8
22/07/08 12:38 AM	9996	3605	1802	-45.6	-50.8	4.8
22/07/08 12:39 AM	9994	3604	1802	-45.6	-50.8	4.8
22/07/08 12:40 AM	9992	3603	1802	-45.6	-50.8	4.8
22/07/08 12:41 AM	9992	3603	1802	-45.6	-50.8	4.8
22/07/08 12:42 AM	9993	3603	1802	-45.6	-50.8	4.8
22/07/08 12:43 AM	9989	3602	1801	-45.6	-50.8	4.8
22/07/08 12:44 AM	9990	3602	1801	-45.6	-50.8	4.8
22/07/08 12:45 AM	9988	3602	1801	-45.6	-50.8	4.8
22/07/08 12:46 AM	9989	3602	1801	-45.6	-50.8	4.8
22/07/08 12:47 AM	9988	3602	1801	-45.6	-50.8	4.8
22/07/08 12:48 AM	9986	3601	1800	-45.6	-50.8	4.8
22/07/08 12:49 AM	9986	3601	1800	-45.6	-50.8	4.8
22/07/08 12:50 AM	9986	3601	1800	-45.6	-50.8	4.8
22/07/08 12:51 AM	9986	3601	1800	-45.6	-50.8	4.8
22/07/08 12:52 AM	9985	3601	1800	-45.6	-50.8	4.8
22/07/08 12:53 AM	9984	3600	1800	-45.6	-50.8	4.8
22/07/08 12:54 AM	9984	3600	1800	-45.6	-50.8	4.8
22/07/08 12:55 AM	9980	3599	1799	-45.6	-50.8	4.8
22/07/08 12:56 AM	9978	3598	1799	-45.6	-50.8	4.8
22/07/08 12:57 AM	9975	3597	1798	-45.6	-50.8	4.8
22/07/08 12:58 AM	9974	3597	1798	-45.6	-50.8	4.8
22/07/08 12:59 AM	9973	3596	1798	-45.6	-50.8	4.8
22/07/08 01:00 AM	9972	3596	1798	-45.6	-50.8	4.8

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
22/07/08 01:01 AM	9972	3596	1798	-45.6	-50.8	4.8
22/07/08 01:02 AM	9968	3594	1797	-45.6	-50.8	4.8
22/07/08 01:03 AM	9966	3594	1797	-45.6	-50.8	4.8
22/07/08 01:04 AM	9966	3594	1797	-45.6	-50.8	4.8
22/07/08 01:05 AM	9964	3593	1797	-45.6	-50.8	4.8
22/07/08 01:06 AM	9963	3593	1796	-45.6	-50.8	4.8
22/07/08 01:07 AM	9962	3592	1796	-45.6	-50.8	4.8
22/07/08 01:08 AM	9959	3591	1796	-45.6	-50.8	4.8
22/07/08 01:09 AM	9958	3591	1795	-45.6	-50.8	4.8
22/07/08 01:10 AM	9957	3590	1795	-45.6	-50.8	4.8
22/07/08 01:11 AM	9954	3589	1795	-45.6	-50.8	4.8
22/07/08 01:12 AM	9952	3589	1794	-45.6	-50.8	4.8
22/07/08 01:13 AM	9951	3588	1794	-45.6	-50.8	4.8
22/07/08 01:14 AM	9950	3588	1794	-45.6	-50.8	4.8
22/07/08 01:15 AM	9950	3588	1794	-45.6	-50.8	4.8
22/07/08 01:16 AM	9948	3587	1794	-45.6	-50.8	4.8
22/07/08 01:17 AM	9946	3587	1793	-45.6	-50.8	4.8
22/07/08 01:18 AM	9944	3586	1793	-45.6	-50.8	4.8
22/07/08 01:19 AM	9941	3585	1792	-45.6	-50.8	4.8
22/07/08 01:20 AM	9941	3585	1792	-45.6	-50.8	4.8
22/07/08 01:21 AM	9939	3584	1792	-45.6	-50.8	4.8
22/07/08 01:22 AM	9940	3584	1792	-45.6	-50.8	4.8
22/07/08 01:23 AM	9940	3584	1792	-45.6	-50.8	4.8
22/07/08 01:24 AM	9940	3584	1792	-45.6	-50.8	4.8
22/07/08 01:25 AM	9939	3584	1792	-45.6	-50.8	4.8
22/07/08 01:26 AM	9938	3584	1792	-45.6	-50.8	4.8
22/07/08 01:27 AM	9936	3583	1791	-45.6	-50.8	4.8
22/07/08 01:28 AM	9936	3583	1791	-45.6	-50.8	4.8
22/07/08 01:29 AM	9936	3583	1791	-45.6	-50.8	4.8
22/07/08 01:30 AM	9937	3583	1792	-45.6	-50.8	4.8
22/07/08 01:31 AM	9937	3583	1792	-45.6	-50.8	4.8
22/07/08 01:32 AM	9935	3583	1791	-45.6	-50.8	4.8
22/07/08 01:33 AM	9936	3583	1791	-45.6	-50.8	4.8
22/07/08 01:34 AM	9935	3583	1791	-45.6	-50.8	4.8
22/07/08 01:35 AM	9933	3582	1791	-45.6	-50.8	4.8
22/07/08 01:36 AM	9932	3581	1791	-45.6	-50.8	4.8
22/07/08 01:37 AM	9934	3582	1791	-45.6	-50.8	4.8
22/07/08 01:38 AM	9935	3583	1791	-45.6	-50.8	4.8
22/07/08 01:39 AM	9932	3581	1791	-45.6	-50.8	4.8
22/07/08 01:40 AM	9928	3580	1790	-45.6	-50.8	4.8
22/07/08 01:41 AM	9924	3579	1789	-45.6	-50.8	4.8
22/07/08 01:42 AM	9922	3578	1789	-45.6	-50.8	4.8
22/07/08 01:43 AM	9918	3576	1788	-45.6	-50.8	4.8
22/07/08 01:44 AM	9916	3576	1788	-45.7	-50.8	4.8
22/07/08 01:45 AM	9911	3574	1787	-45.9	-50.8	4.8
22/07/08 01:46 AM	8996	3244	1622	-46.2	-50.3	4.8
22/07/08 01:47 AM	9206	3320	1680	-46.4	-50.3	4.7
22/07/08 01:48 AM	9108	3284	1642	-46.6	-50.3	4.7
22/07/08 01:49 AM	9004	3247	1623	-46.6	-50.3	4.7
22/07/08 01:50 AM	8789	3169	1585	-46.7	-50.2	4.7
22/07/08 01:51 AM	8655	3121	1560	-46.7	-50.2	4.7
22/07/08 01:52 AM	8675	3128	1564	-46.7	-50.2	4.7
22/07/08 01:53 AM	9181	3311	1635	-46.7	-50.2	4.7
22/07/08 01:54 AM	9274	3344	1672	-46.8	-50.2	4.7
22/07/08 01:55 AM	9320	3361	1680	-46.7	-50.2	4.7

Date / Time	Applied Pressure (PSI)	Effective Bidirectional Load (tonne)	Actual Jack Load (tonne)	Average Displacement (mm)		
				Pile Top	Cell Top	Cell Bottom
22/07/08 01:56 AM	8264	2980	1420	-46.8	-50.2	4.6
22/07/08 01:57 AM	8383	3023	1511	-46.7	-50.1	4.7
22/07/08 01:58 AM	8432	3041	1520	-46.8	-50.1	4.7
23/07/08 01:58 AM	8465	3052	1526	-46.7	-50.0	4.7
22/07/08 02:00 AM	8494	3063	1531	-46.8	-50.1	4.6
22/07/08 02:01 AM	8515	3071	1535	-46.7	-50.0	4.7
23/07/08 02:02 AM	8526	3074	1537	-46.8	-50.0	4.6
22/07/08 02:03 AM	8548	3082	1541	-46.7	-50.0	4.7
22/07/08 02:04 AM	8558	3086	1543	-46.7	-50.0	4.6
22/07/08 02:05 AM	7156	2580	1290	-46.7	-50.0	4.6
23/07/08 02:05 AM	7977	2877	1438	-46.7	-50.0	4.6
22/07/08 02:07 AM	8051	2903	1452	-46.7	-50.0	4.6
22/07/08 02:08 AM	8093	2918	1489	-46.7	-50.0	4.6
22/07/08 02:09 AM	8121	2928	1464	-46.7	-50.0	4.6
22/07/08 02:10 AM	8142	2936	1468	-46.7	-50.0	4.6
22/07/08 02:11 AM	8162	2943	1472	-46.7	-50.0	4.6
23/07/08 02:11 AM	8182	2950	1475	-46.7	-50.0	4.6
22/07/08 02:13 AM	8190	2953	1477	-46.7	-49.9	4.6
22/07/08 02:14 AM	7324	2641	1321	-46.7	-50.0	4.6
22/07/08 02:15 AM	6947	2505	1253	-46.7	-49.9	4.6
22/07/08 02:16 AM	6988	2520	1260	-46.7	-49.4	4.2
22/07/08 02:17 AM	7014	2529	1265	-46.7	-47.6	3.4
22/07/08 02:18 AM	7038	2538	1269	-46.7	-47.5	3.3
22/07/08 02:19 AM	7060	2546	1273	-46.7	-47.5	3.3
22/07/08 02:20 AM	7059	2545	1273	-46.7	-47.5	3.3
22/07/08 02:21 AM	7101	2561	1280	-46.7	-47.5	3.2
22/07/08 02:22 AM	7120	2567	1284	-46.7	-47.5	3.2
22/07/08 02:23 AM	7133	2572	1286	-46.7	-47.5	3.2
22/07/08 02:24 AM	6036	2177	1088	-46.7	-47.5	3.2
22/07/08 02:25 AM	6116	2205	1103	-46.7	-47.5	3.2
22/07/08 02:26 AM	6162	2222	1111	-46.7	-47.5	3.2
22/07/08 02:27 AM	6195	2234	1117	-46.7	-47.5	3.2
22/07/08 02:28 AM	6221	2243	1122	-46.7	-47.5	3.2
22/07/08 02:29 AM	6243	2251	1126	-46.7	-47.5	3.2
22/07/08 02:30 AM	6261	2258	1129	-46.7	-47.4	3.2
22/07/08 02:31 AM	6279	2264	1132	-46.7	-47.5	3.2
22/07/08 02:32 AM	6297	2271	1135	-46.7	-47.4	3.2
22/07/08 02:33 AM	4336	1564	783	-46.7	-47.5	3.2
22/07/08 02:34 AM	4445	1603	801	-46.7	-47.4	3.2
22/07/08 02:35 AM	3944	1422	711	-46.7	-47.5	3.2
22/07/08 02:36 AM	4013	1447	724	-46.7	-47.4	3.2
22/07/08 02:37 AM	4059	1464	731	-46.7	-47.4	3.2
22/07/08 02:38 AM	4096	1477	739	-46.7	-47.5	3.1
22/07/08 02:39 AM	4126	1488	744	-46.7	-47.5	3.1
22/07/08 02:40 AM	4152	1497	749	-46.7	-47.4	3.2
22/07/08 02:41 AM	4175	1506	753	-46.7	-47.5	3.2
22/07/08 02:42 AM	4195	1513	756	-46.7	-47.5	3.2
22/07/08 02:43 AM	3121	1125	563	-46.7	-47.4	3.2
22/07/08 02:44 AM	3168	1142	57	-46.7	-47.4	3.2
22/07/08 02:45 AM	3204	1155	578	-46.7	-47.4	3.1
22/07/08 02:46 AM	3233	1166	583	-46.7	-47.4	3.2
22/07/08 02:47 AM	3259	1175	588	-46.7	-47.4	3.2
22/07/08 02:48 AM	3281	1183	592	-46.7	-47.4	3.2
22/07/08 02:49 AM	3302	1191	595	-46.7	-47.4	3.2
22/07/08 02:50 AM	3319	1197	598	-46.7	-47.4	3.2

Dubai Mixed Use Development, Dubai, U.A.E
 BUL # 0702

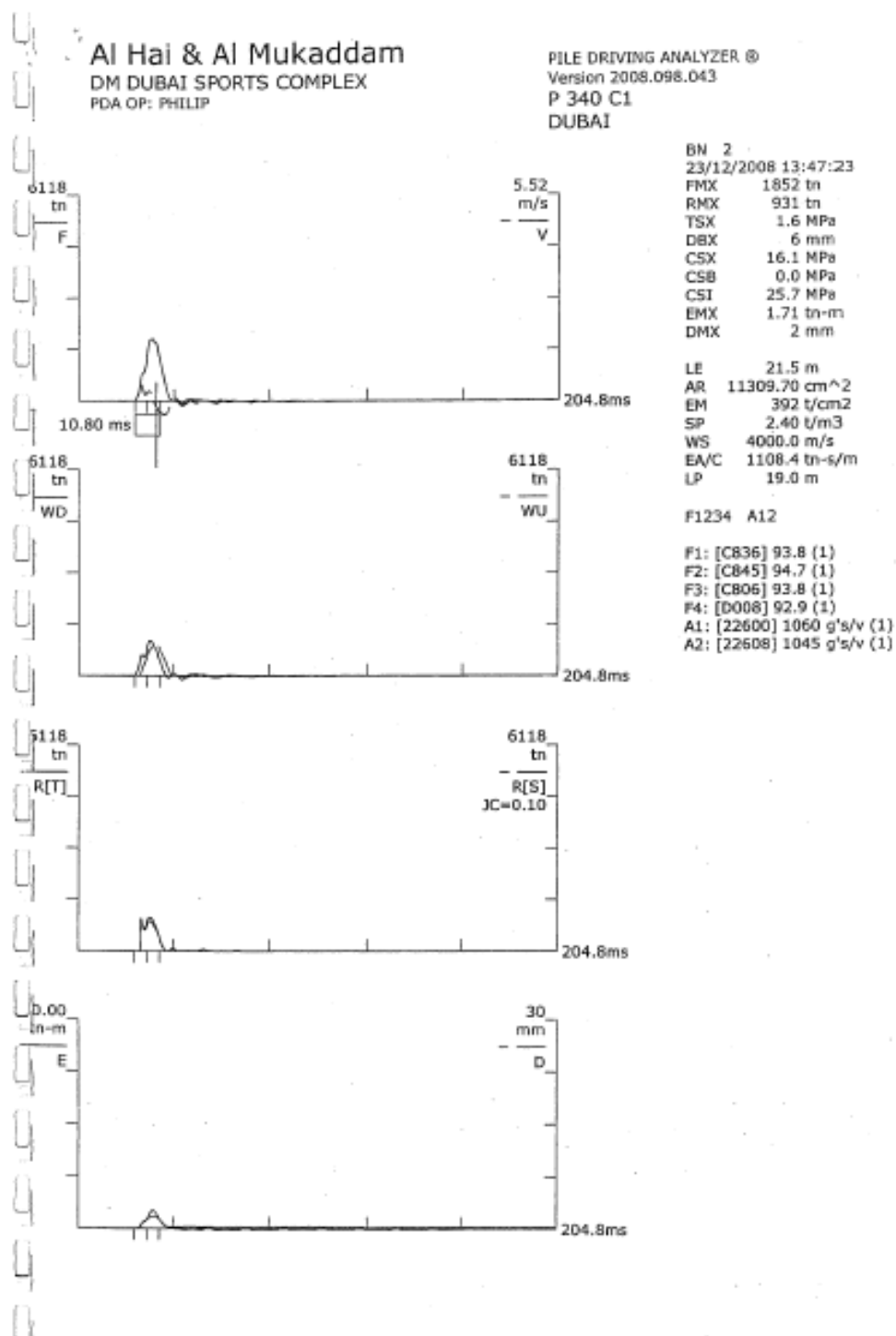
APPENDIX D: CASE STUDY 3 – STATIC LOAD TESTING

104th TECHNICALS A.S.			WORKING PILE LOAD TEST							Consultant	ARUP
Place			Test no.		PWL	11,500 kN				Main Contractor	
Duration	20:20 h										
Extra load	0 kN		Pile no.	340						Supervisor	AL HAMMADIAN
Max. load	17,250 kN		Pile Dia.	1200							
Time		Load		Strain gauge readings							
real	test	stop	kN	bar	1	2	3	4	average		
17:00	00:00:00	00:00	0	0	0.00	0.00	0.00	0.00	0.00		
17:00	00:00:00	00:00	2,875	80	0.12	0.02	0.34	0.00	0.12		
17:10	00:10:00	00:10	2,875	80	0.13	0.02	0.36	0.02	0.13		
17:20	00:20:00	00:10	2,875	80	0.13	0.00	0.36	0.02	0.14		
17:30	00:30:00	00:10	2,875	80	0.13	0.00	0.36	0.02	0.14		
17:30	00:30:00	00:00	5,750	173	0.47	0.10	0.73	0.26	0.40		
17:40	00:40:00	00:10	5,750	173	0.51	0.10	0.80	0.26	0.42		
17:50	00:50:00	00:10	5,750	173	0.51	0.14	0.80	0.28	0.43		
18:00	01:00:00	00:10	5,750	173	0.51	0.14	0.80	0.28	0.43		
18:00	01:00:00	00:00	8,625	259	1.07	0.35	1.42	0.66	0.86		
18:10	01:10:00	00:10	8,625	259	1.12	0.37	1.46	0.67	0.91		
18:20	01:20:00	00:10	8,625	259	1.13	0.37	1.46	0.68	0.92		
18:30	01:30:00	00:10	8,625	259	1.13	0.37	1.46	0.68	0.92		
18:30	01:30:00	00:00	11,500	346	1.67	0.64	2.13	1.21	1.41		
18:30	02:00:00	00:30	11,500	346	1.69	0.65	2.20	1.28	1.48		
18:30	02:30:00	00:30	11,500	346	1.81	0.66	2.28	1.30	1.61		
20:30	03:00:00	00:30	11,500	346	1.81	0.66	2.28	1.30	1.61		
20:30	03:30:00	00:30	11,500	346	1.86	0.68	2.38	1.30	1.69		
21:00	04:00:00	00:30	11,500	346	1.88	0.69	2.39	1.30	1.69		
21:30	04:30:00	00:30	11,500	346	1.86	0.68	2.30	1.31	1.54		
22:00	05:00:00	00:30	11,500	346	1.86	0.68	2.31	1.31	1.54		
22:30	05:30:00	00:30	11,500	346	1.86	0.68	2.31	1.31	1.54		
23:00	06:00:00	00:30	11,500	346	1.86	0.68	2.31	1.31	1.54		
23:30	06:30:00	00:30	11,500	346	1.88	0.70	2.34	1.31	1.67		
00:00	07:00:00	00:30	11,500	346	1.88	0.70	2.35	1.31	1.67		
00:30	07:30:00	00:30	11,500	346	1.86	0.70	2.35	1.31	1.67		
00:30	07:30:00	00:00	8,625	259	1.74	0.75	2.22	1.31	1.69		
00:35	07:35:00	00:05	8,625	259	1.71	0.72	2.22	1.31	1.69		
00:40	07:40:00	00:05	8,625	259	1.71	0.72	2.20	1.31	1.69		
00:40	07:40:00	00:00	5,750	173	1.47	0.66	1.87	1.31	1.33		
00:45	07:45:00	00:05	5,750	173	1.47	0.66	1.87	1.31	1.33		
00:50	07:50:00	00:05	5,750	173	1.47	0.66	1.87	1.31	1.33		
00:50	07:50:00	00:00	2,875	80	1.23	0.60	1.60	1.31	1.19		
00:55	07:55:00	00:05	2,875	80	1.23	0.60	1.60	1.31	1.19		
01:00	08:00:00	00:05	2,875	80	1.23	0.60	1.59	1.31	1.18		

