APPLICATION OF THE WAVE EQUATION ANALYSIS TO PILE DRIVING

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By

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ABSTRACT

The methods of analysis used to determine ultimate bearing capacity of piles are applied to a field case in Turkey. During the pile driving works of the Atatürk Culture Center in Samsun, Delmag D22-13 and Delmag D-15 diesel pile driving hammers have been used. The ultimate bearing capacity and skin friction values calculated using static formulas have been given as input to the wave equation pile analysis program FADWAVE written by J.E.Bowles. The analytical method used in this program was based on the wave propogation in solids originated in 1962 by E.A.L.Smith. The results obtained from the program are compared with each other.

Keywords : Ultimate bearing capacity, wave equation pile analysis, drivability.

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DALGA DENKLEM ANALİZİNİN KAZIK ÇAKMA İŞLEMİNE UYGULANMASI

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ÖZET

Kazıkların yük taşıma kapasitesinin belirlenmesi için kullanılan metodlar Türkiyedeki bir arazi durumuna uygulanmıştır. Samsun Atatürk Kültür Merkezinin kazık çakma işlemleri icin Delmag D22-13 ve Delmag D-15 dizel şahmerdan tokmakları kullanılmıştır. Statik formüller kullanılarak elde edilen yük taşıma kapasiteleri ve sürtünme değerleri J.E.Bowles tarafından yazılan FADWAVE adındaki kazıklarda dalga denklem analiz programına girdi olarak verilmiştir. Bu programda kullanılan analitik metod katı cisimlerde dalga dağılımı prensibi ile 1962 yılında E.A.L.Smith tarafından ortaya konmuştur. Programdan elde edilen sonuçlar birbirleriyle karşılaştırılmıştır.

Anahtar Sözcükler : Yük taşıma kapasitesi, kazıklarda dalga denklem analizi, çakılabilirlik.

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A	pile cross-sectional area
E	modulus of elasticity
e_h	hammer efficiency (1.0 is accepted in the thesis)
E_h	manufacturers hammer-energy rating
k_1	elastic compression of capblock and pile cap and is a form of P_uL/AE
k_2	elastic compression of pile and is of a form of P_uL/AE
k_3	elastic compression of soil, also termed quake for wave-equation analysis
L	pile length
n	coefficient of restitution
P_u	ultimate pile capacity
s	amount of point penetration per blow also termed set
W_p	weight of pile including weight of pile cap, driving shoe and capblock $[W_p^* + W_{pc}]$
W_p^*	pile weight
W_{pc}	weight of pile cap
W_r	weight of ram
h	height of the fall of the ram of the pile hammer
v_1	velocity of the pile cap

P_u^* ultimate test load.

P_d design capacity, using the safety factor recommended for the equation (values range from 2 to 6, depending on the formula).

SF true safety factor [$SF = P_u^* / P_d$]

Q_{ult} ultimate bearing capacity

Q_p point resistance

Q_s skin resistance

τ_f shear strength

c' soil cohesion

ϕ' angle of internal friction

D_r relative density

P_T effective vertical stress at pile tip

N_q bearing capacity factor

A_T area of pile tip

D diameter of the pile

q_{ult} ultimate point resistance of driven pile

$\bar{N}$ average standard penetration resistance corrected for the effective overburden stress at pile tip (blows/ft)

N Standard Penetration Resistance near pile tip (blow/feet)

C_N correction factor for $\bar{N}$

p effective overburden stress at pile tip

E_d	depth driven into granular bearing stratum (ft)
q_1	limiting point resistance (TSF), equal to 4N for sand and 3N for non-plastic silt.
N_{pt}	uncorrected average SPT blow count over a distance of 4 diameter either side of the pile point
K_{HC}	Ratio of horizontal to vertical effective stress on side of element when element is in compression (K_{HC} = 1 to 2 for sand)
P_o	effective vertical stress over embedment length
δ	friction angle between pile and soil
s	surface area of pile per unit length
A_s	pile shaft area (πDL)
E_1	embedment length of the pile
f_s	ultimate skin resistance for driven pile (TSF)
f_1	limiting skin resistance (for driven pile, f_1=1 TSF)
N_{side}	uncorrected average SPT blow count within the shaft length considered
C_m	relative displacement between two adjacent pile elements
D_m''	element displacement two time intervals back
D_m'	element displacement in the preceding time interval DT
D_m	current element displacement

DT	time interval
F_m	element force
F_{am}	unbalanced force in element causing acceleration
g	gravitation constant
J	damping constant
K_m	element springs = AE/L for pile segments
K_m'	soil springs = R/quake
R_m	side or point resistance including damping effects
R_m'	amount of estimated Pu on each element
v_m	velocity of element m at DT
v_m'	velocity of element m at DT-1
W_m	weight of pile segment m
U.S.C.	unified soil classification
SP	poorly graded sands, gravely sands with little or no fines
CH	inorganic clays of high plasticity
W_n	natural water of a soil relative to the liquid and plastic limits
LL	liquid limit
PL	plastic limit
PI	plasticity index
N.P.	non-plastic

CHAPTER 1

INTRODUCTION

It is well known that unless a certain amount of engineering judgement based on experience is used, results obtained up to recently from methods of analysis used to determine Ultimate Pile Load Capacity of piles driven in sand vary in broad range.

Presently, available methods of analysis can be grouped in three (3) main categories, which are in the historical order:

- dynamic formulas,
- static formulas,
- application of the wave equation to piles.