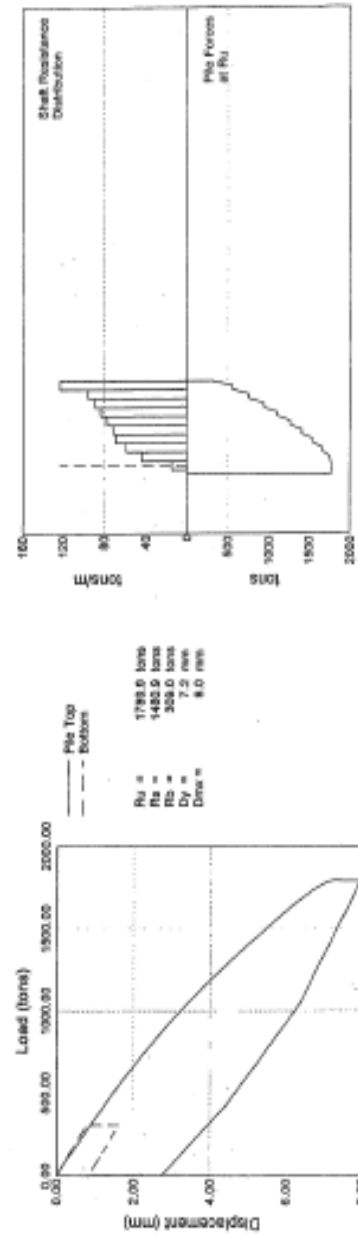
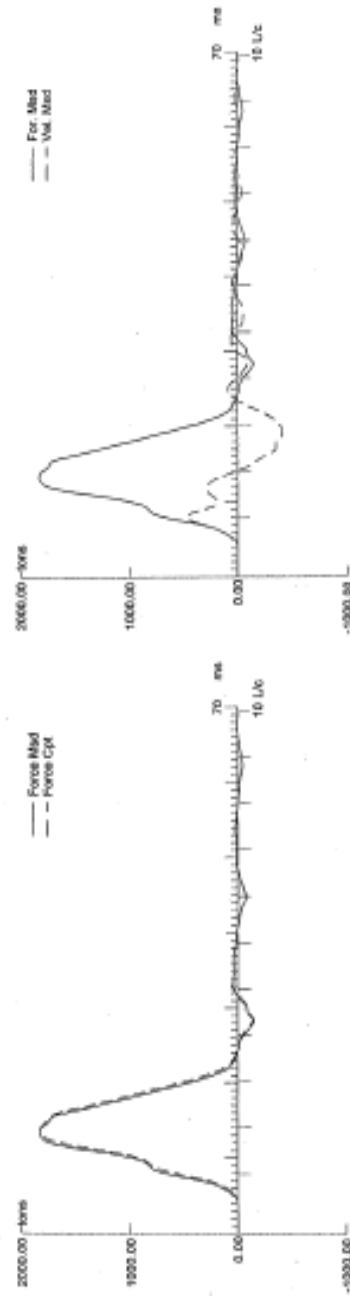
 ZMN ENJOUS A.S.		WORKING FILE LOAD TEST								Consultant	20/7/2008
Place		Test no.		PWL	11,500 kN					Main Contractor	ARUP
Duration	20:20 h										
Extra load	0 kN	Pile no.	340							Supervisor	AL HAMMUKADAM
Max. load	17,250 kN	Pile Dia.	1200								
time		load		dial gauge readings							
real	test	stop	kN	bar	1	2	3	4	average		
0% PWL											
01:00	00:00:00	00:00	0	0	0.83	0.82	1.28	1.31	0.99		
01:15	00:15:00	00:15	0	0	0.83	0.82	1.28	1.31	0.99		
01:30	00:30:00	00:15	0	0	0.83	0.81	1.28	1.31	0.99		
01:45	00:45:00	00:15	0	0	0.72	0.50	0.95	1.31	0.87		
02:10	00:50:00	00:15	0	0	0.72	0.50	0.95	1.31	0.87		
20% PWL											
12:00	00:00:00	00:00	2,375	79	0.16	0.12	0.02	0.00	0.08		
12:15	00:15:00	00:10	2,375	79	0.17	0.12	0.02	0.00	0.08		
12:20	00:20:00	00:10	2,375	79	0.17	0.14	0.04	0.00	0.09		
12:30	00:30:00	00:10	2,375	79	0.17	0.14	0.04	0.00	0.09		
50% PWL											
12:30	00:30:00	00:00	5,750	159	0.42	0.19	0.41	0.16	0.30		
12:40	00:40:00	00:10	5,750	159	0.44	0.21	0.42	0.17	0.31		
12:50	00:50:00	00:10	5,750	159	0.45	0.22	0.43	0.17	0.32		
13:00	00:00:00	00:10	5,750	159	0.45	0.22	0.43	0.17	0.32		
75% PWL											
13:00	00:00:00	00:00	8,625	238	0.74	0.29	0.74	0.29	0.54		
13:10	00:10:00	00:10	8,625	238	0.78	0.33	0.77	0.40	0.57		
13:20	00:20:00	00:10	8,625	238	0.78	0.33	0.78	0.41	0.58		
13:30	00:30:00	00:10	8,625	238	0.78	0.33	0.80	0.42	0.58		
100% PWL											
13:30	00:30:00	00:00	11,500	318	1.14	0.45	0.90	0.84	0.78		
13:45	00:45:00	00:15	11,500	318	1.16	0.17	0.93	0.85	0.80		
14:00	01:00:00	00:15	11,500	318	1.21	0.19	0.90	0.85	0.83		
14:15	01:15:00	00:15	11,500	318	1.28	0.31	1.03	0.88	0.87		
14:30	01:30:00	00:15	11,500	318	1.28	0.31	1.04	0.88	0.87		
125% PWL											
14:30	01:30:00	00:00	14,375	397	1.72	0.90	1.58	1.09	1.50		
14:45	01:45:00	00:15	14,375	397	1.78	0.97	1.68	1.10	1.58		
15:00	02:00:00	00:15	14,375	397	1.82	0.90	1.73	1.15	1.60		
15:15	02:15:00	00:15	14,375	397	1.85	0.91	1.77	1.20	1.63		
15:30	02:30:00	00:15	14,375	397	1.87	0.91	1.80	1.23	1.68		
150% PWL											
16:00	02:30:00	00:00	17,250	477	2.50	1.54	2.50	1.89	2.08		
16:30	02:30:00	01:00	17,250	477	2.75	1.47	2.91	2.10	2.31		
17:00	02:30:00	01:00	17,250	477	2.80	1.51	2.93	2.18	2.37		
18:30	02:30:00	01:00	17,250	477	2.80	1.51	3.09	2.19	2.40		
19:30	02:30:00	01:00	17,250	477	2.82	1.51	3.15	2.22	2.42		
20:00	02:30:00	01:00	17,250	477	2.85	1.51	3.15	2.24	2.44		
21:30	02:30:00	01:00	17,250	477	2.85	1.51	3.18	2.25	2.45		

WORKING FILE LOAD TEST 1			Consultant		ARUP				
Place		Test no.	PWL	11,500 kN		Main Contractor			
Duration	20:20 h								
Extra load	0 kN	Pile no.	340			Supervisor			
Max. load	17,250 kN	Pile Dia.	1200			AL HAMAMUKADAM			
time		load		dial gauge readings					
real	test	stop	kN	bar	1	2	3	4	average
125% PWL									
21:30	18:30:00	00:00	14,375	397	2.65	1.42	2.90	2.90	2.24
21:35	18:35:00	00:05	14,375	397	2.83	1.42	2.90	2.90	2.24
21:40	18:40:00	00:05	14,375	397	2.83	1.42	2.90	2.90	2.24
100% PWL									
21:40	18:40:00	00:00	11,500	318	2.38	1.30	2.62	1.75	2.01
21:45	18:45:00	00:05	11,500	318	2.37	1.29	2.62	1.74	2.01
21:50	18:50:00	00:05	11,500	318	2.37	1.29	2.61	1.74	2.00
75% PWL									
21:50	18:50:00	00:05	8,625	328	2.07	1.14	2.25	1.43	1.72
21:55	18:55:00	00:05	8,625	328	2.05	1.13	2.25	1.43	1.72
22:00	19:00:00	00:05	8,625	328	2.05	1.13	2.25	1.43	1.72
50% PWL									
22:00	19:00:00	00:00	5,750	159	1.09	1.00	1.85	1.07	1.40
22:05	19:05:00	00:05	5,750	159	1.85	0.98	1.83	1.06	1.38
22:10	19:10:00	00:05	5,750	159	1.85	0.98	1.83	1.05	1.38
25% PWL									
22:10	19:10:00	00:00	2,875	79	1.25	0.80	1.31	0.61	0.99
22:15	19:15:00	00:05	2,875	79	1.25	0.80	1.31	0.61	0.99
22:20	19:20:00	00:05	2,875	79	1.22	0.80	1.30	0.60	0.96
0% PWL									
22:20	19:20:00	00:00	0	0	1.03	0.65	0.37	0.10	0.84
22:35	19:35:00	00:15	0	0	0.81	0.81	0.31	0.10	0.41
22:50	19:50:00	00:15	0	0	0.80	0.80	0.31	0.10	0.40
23:05	20:05:00	00:15	0	0	0.80	0.80	0.30	0.10	0.40
23:20	20:20:00	00:15	0	0	0.80	0.80	0.30	0.10	0.40

APPENDIX E: CASE STUDY 3 – DYNAMIC LOAD TESTING



DM DUBAI SPORTS COMPLEX; Pile: P 340 C1; DUBAI; BN: 2 (Test: 23-Dec-2008)
 Al Hal & Al Mukaddam for Geotechnical WC
 27-Dec-2008
 CAPWAP® Ver. 2000-1



DM DUBAI SPORTS COMPLEX; File: P 340 C1
 DUBAI; Blow: 2
 Al Rai & Al Mukaddas for Geotechnical WC

Test: 23-Dec-2008
 CAPWAP Ver. 2000-1
 GP: PHILIP

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 1789.9; along Shaft 1480.9; at Toe 309.0 tons

Soil Spent No.	Dist. Below Gages m	Depth Below Grade m	Ru tons	Force in Pile tons	Sum of Ru tons	Unit Resist. Ru tons/m	Unit Resist. (Area) tons/m ²	Smith Damping Factor s/n	Quake mm
				1789.9					
1	3.1	0.6	30.0	1759.9	30.0	14.63	3.88	1.364	1.000
2	5.1	2.6	89.9	1670.0	119.9	43.92	11.65	1.364	1.241
3	7.2	4.7	119.9	1550.1	239.8	58.55	15.53	1.364	1.334
4	9.2	6.7	140.6	1409.5	380.4	68.88	18.22	1.364	1.334
5	11.3	8.8	144.9	1264.6	525.3	70.75	18.77	1.364	1.334
6	13.3	10.8	157.6	1107.0	682.9	76.98	20.42	1.364	1.334
7	15.4	12.9	170.5	936.5	853.4	83.28	22.09	1.364	1.313
8	17.4	14.9	181.0	755.4	1034.5	88.42	23.45	1.364	1.029
9	19.5	17.0	195.1	560.3	1229.5	95.27	25.27	1.364	0.806
10	21.5	19.0	251.4	309.0	1480.9	122.76	32.56	1.364	0.653
Avg. Skin			148.1			77.94	19.18	1.364	1.097
Toe			309.0				273.20	0.391	0.800

Soil Model Parameters/Extensions

	Skin	Toe
Case Damping Factor	1.822	0.109
Unloading Quake (% of loading quake)	26	100
Reloading Level (% of Ru)	98	98
Unloading Level (% of Ru)	57	
Resistance Gap (included in Toe Quake) (mm)		0.010
Soil Plug Weight (tons)		2.41

CAPWAP match quality: 0.82 (Wave Up Match)
 Observed: final set = 1.000 mm; blow count = 1000 b/m
 Computed: final set = 1.627 mm; blow count = 615 b/m

DM DUBAI SPORTS COMPLEX; File: F 340 C1
 DUBAI; Blow: 2
 Al Hai & Al Mokaddam for Geotechnical WC

Test: 23-Dec-2008
 CAPWAP Ver. 2000-1
 OP: PHILIP

EXTREME TABLE

File Sgmt No.	Dist. Below Gages m	max. Force tons	min. Force tons	max. Comp. Stress tons/cm ²	max. Tens. Stress tons/cm ²	max. Enefcd. Energy tonne-m	max. Veloc. m/s	max. Displ. mm
1	1.0	1837.9	-147.7	0.163	-0.013	12.89	2.4	10.884
2	2.0	1896.9	-162.3	0.168	-0.014	9.13	1.1	6.437
3	3.1	1951.8	-212.5	0.173	-0.019	8.91	1.1	6.114
4	4.1	1940.0	-245.9	0.172	-0.022	8.33	1.0	5.789
5	5.1	1997.1	-282.6	0.177	-0.025	8.10	1.0	5.456
6	6.1	1858.6	-251.7	0.164	-0.022	7.02	0.9	5.137
7	7.2	1912.7	-253.9	0.169	-0.022	6.79	0.9	4.810
8	8.2	1726.9	-191.2	0.153	-0.017	5.66	0.8	4.499
9	9.2	1773.0	-187.8	0.157	-0.017	5.44	0.8	4.174
10	10.2	1576.4	-138.3	0.139	-0.012	4.37	0.7	3.869
11	11.3	1615.4	-152.0	0.143	-0.013	4.16	0.7	3.553
12	12.3	1434.7	-96.3	0.127	-0.009	3.29	0.6	3.265
13	13.3	1496.2	-117.1	0.132	-0.010	3.13	0.6	2.982
14	14.3	1333.2	-46.6	0.118	-0.004	2.44	0.5	2.739
15	15.4	1370.4	-65.5	0.121	-0.006	2.32	0.5	2.504
16	16.4	1168.5	-0.2	0.103	-0.000	1.79	0.4	2.313
17	17.4	1182.7	-0.2	0.105	-0.000	1.72	0.3	2.131
18	18.4	945.3	-0.2	0.084	-0.000	1.29	0.3	1.990
19	19.5	951.2	-0.2	0.084	-0.000	1.25	0.3	1.849
20	20.5	707.3	-0.2	0.063	-0.000	0.88	0.3	1.747
21	21.5	706.2	-0.2	0.062	-0.000	0.42	0.3	1.636
Absolute	5.1			0.177			(T =	22.0 ms)
	5.1				-0.025		(T =	35.3 ms)

DM DUBAI SPORTS COMPLEX; File: P 340 C1
 DUBAI; Blow: 2
 Al Sai & Al Muhaddam for Geotechnical WC

Test: 23-Dec-2008
 CAPWAP Ver. 2000-1
 OP: PHILIP

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RS1	953.9	931.0	908.0	885.1	862.1	839.1	816.2	793.2	770.3	747.3
RMX	989.8	931.0	908.0	885.1	862.1	839.1	816.2	793.2	770.3	747.3
RSU	1755.4	1812.5	1869.7	1926.9	1984.1	2041.3	2098.4	2155.6	2212.8	2270.0

RAU= 0.2 (tons); RA2= 953.6 (tons)

Current CAPWAP R= 1789.9 (tons); Corresponding J(Ra)= 0.00; J(Rx)=0.00

VAK	VFN	VT1+2	FT1	DMX	DMX	DFN	RMX	RLT
m/s	m/s	tons	tons	tons	mm	mm	tonne-m	tons
0.44	0.01	486.5	697.0	1851.5	1.769	1.000	1.7	1238.1

DM DUBAI SPORTS COMPLEX; File: P 340 C1
 DUBAI; Blow: 2
 Al Hai & Al Mukaddam for Geotechnical WC

Test: 23-Dec-2008
 CAPWAP® Ver. 2000-1
 OP: PHILIP

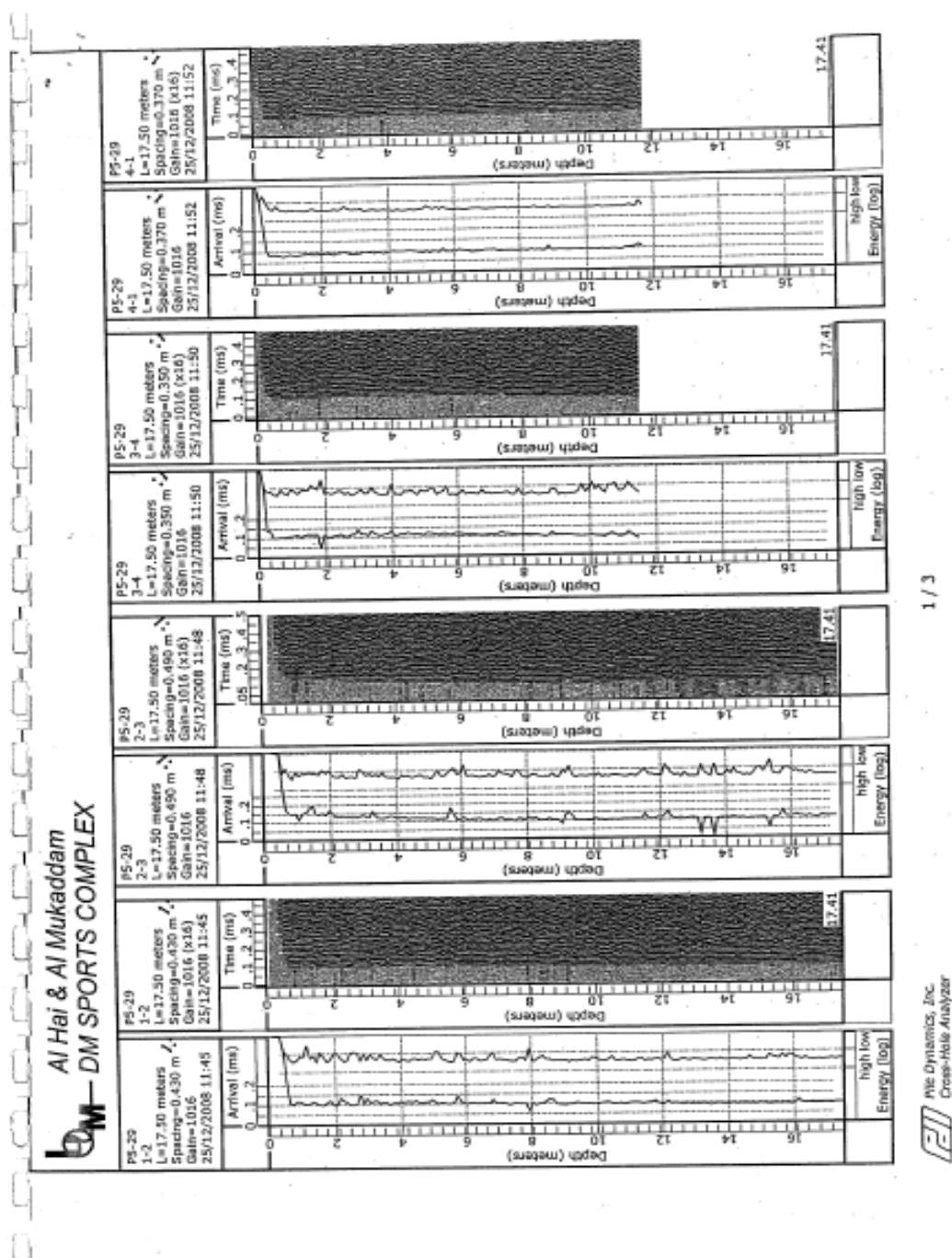
PILE PROFILE AND PILE MODEL

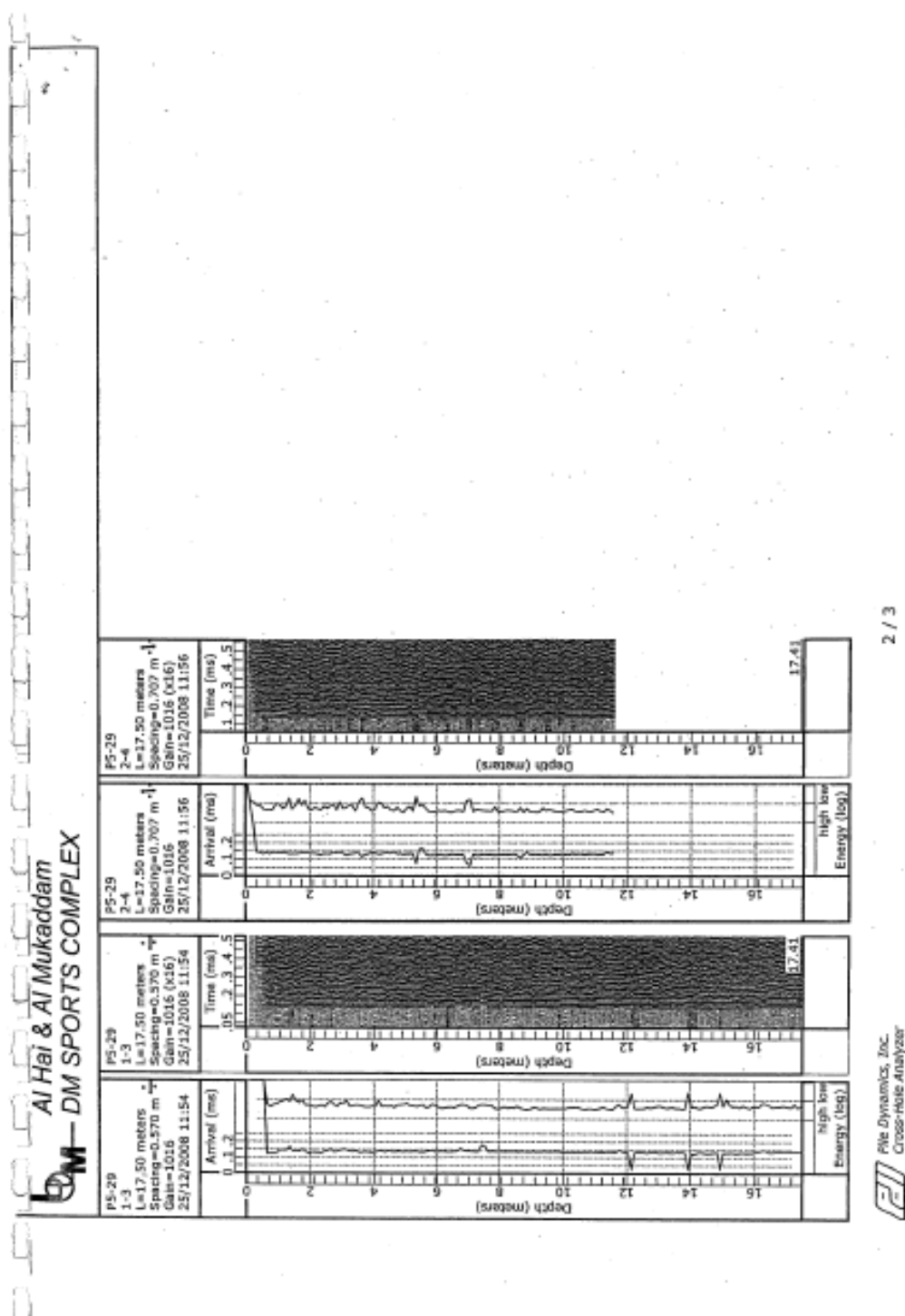
Depth m	Area cm ²	E-Modulus tons/cm ²	Spec. Weight tons/m ³	Circumf. m
0.00	11309.70	392.0	2.403	3.770
21.50	11309.70	392.0	2.403	3.770
Tot Area		1.131	m ²	

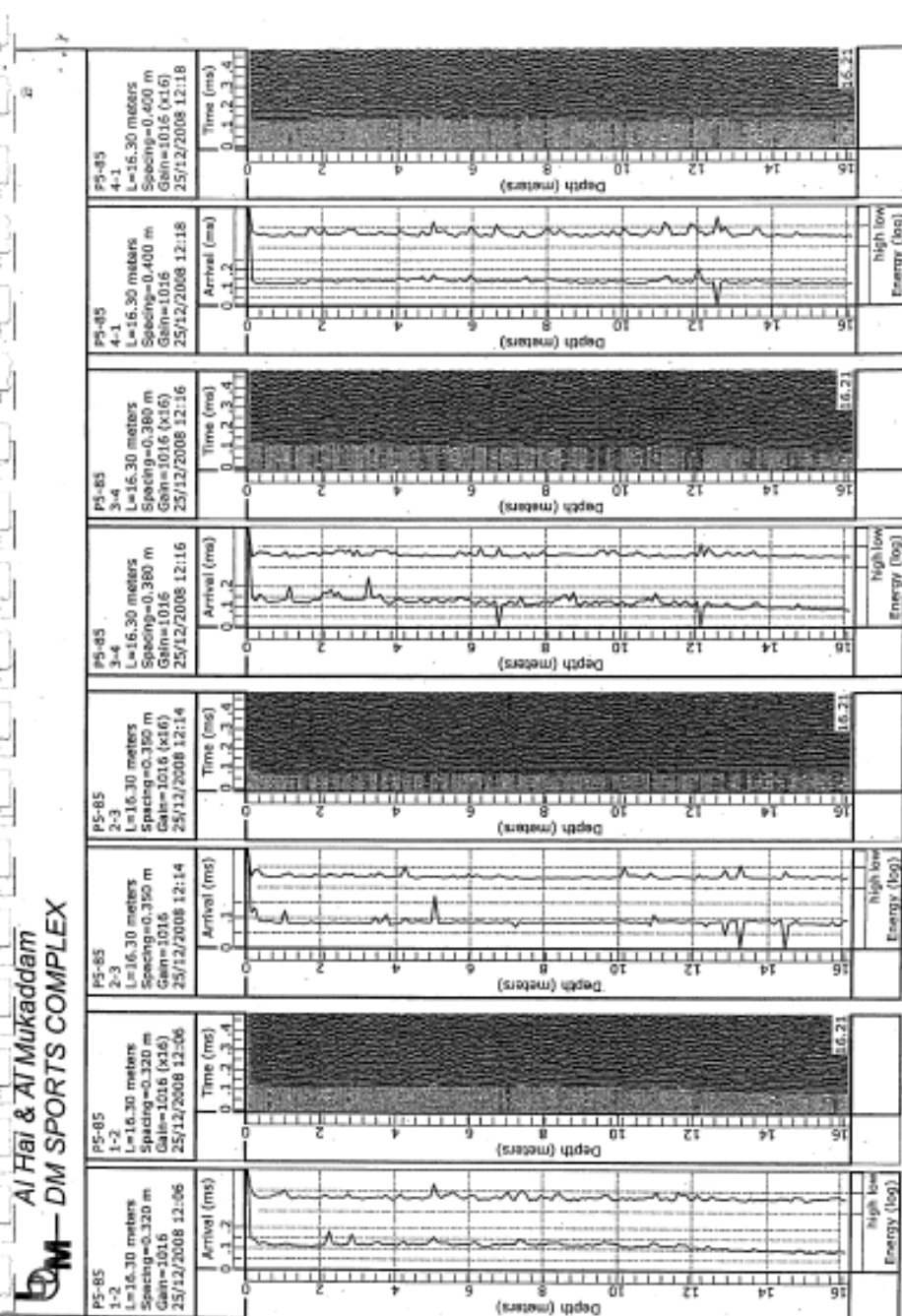
Top Segment Length 1.02 m, Top Impedance 1108.42 tons/m/s

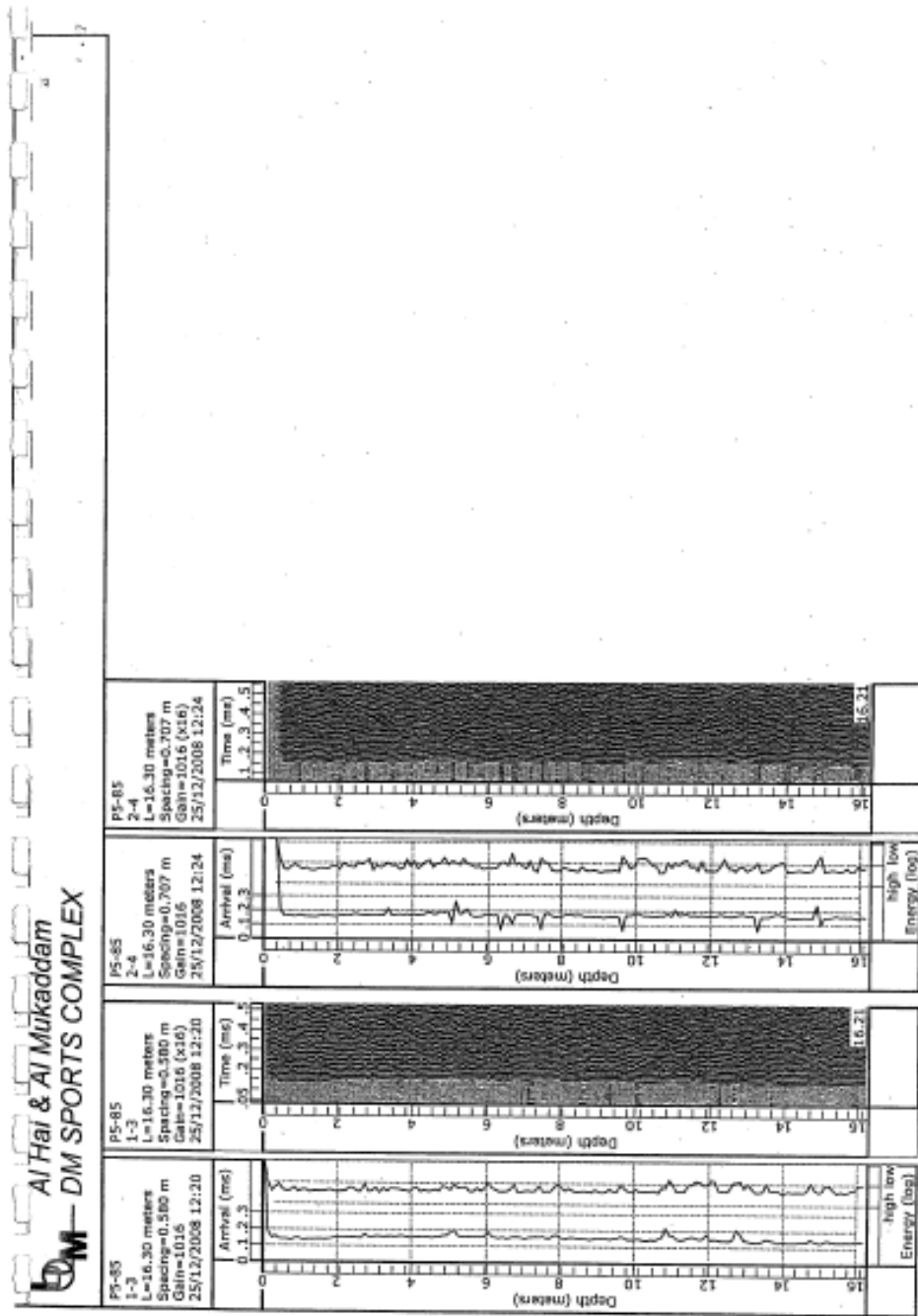
Pile Damping 2.0 %, Time Incr 0.256 ms, Wave Speed 4000.0 m/s

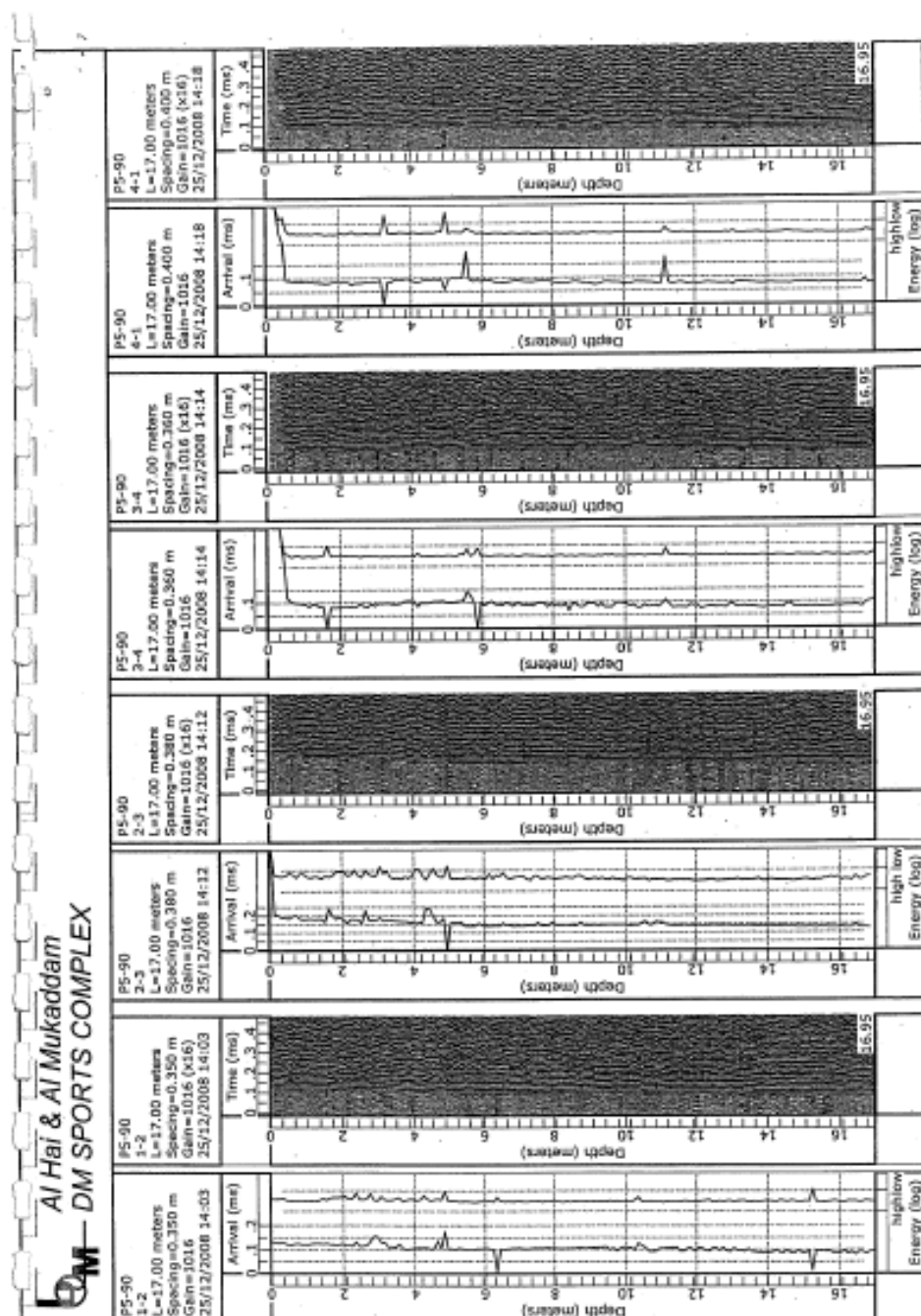
APPENDIX F: CASE STUDY 4 – CROSS-HOLE SONIC LOGGING TEST

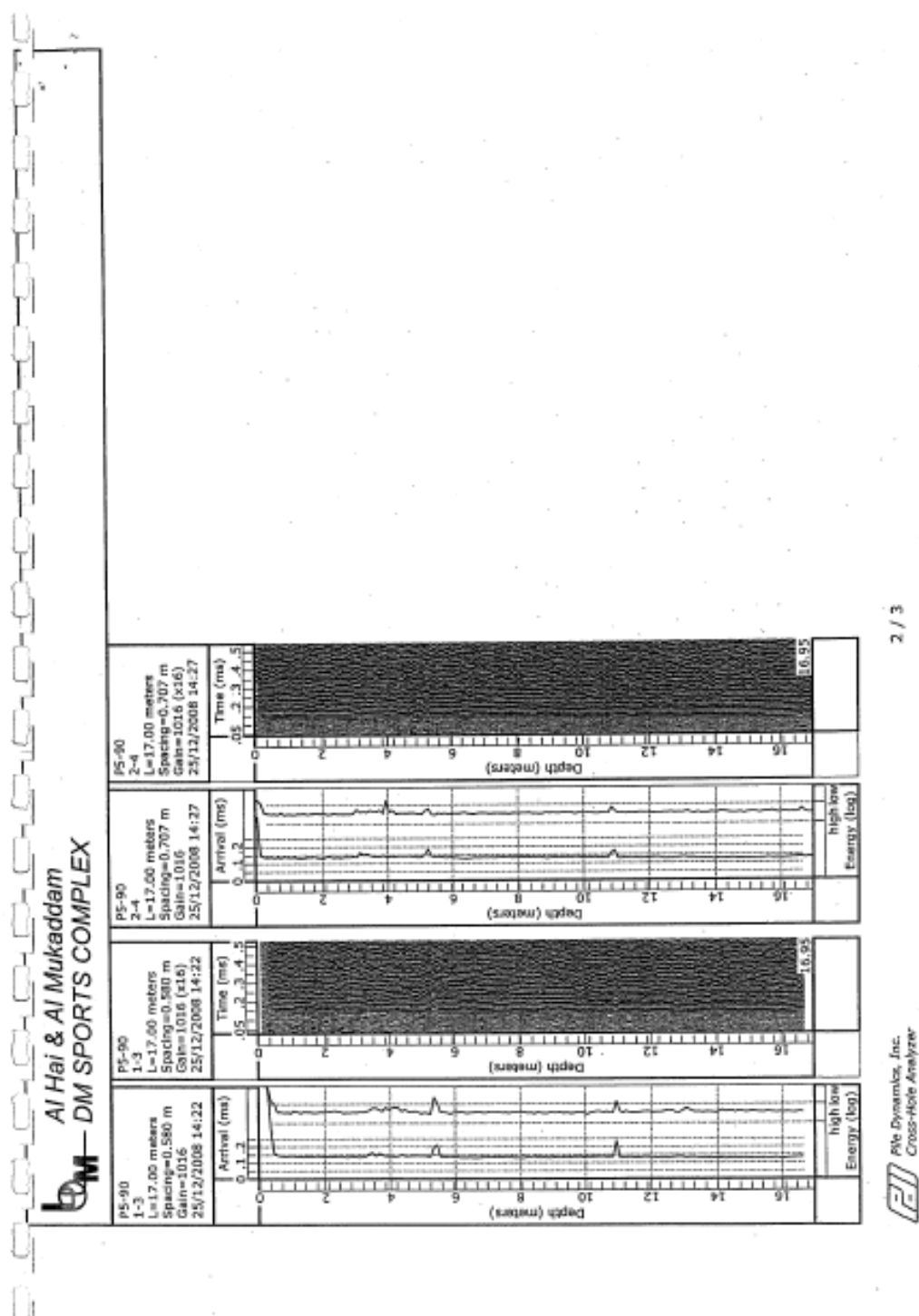


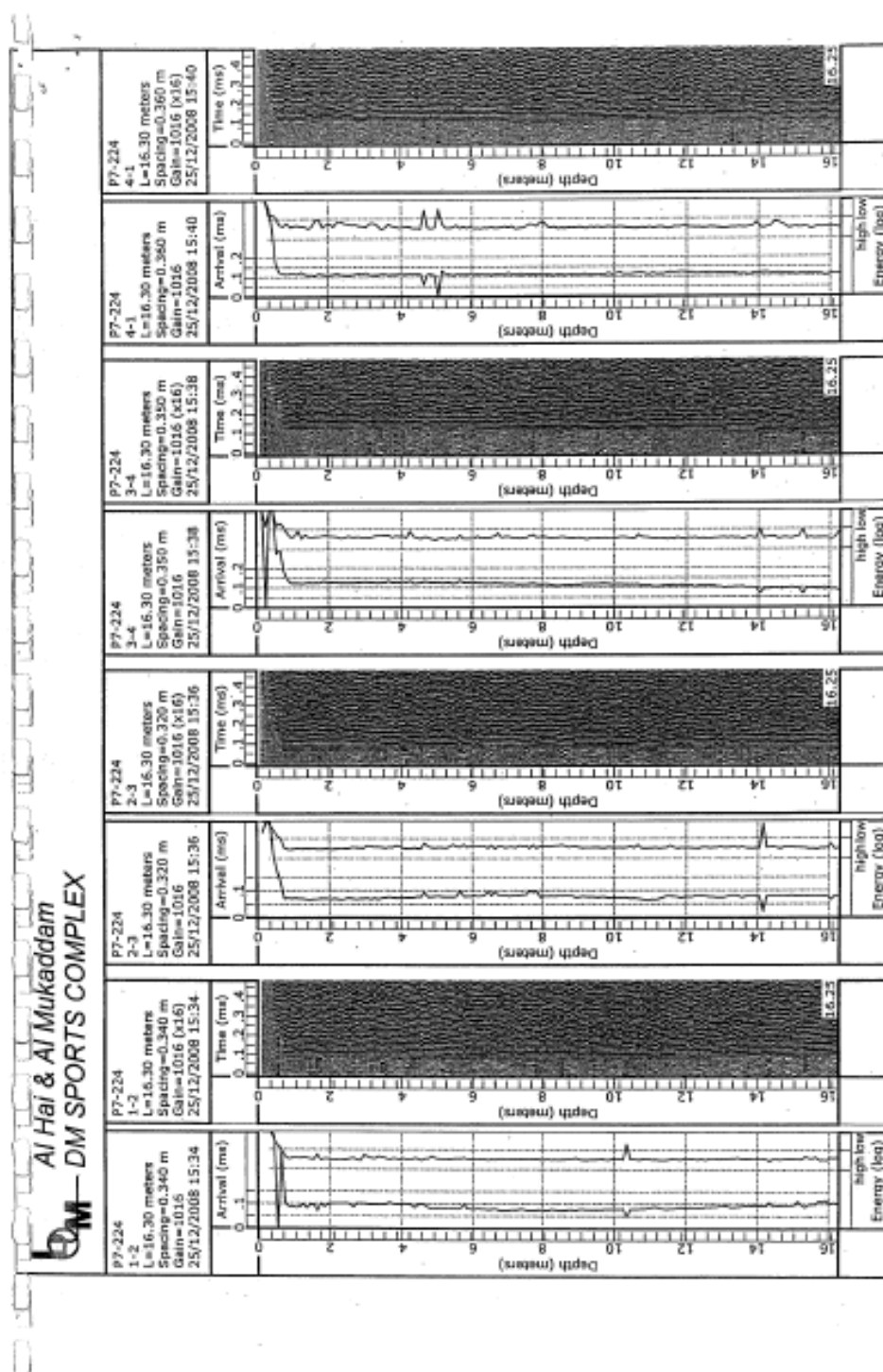


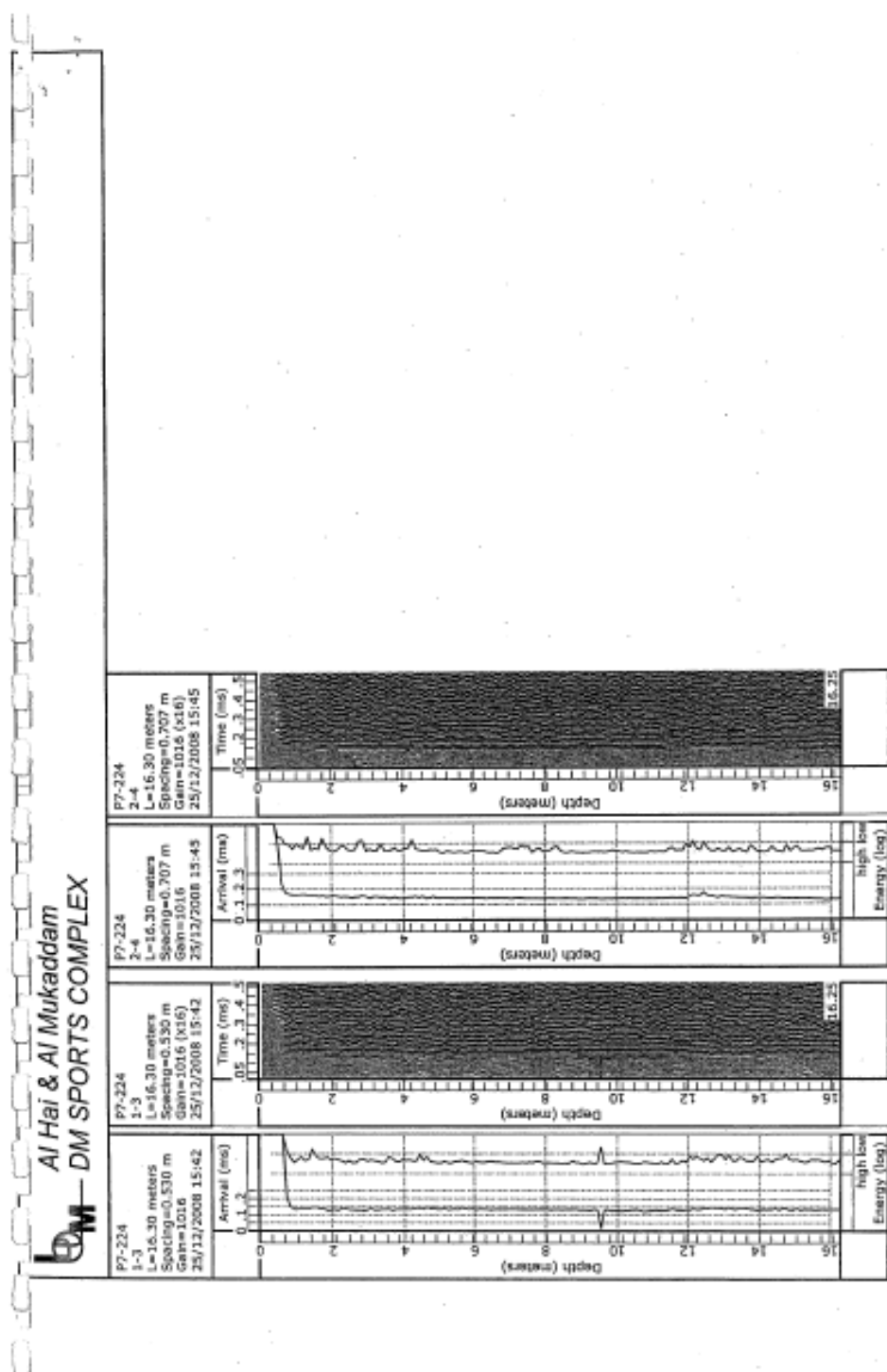


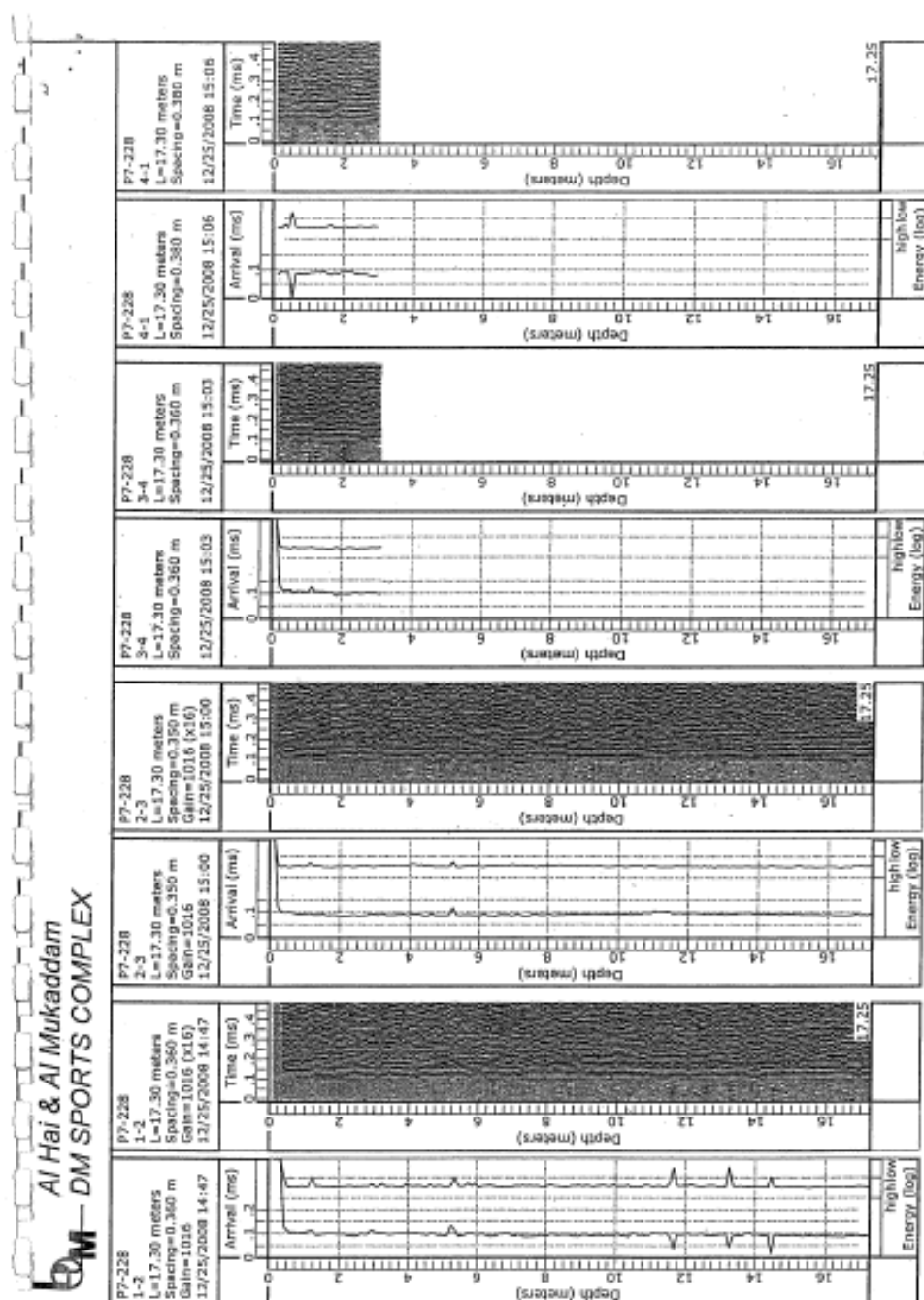


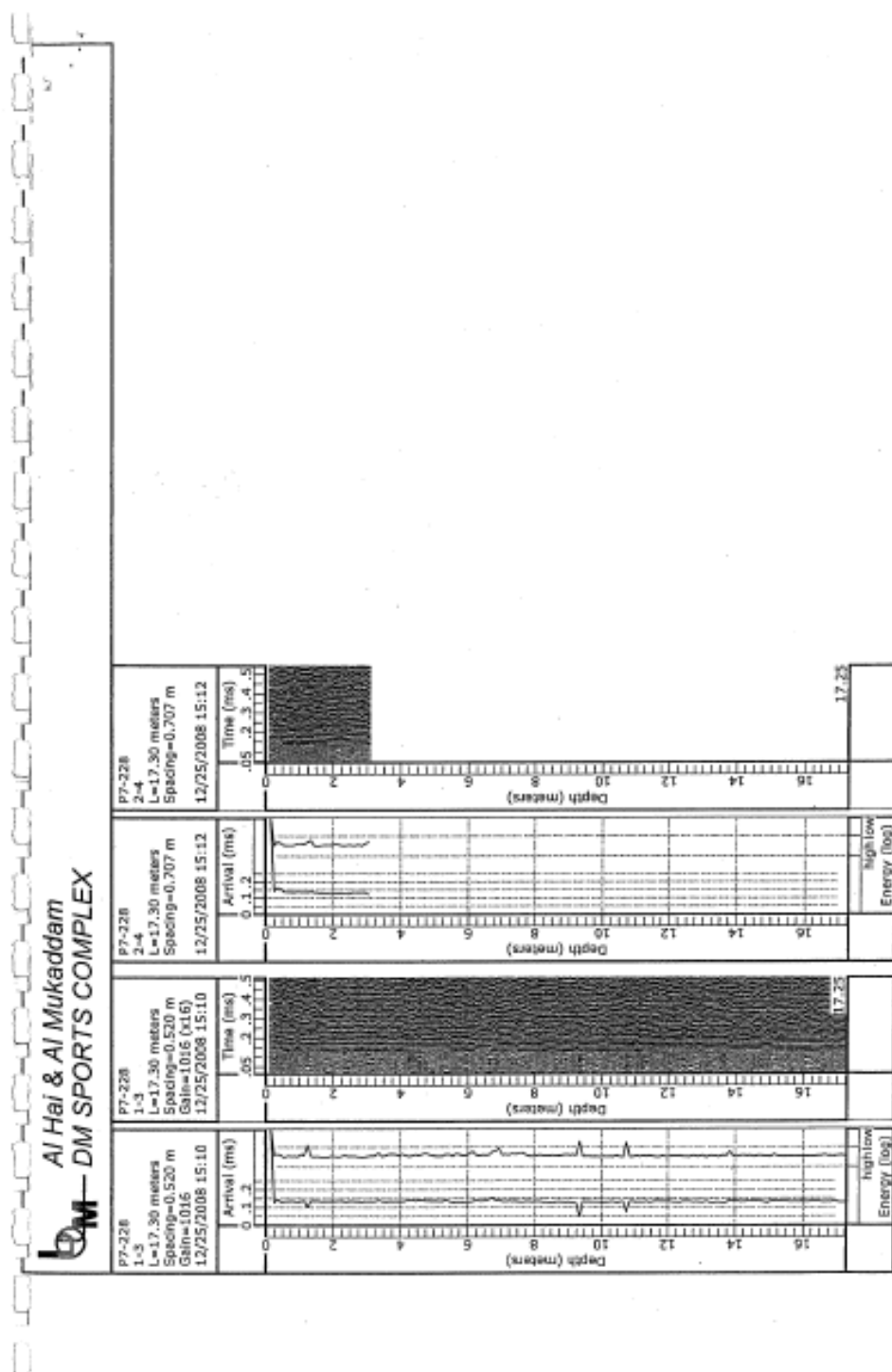




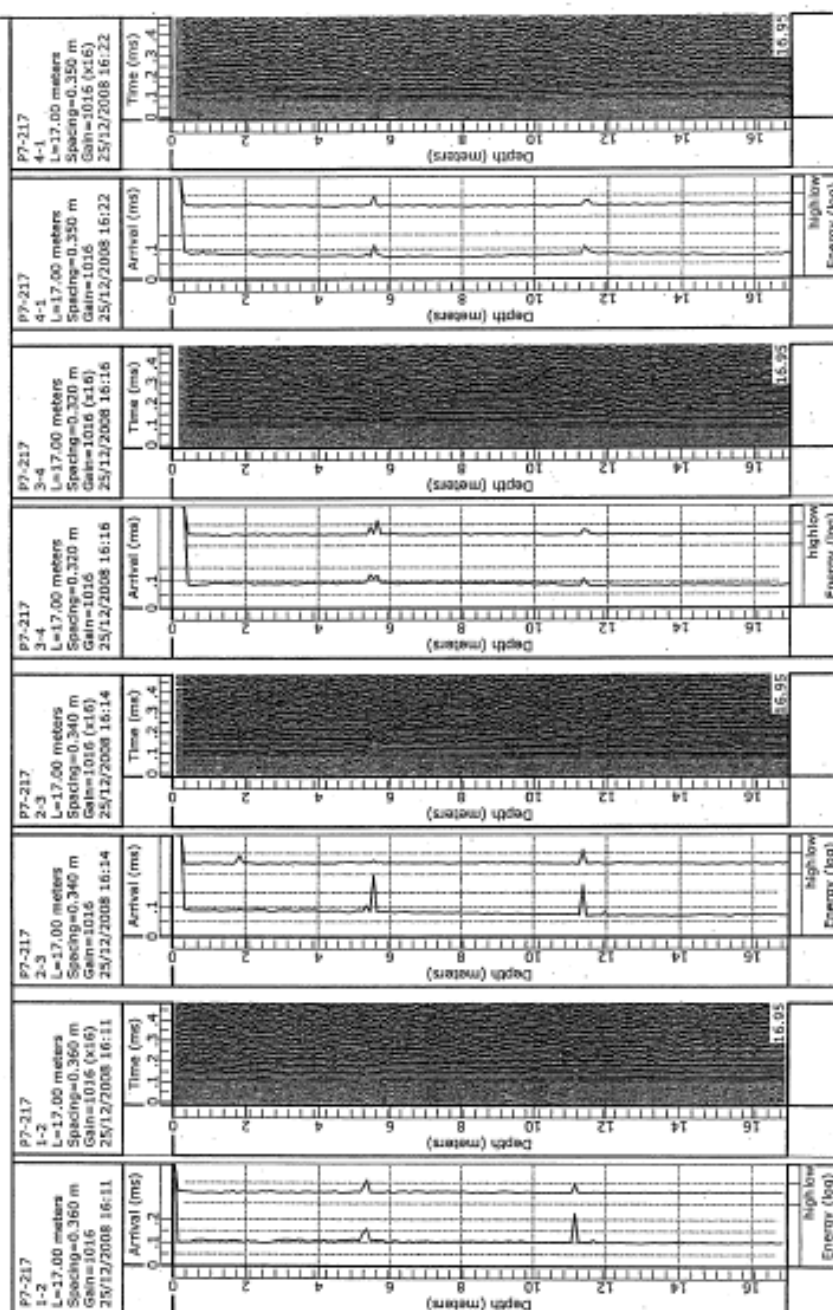


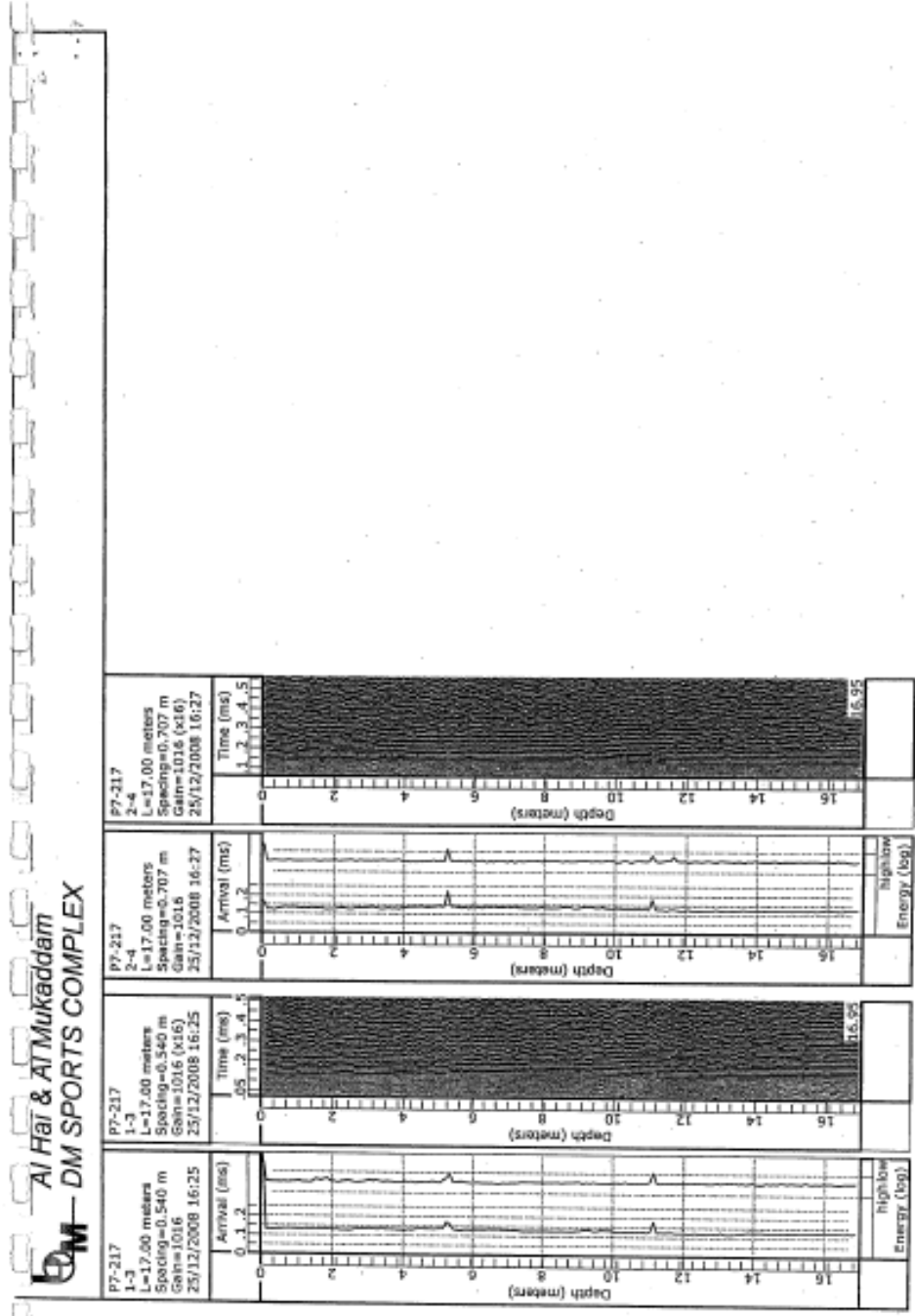


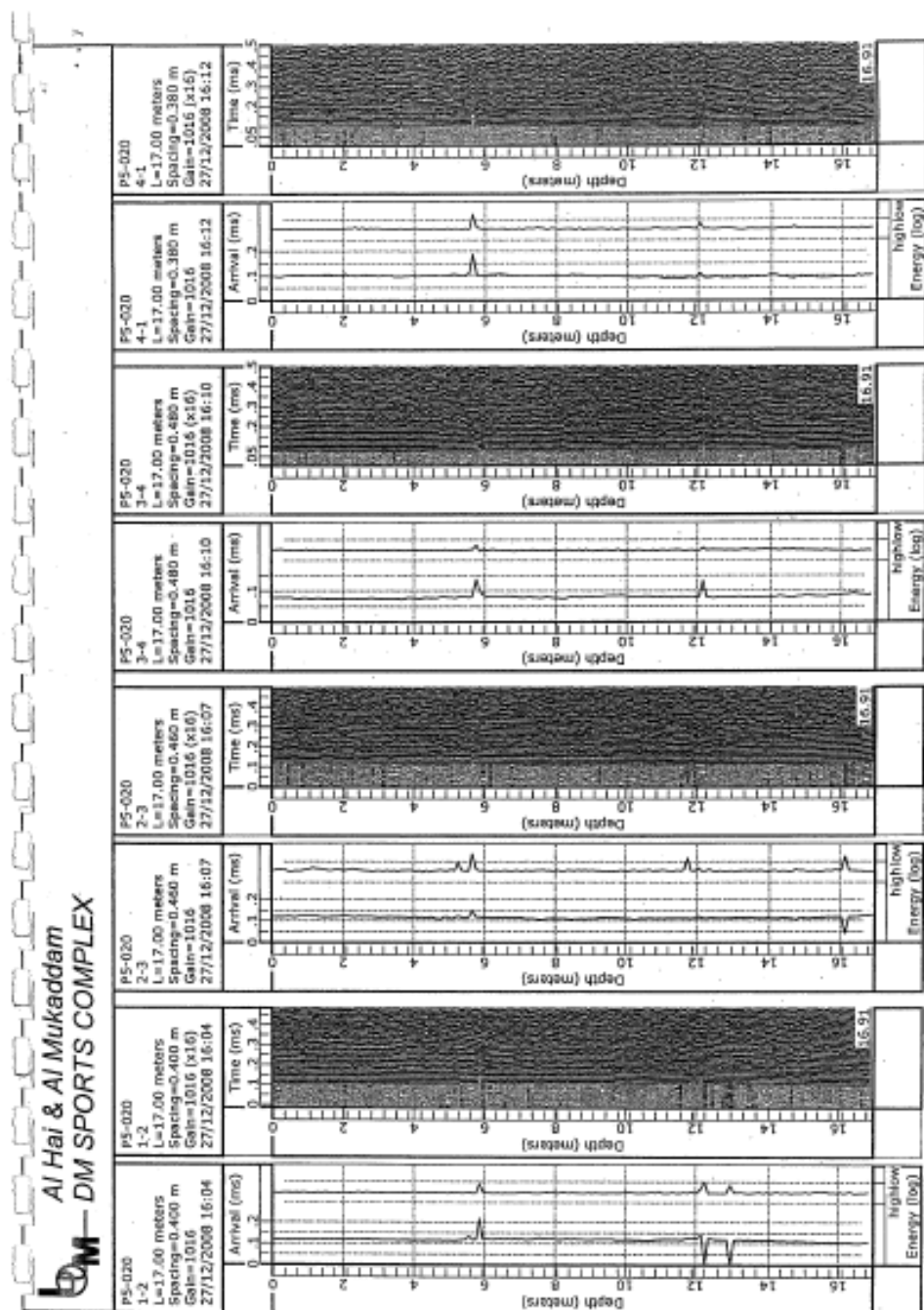




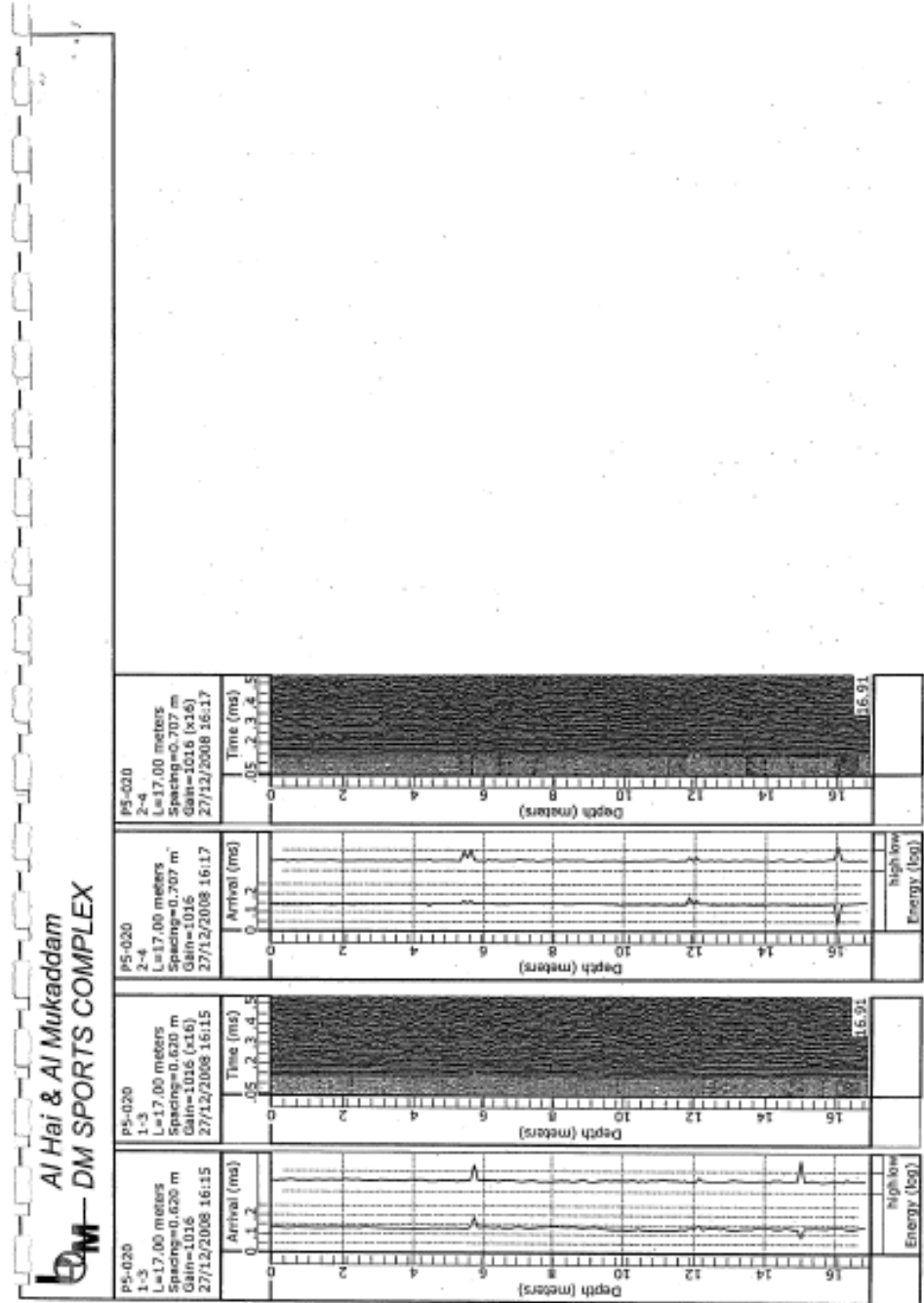
Al Hai & Al Mukaddam
DM - DM SPORTS COMPLEX

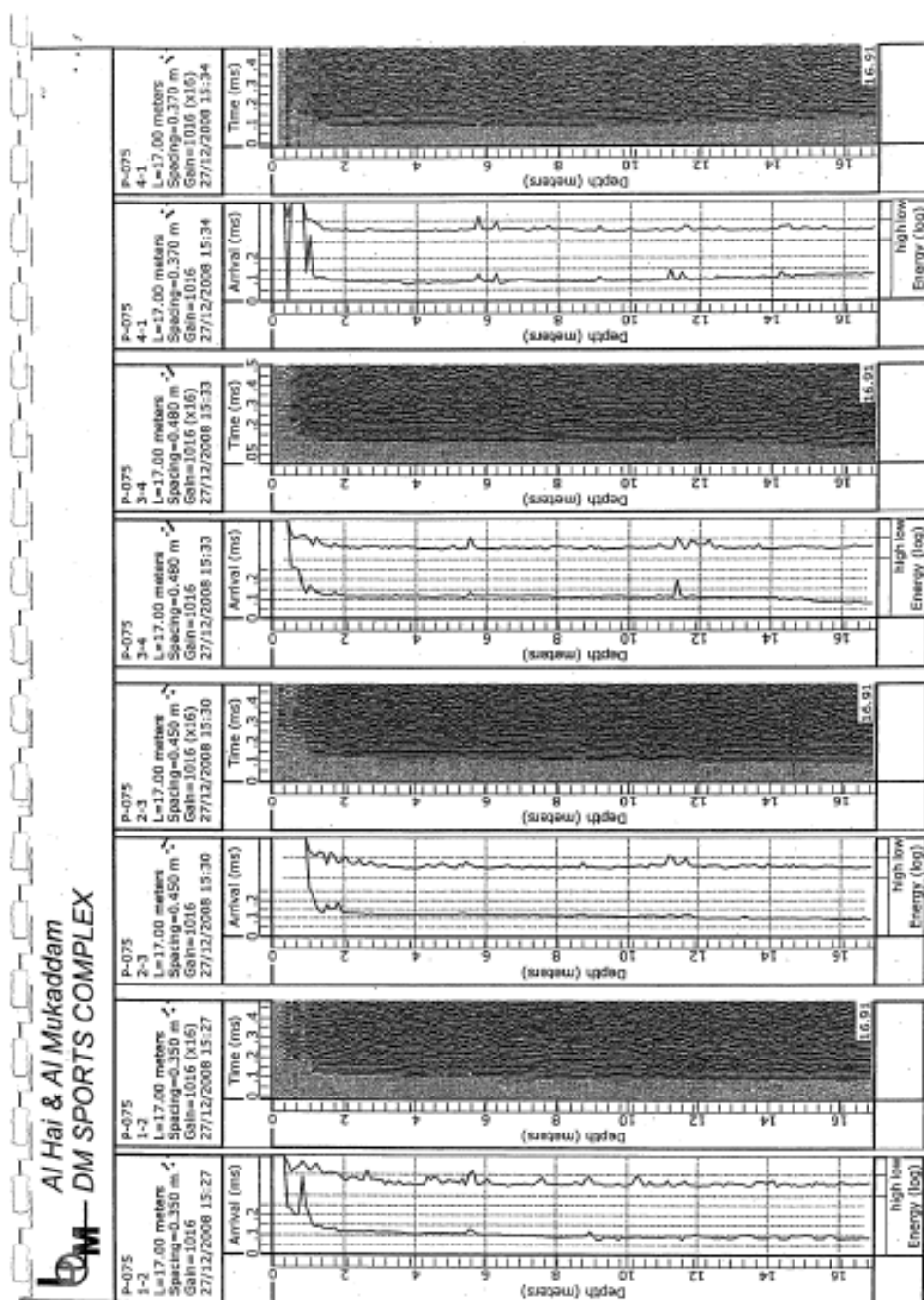


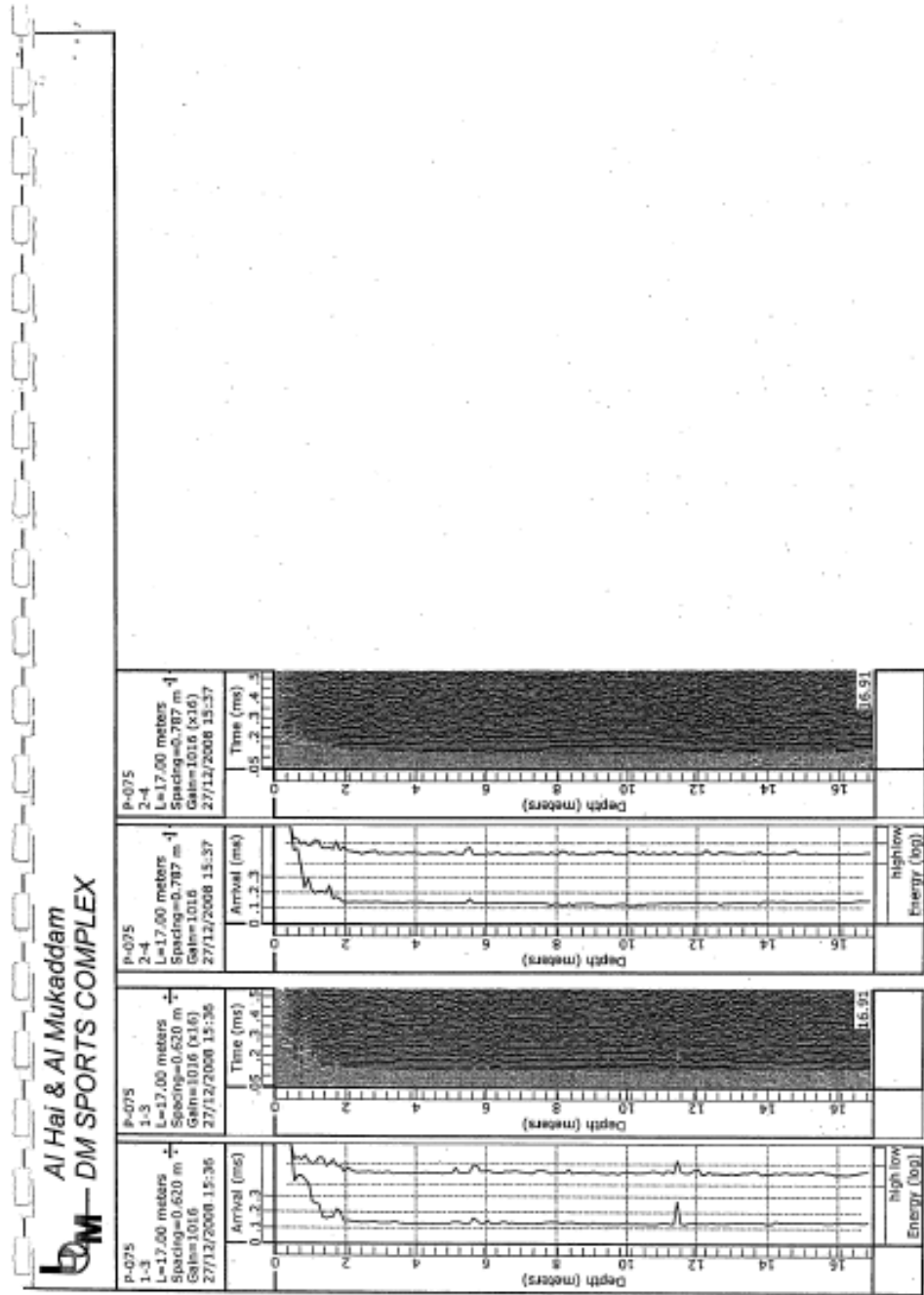




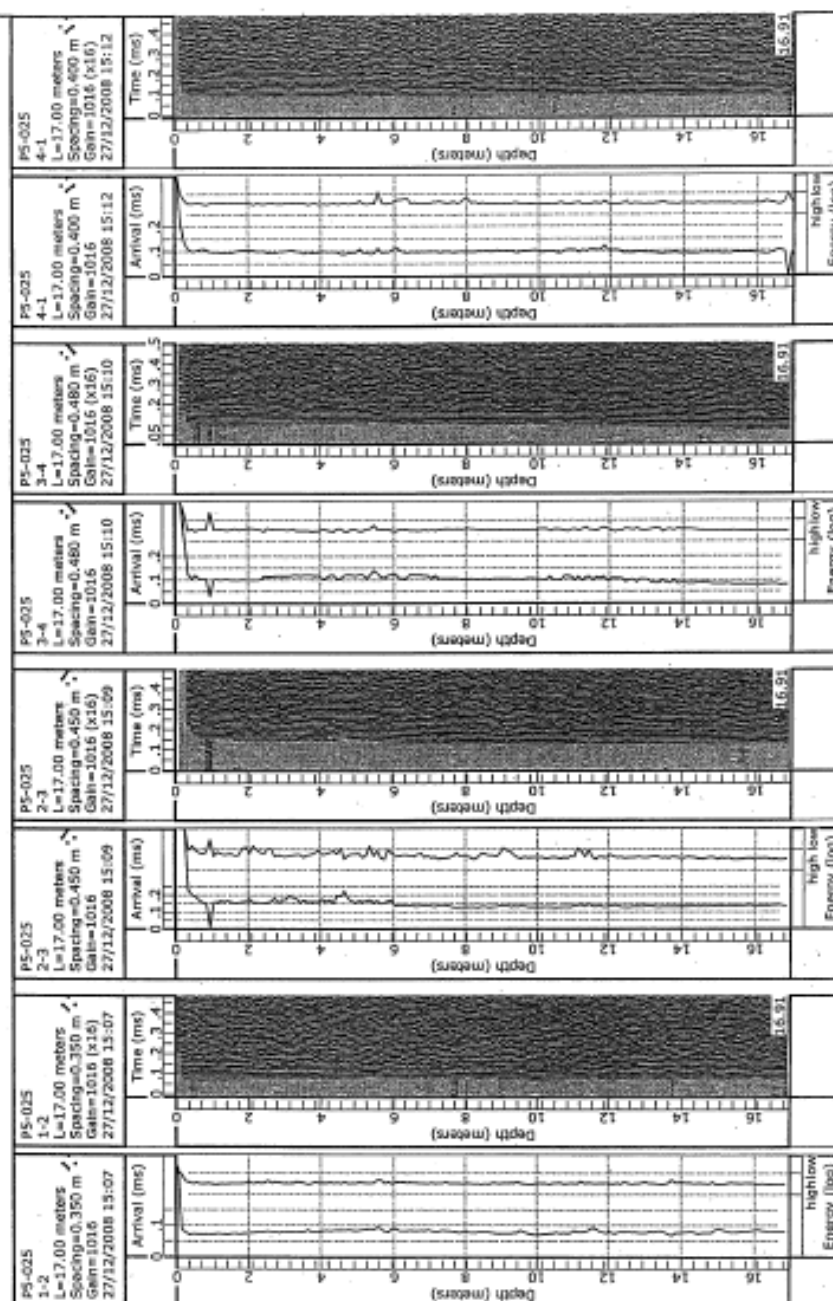
File Dynamics, Inc.
Cross-Hole Analyzer

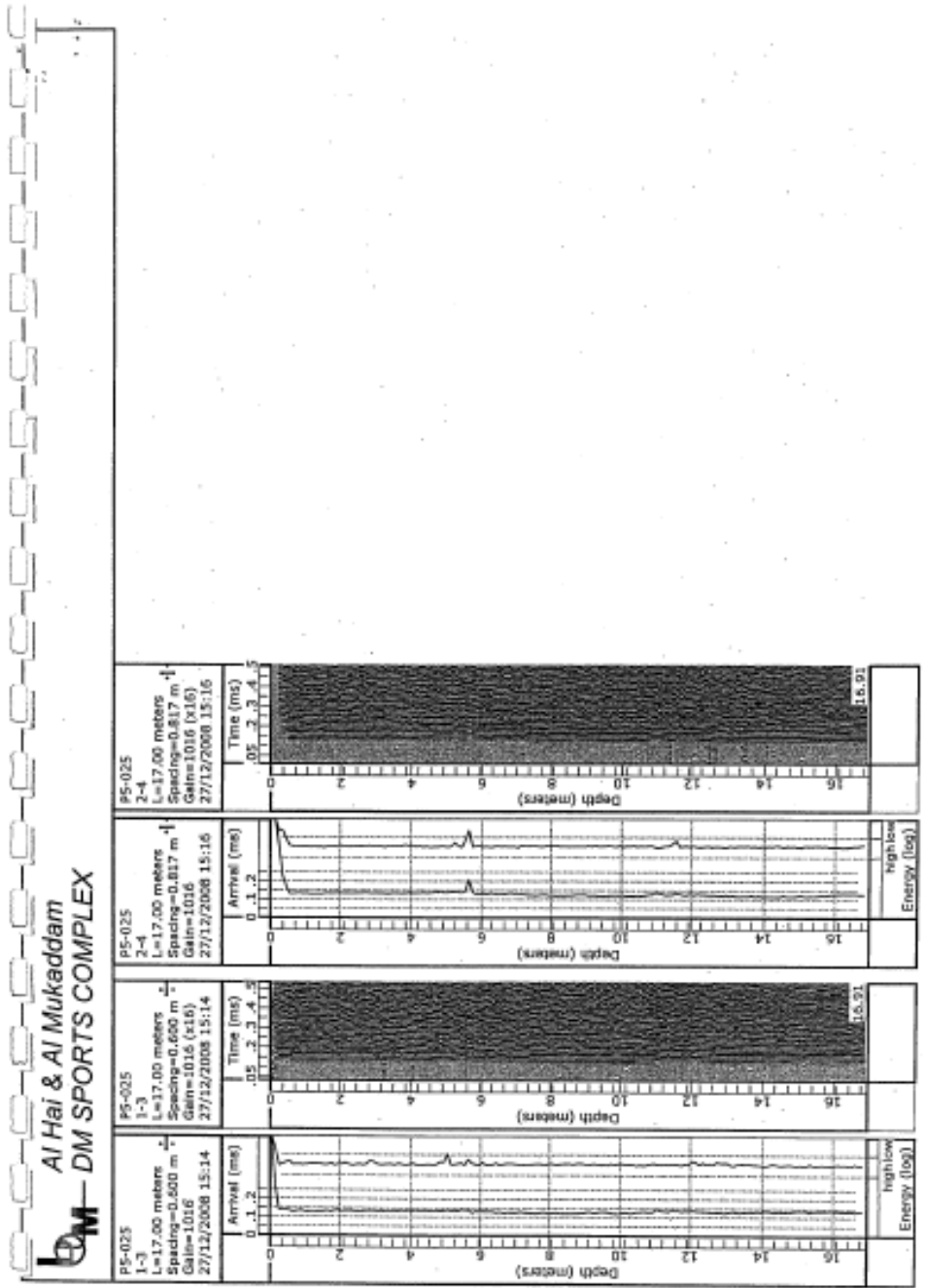




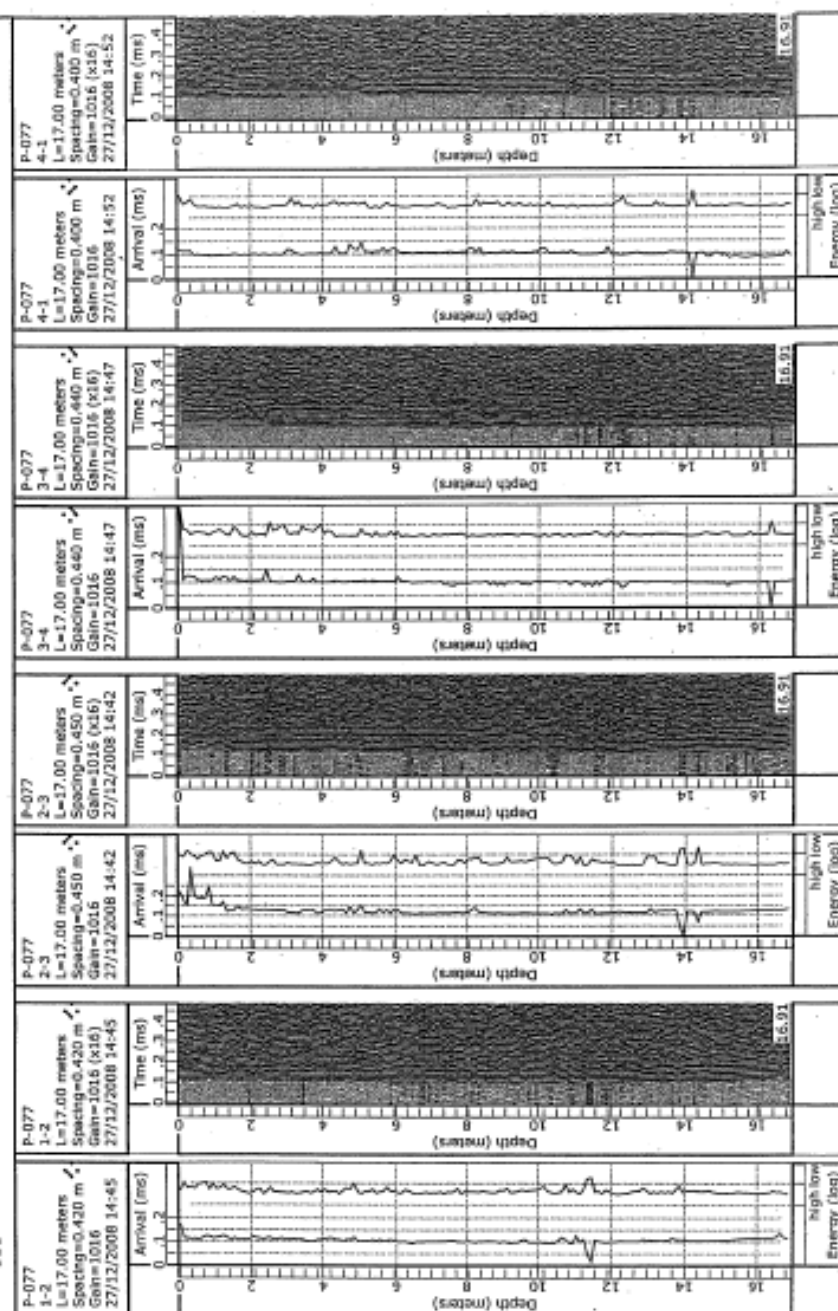


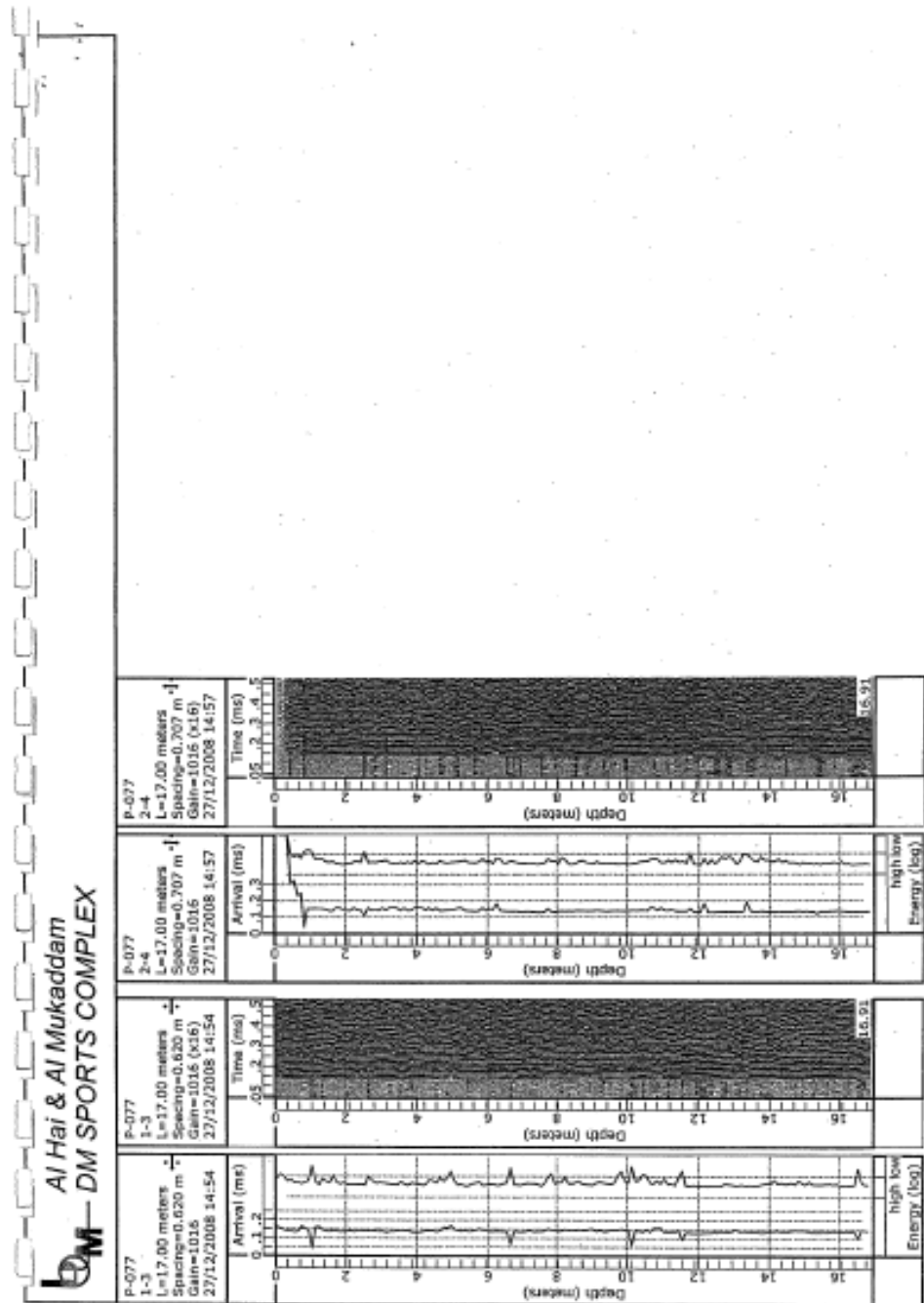
AI Hai & Al Mukaddam
DM - DM SPORTS COMPLEX



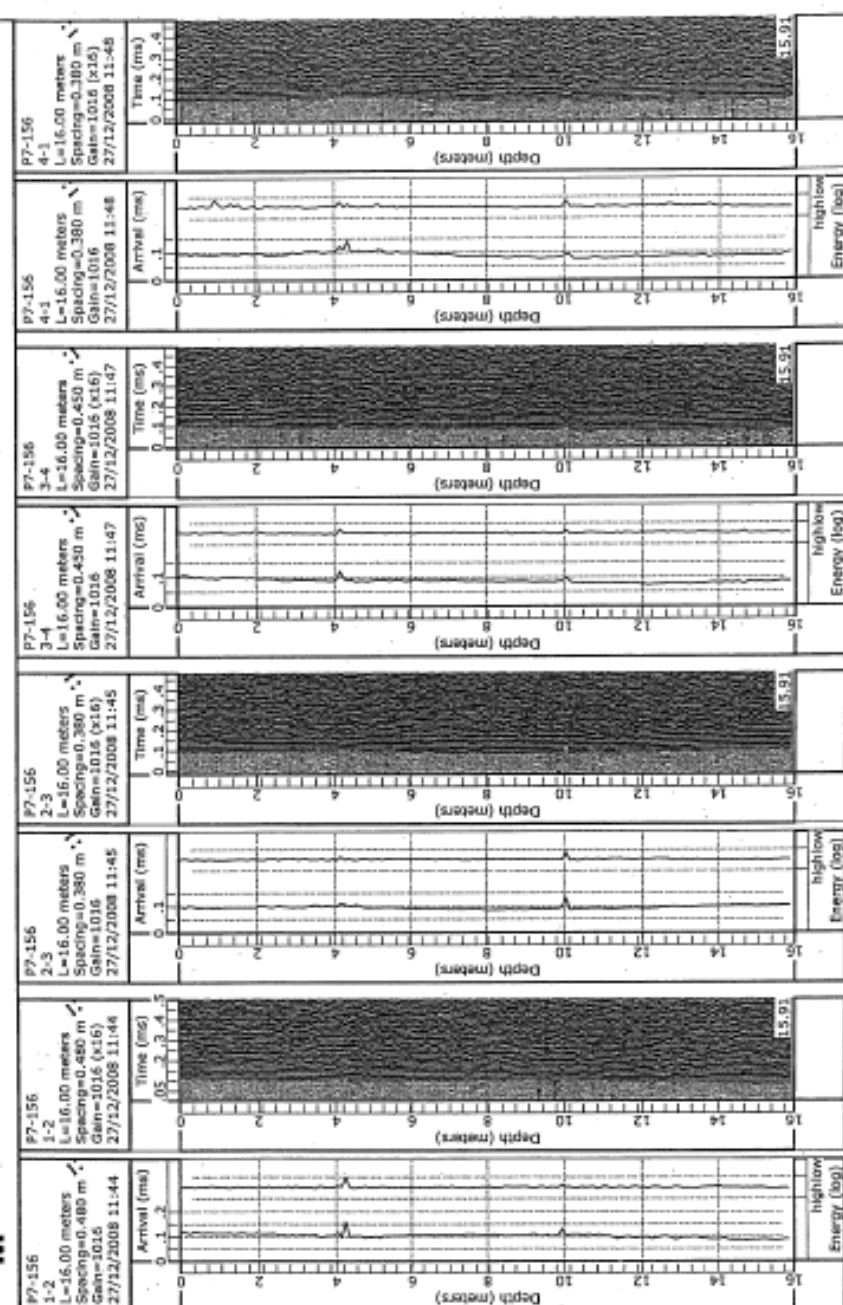


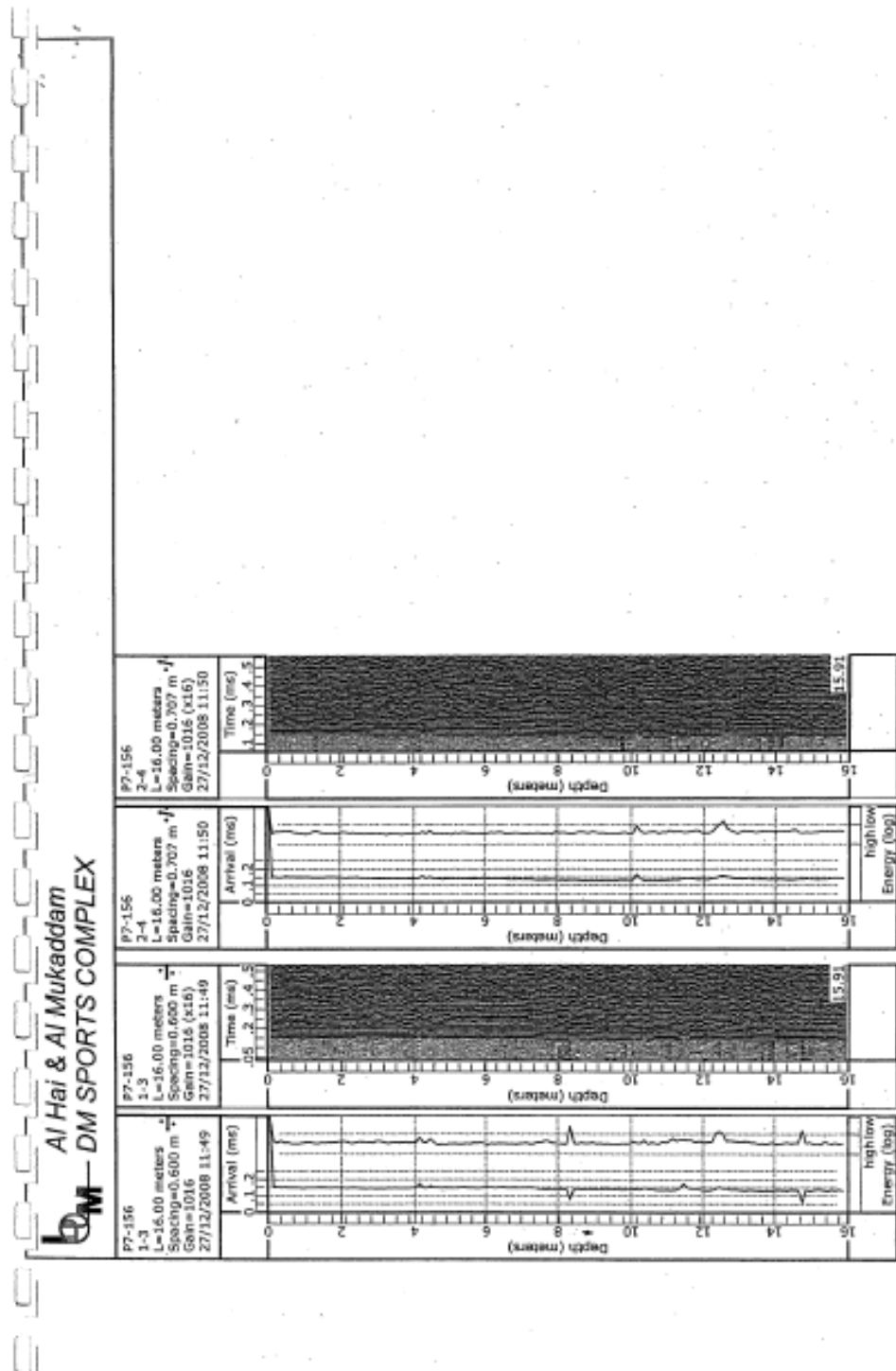
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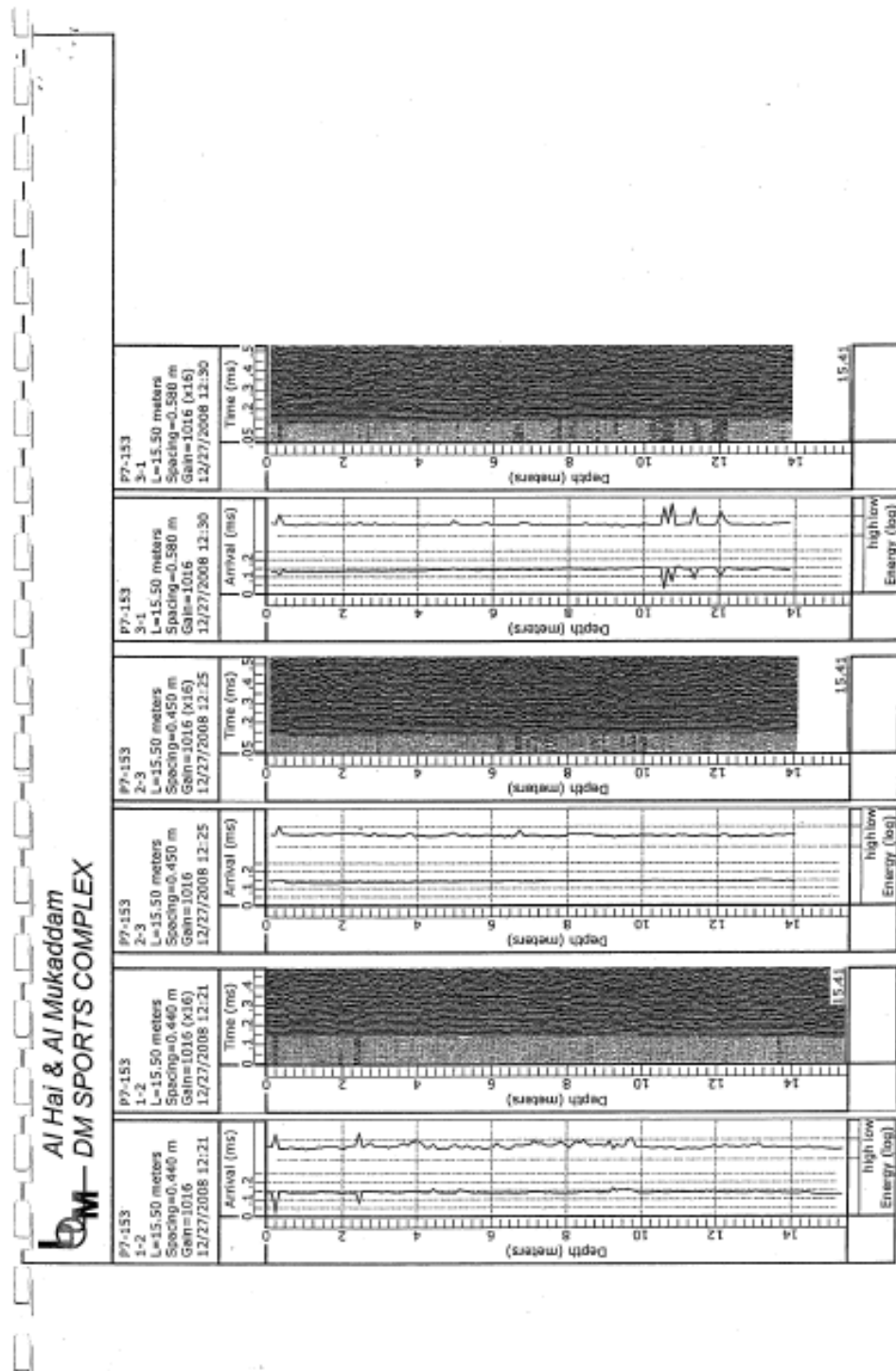




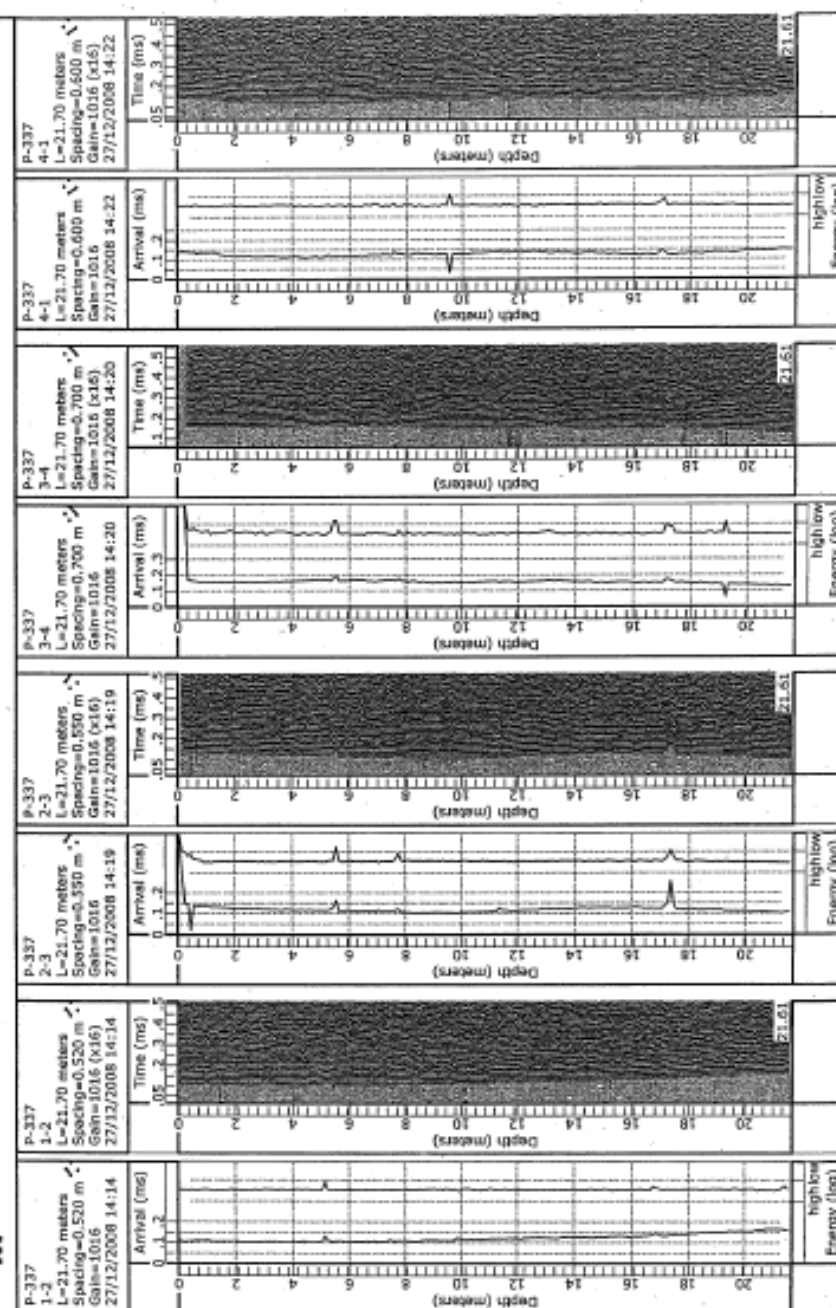
Al Hai & Al Mukaddam DM - DM SPORTS COMPLEX



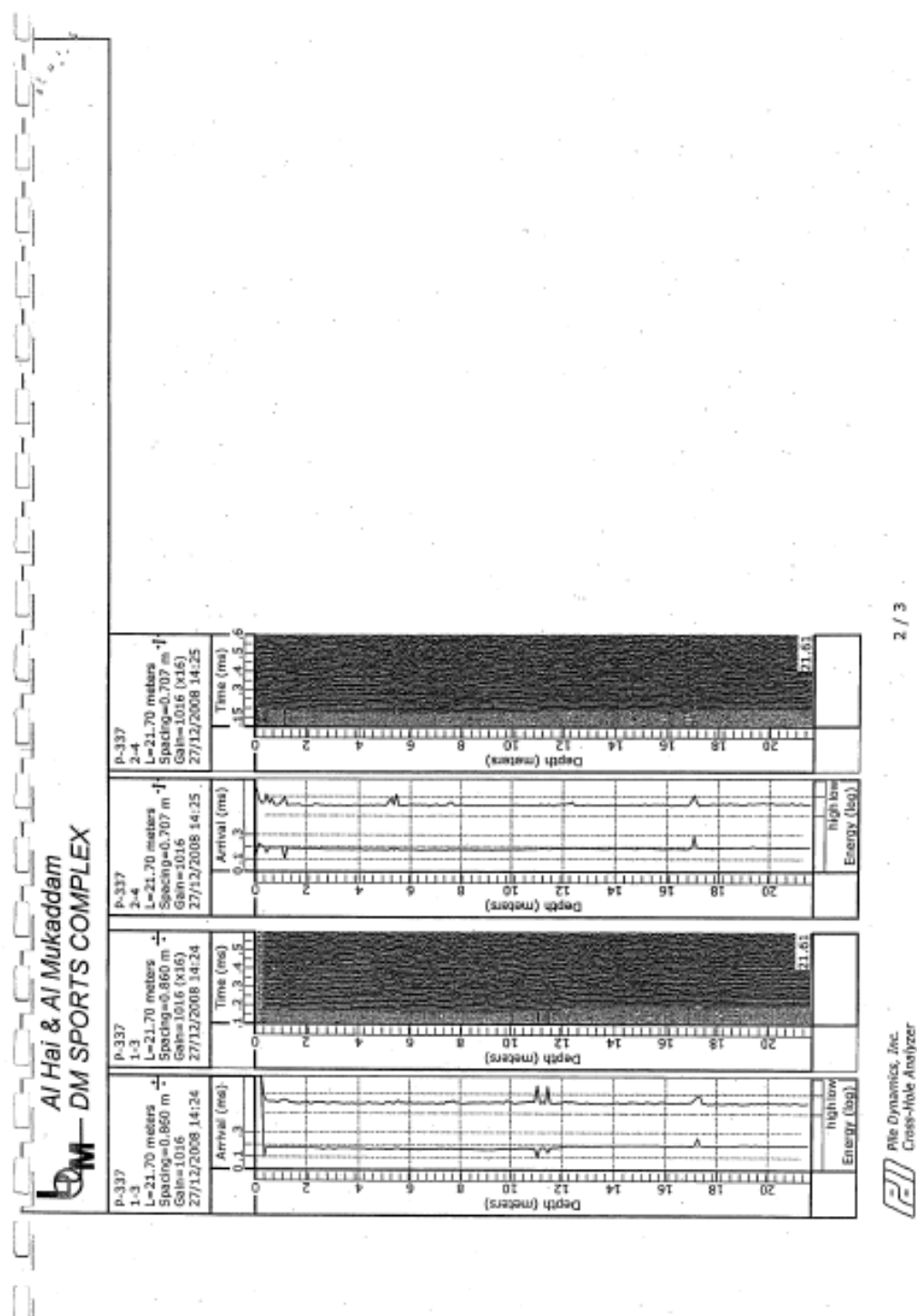




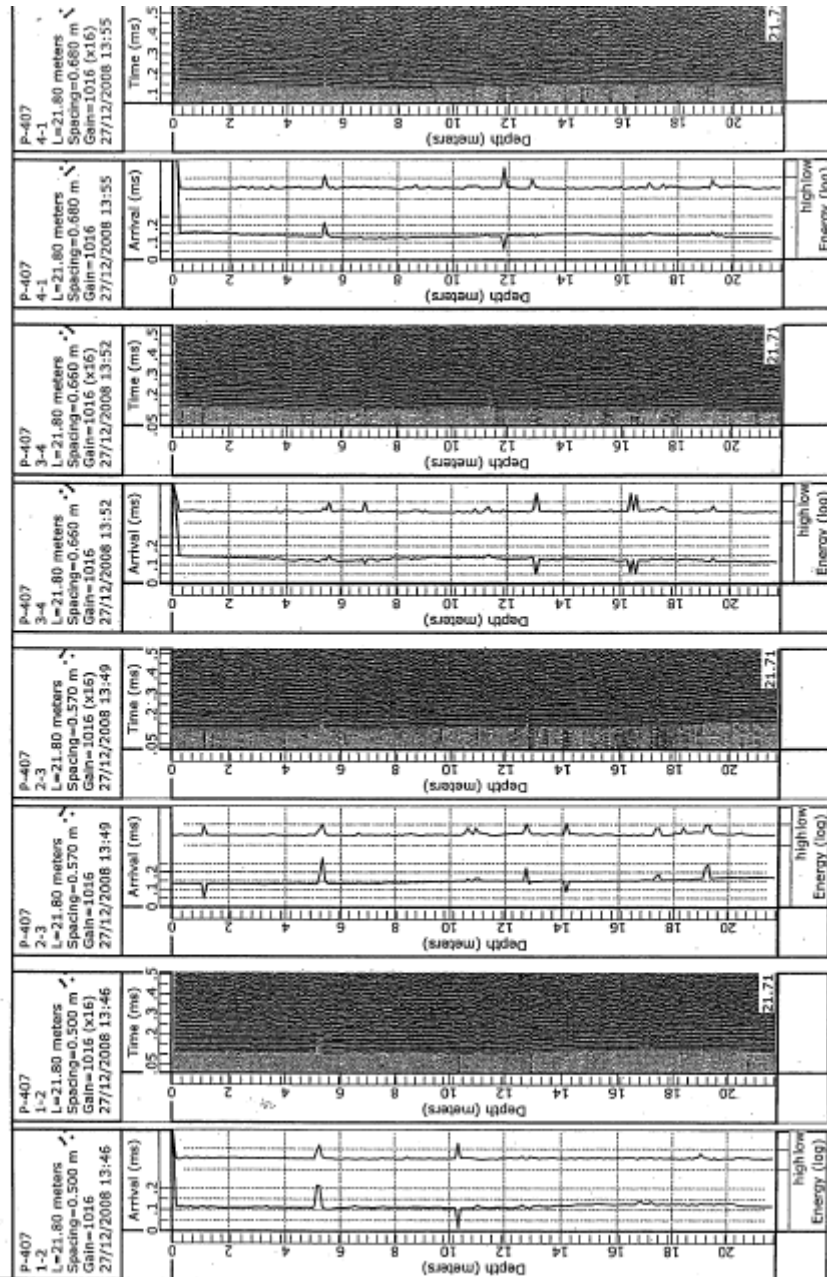
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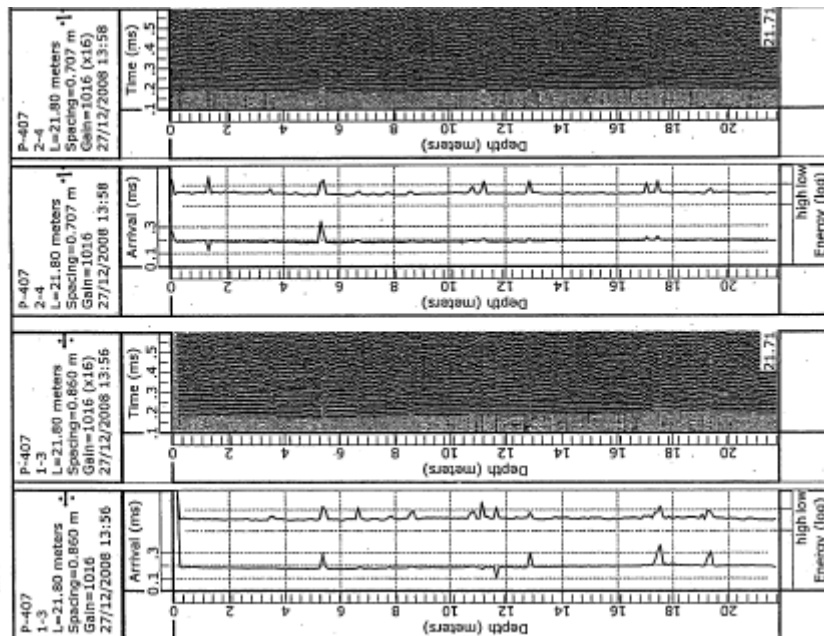
Pile Dynamics, Inc.
Cross-Hole Analyzer



Al Hai & Al Mukaddam
DM - DM SPORTS COMPLEX



**Al Hai & Al Mukaddam
DM-DM SPORTS COMPLEX**

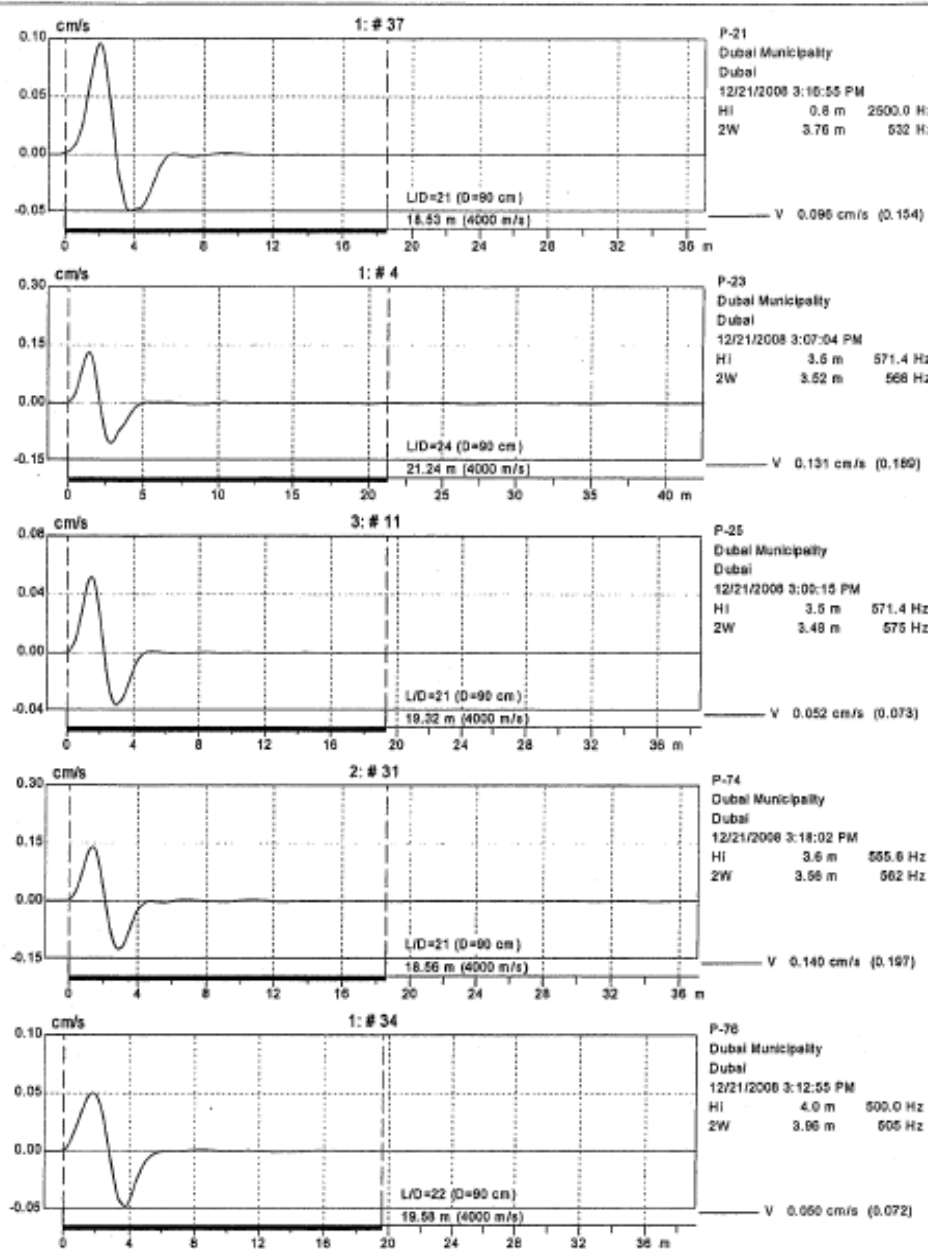


APPENDIX G: CASE STUDY 4 – LOW STRAIN INTEGRITY TESTING WITH PULSE-ECHO METHOD

Al Hai & Al Mukaddam
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12/22/2008

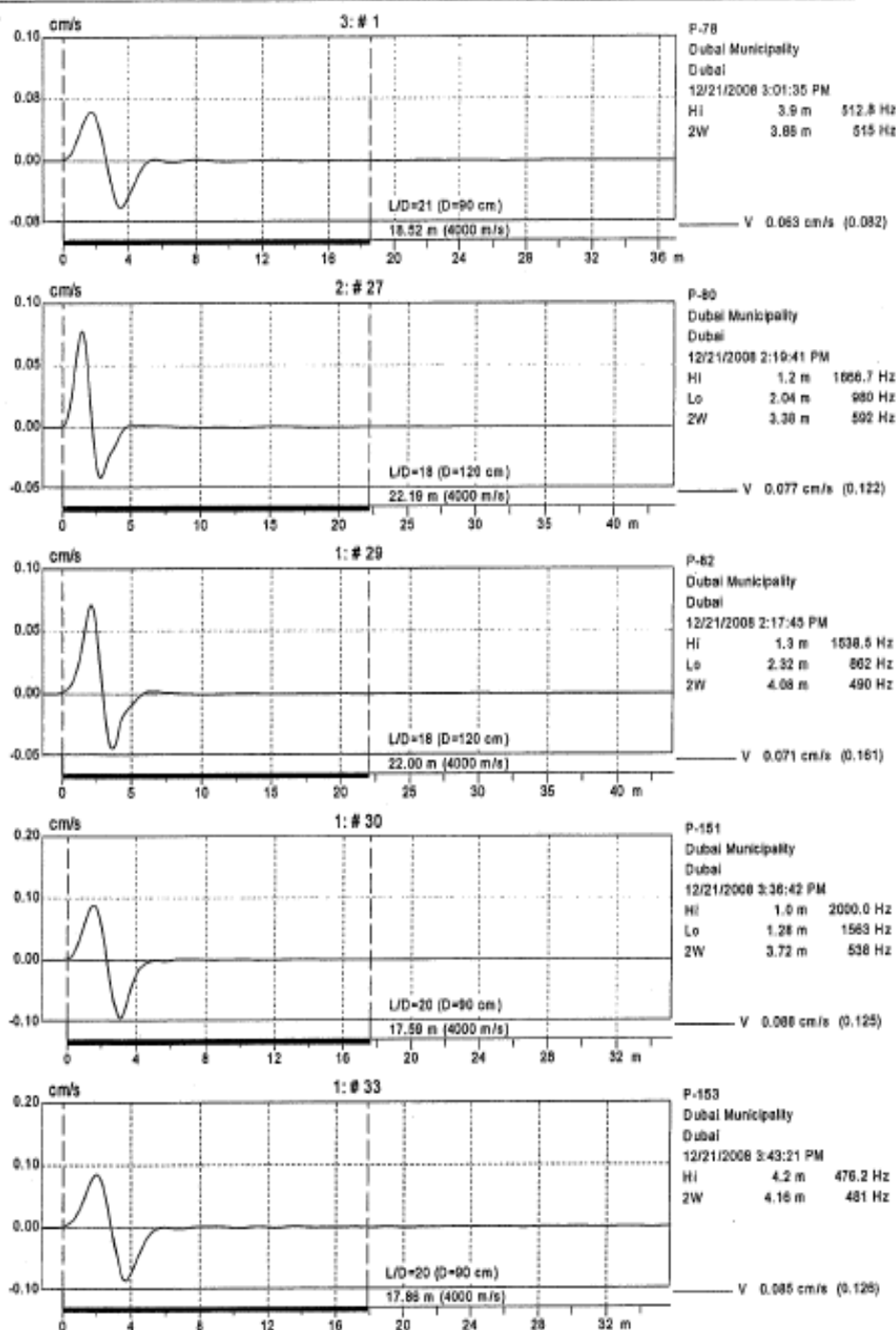
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Al Hai & Al Mukaddam
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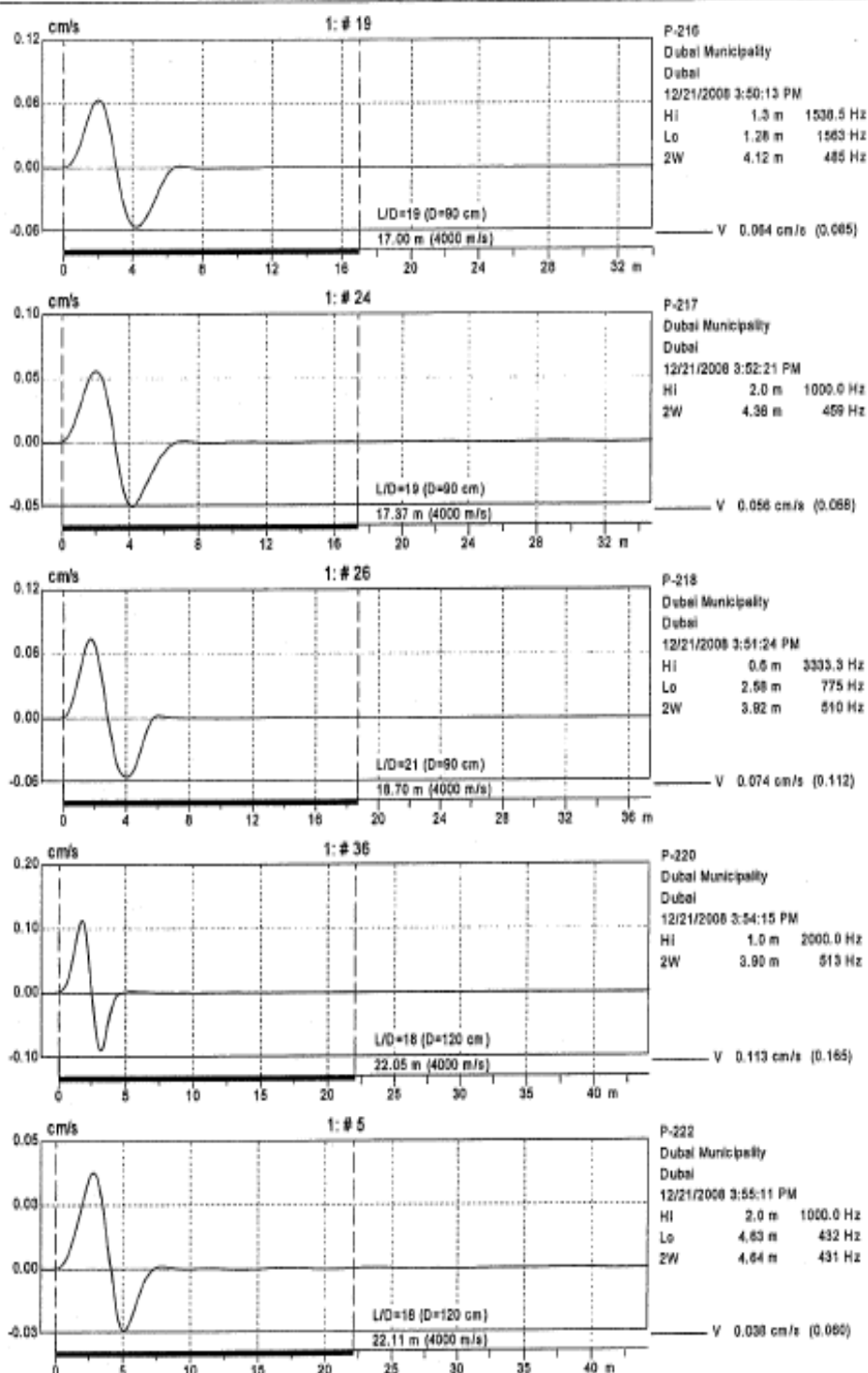
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Al Hai & Al Mukaddam
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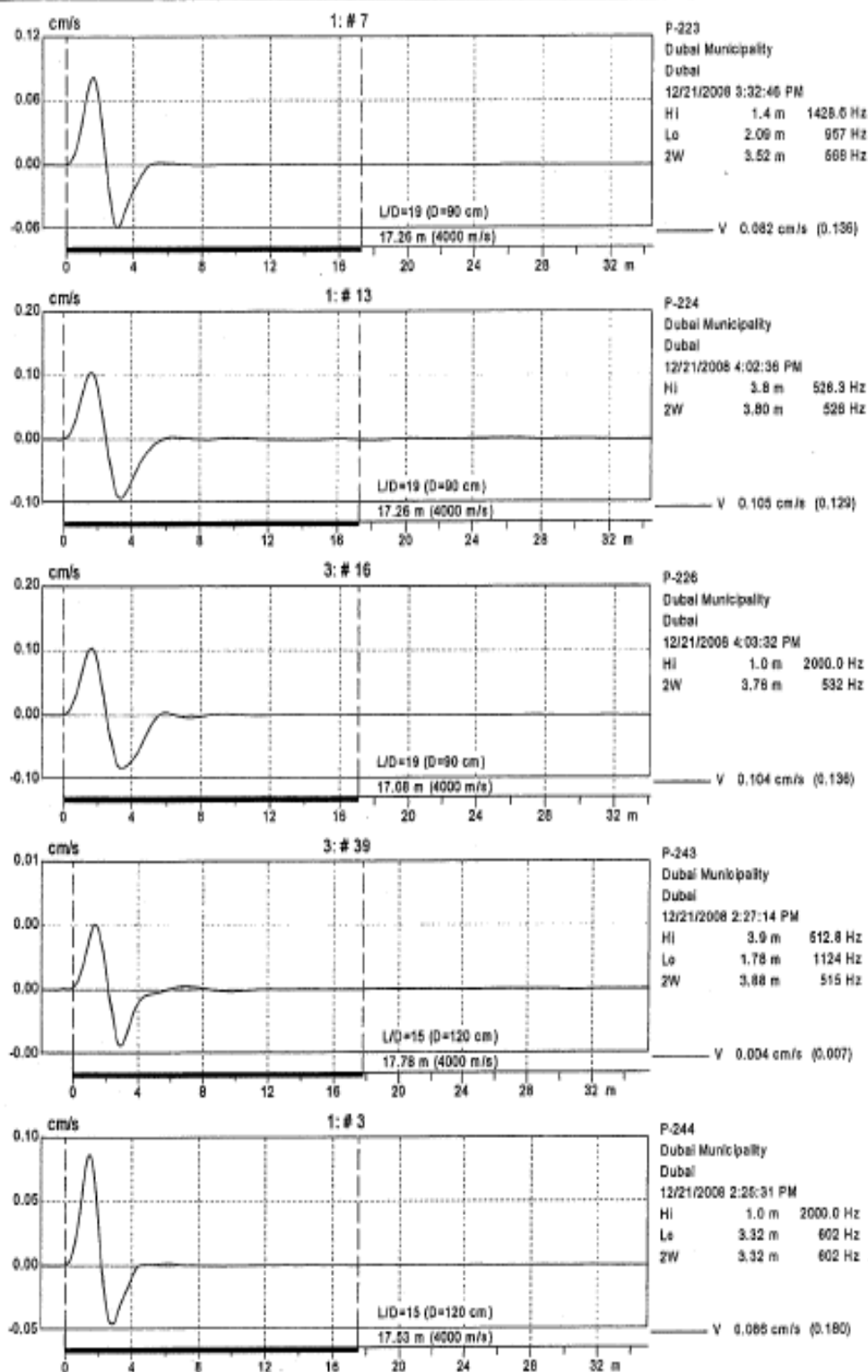
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Al Hai & Al Mukaddam
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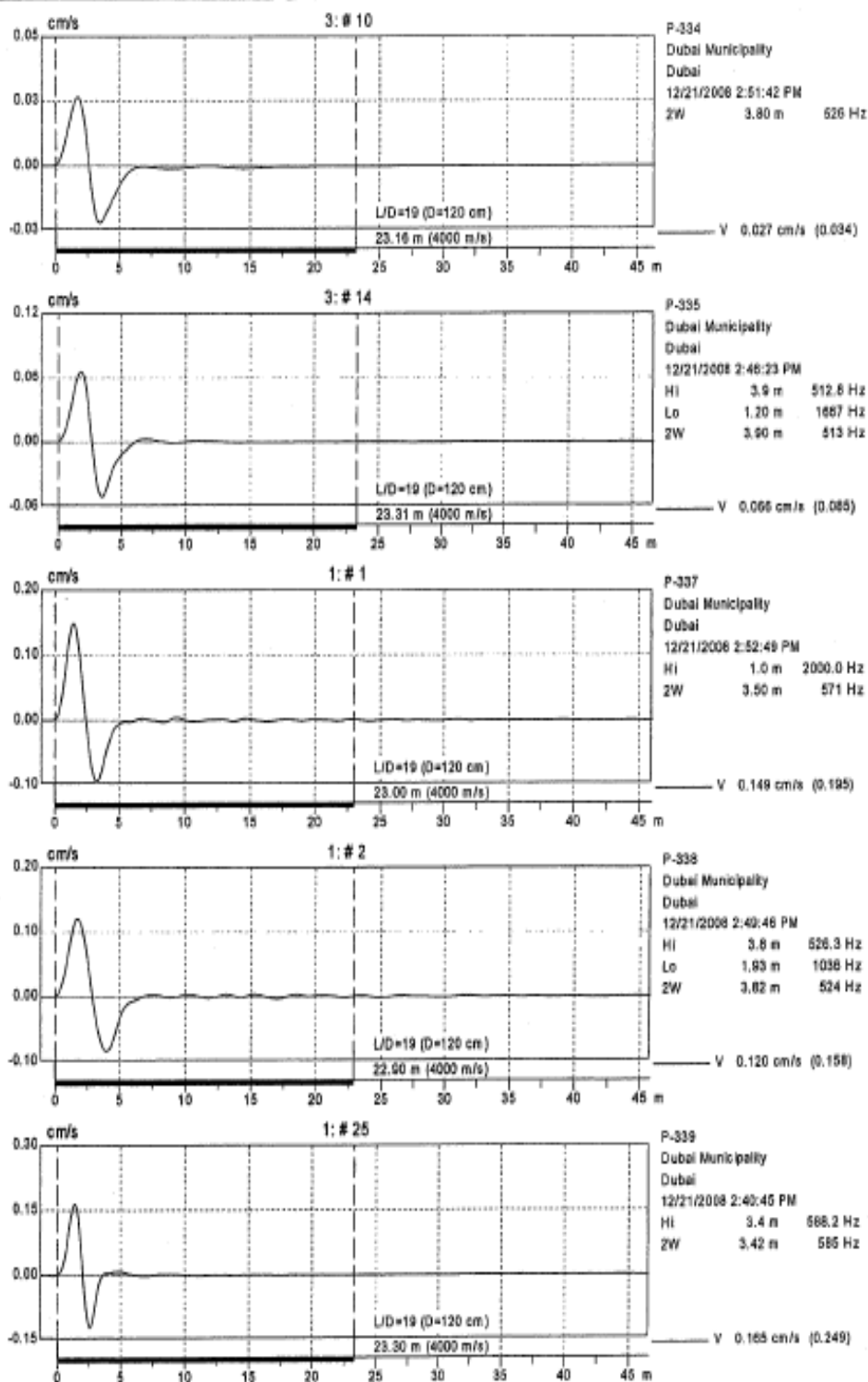
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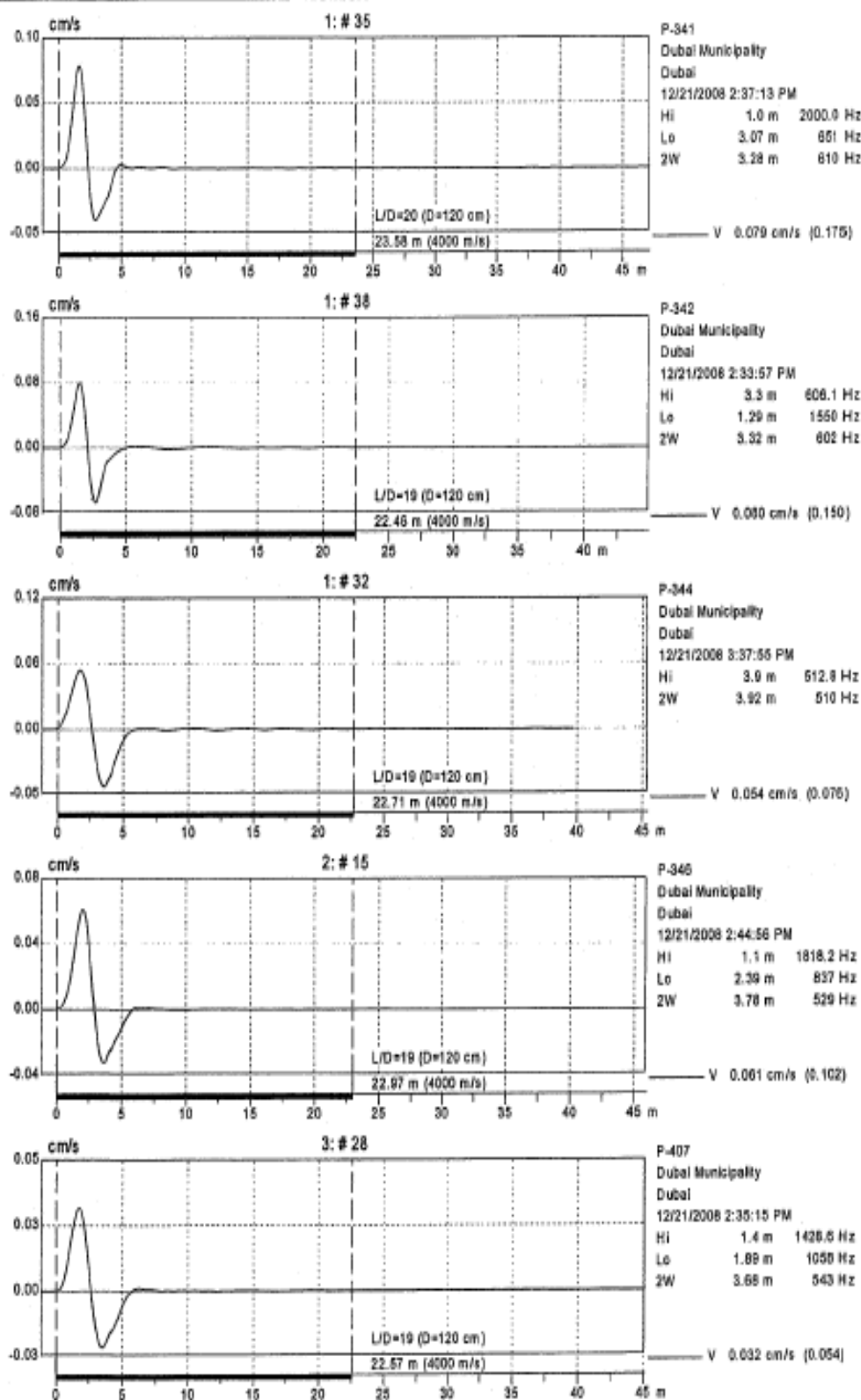
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Al Hai & Al Mukaddam
DUBAI SPORTS COMPLEX
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PIT-W™ 2003-2



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