Starting from the 1950 's attempts have been made to compare and correlate the results obtained from dynamic and static formulas with that obtained from pile loading tests.

But these attempts were not satisfactory as expected. A new approach was the Wave Equation Pile Analysis based on the Wave Propagation in Solids originated in the 1960 's by E.A.L.Smith [34]. Today versions based on the same theory exist such as the TTI, CAPWAPC, WEAP, FADWAVE programs. ([11], [7])

The scope of this study is to apply the methods of analysis available to the engineer to a field case in Turkey and to compare the results obtained.

In the field case selected for this study, the sand layer into which the piles were driven is possessing relatively uniform geotechnical characteristics.

For this project of limited size no pile loading test had been foreseen in view of its cost but before starting to the pile driving works of the Atatürk Culture Center in Samsun (Samsun Atatürk Kültür Merkezi) twenty pile driving tests, have been performed using two different pile driving hammers: Delmag D22-13 and Delmag D-15 diesel pile driving hammers in order to experimentally determine the compatibility between the pile and the driving hammer, to come to a decision as regards to the selection of an appropriate combination. [42]

Piles used in the driving tests are centrifugally spun 12-meter long hollow cylindrical reinforced concrete piles with 10 cm wall thickness and 40 cm outside diameter. **[42]**

The wave equation of pile analysis is an attempt towards the simulation of the pile driving process. The computer program entitled FADWAVE used here in this study has been written by J.E.Bowles (1988). **[7]**

The FADWAVE program requires as input the Ultimate Pile Load Capacity (RUTOT) which can be obtained by Pile Loading Tests or alternatively a parametric study can be made using variable values of Ultimate Pile Load Capacity as inputs.

To overcome the situation Ultimate Bearing Capacity of the piles have been calculated using static formulas to determine the Skin Resistance and Point Resistance and give these results as input into the FADWAVE program.

Before using the FADWAVE computer program for the Samsun AKM field case a check has been done with a well documented case existing in the literature. **([36], [37], [38])** The details, evaluation and interpretation of the results obtained and conclusions thereof, are presented in the following sections.

CHAPTER 2

DETERMINATION OF ULTIMATE PILE LOAD CAPACITY WITH DYNAMIC FORMULAS

When the history of foundation construction is investigated it can be seen that until the twentieth century, the design of foundations were based entirely on past experience, ancient rules, and guesswork.

In the beginning piles used were made of wood [35], later on starting from the turn of the century steel piles and reinforced concrete piles have been used. The pile lengths as short as 3 meters to 6 meters were used. Today 15 meters to 20 meters long piles are common and even longer piles are used in special projects. Pile-bearing failure due to the hammer blows gained more importance therefore a relationship between pile capacity and the resistance offered to driving has to be developed.

Dynamic formulas were and are still being used to predict the ultimate capacity of a pile based on data obtained during the pile driving operation but none of the equations are consistently reliable over an extended range of pile capacity.

As pile lengths increased the results varied a lot, therefore to come to a decision results of different types of formulas have to be tested.

" Some means is needed in the field to determine when a pile has reached a satisfactory bearing value other than by simply driving it to some predetermined depth. Driving the pile to a predetermined depth may or may not obtain the required bearing value because of normal soil variations both laterally and vertically.

It is generally accepted that the dynamic formulas do not provide very reliable predictions. Predictions tend to improve by using a load test in conjunction with the equation to adjust the input variables. " **[6]**

Many attempts have been made to improve the reliability of the dynamic formulas; one of the most comprehensive pile-testing program undertaken was the one made under the direction of the Michigan State Highway Commission (1965) **[6]**. This study found that the true safety factors for some dynamic formulas based on pile-load tests are as indicated in Table 2.1.

Table 2.1 True safety factors for some dynamic formulas

Formula	Upper and lower limits of SF = P_u^*/P_d — Range of P_u^*, kips: 0-200	200-400	400-700
Engineering News	1.1-2.4	0.9-2.1	1.2-2.7
Hiley	1.1-4.2	3.0-6.5	4.0-9.6
Rankine	0.9-1.7	1.3-2.7	2.3-5.1
Modified Engineering News	1.7-4.4	1.6-5.2	2.7-5.3
Gates	1.8-3.0	2.5-4.6	3.8-7.3

P_u^* = Ultimate Test Load.
P_d = Design Capacity, using the safety factor recommended for the equation (values range from 2 to 6, depending on the formula).

The statistical analysis performed by Olson and Flaate (1967) **[6]** on some 93 other piles have concluded that the Hiley equation and the Janbu and Gates formulas were with the least deviations or highest statistical correlations.

In the study, pile driving test results have been evaluated by three (3) dynamic formulas proposed by :

- Janbu,
- Gates,
- Hiley

which are given in Table 2.2. **[6]**

Table 2.2 Dynamic formulas

Janbu Formula [6]

$$P_u = \frac{e_h\ E_h}{k_u\ s} \qquad C_d = 0.75 + 0.15\ \frac{W_p}{W_r}$$

$$k_u = C_d\left(1 + \sqrt{1 + \frac{\lambda}{C_d}}\right) \qquad \lambda = \frac{e_h\ E_h\ L}{A\ E\ s^2}$$

Gates Formula [6]

$$P_u = a\sqrt{e_h\ E_h}\ (b - \log s) \qquad a = 104.5 \qquad b = 2.4$$

e_h = 0.75 for drop and 0.85 for all other hammers

Hiley Formula [6]

$$P_u = \frac{e_h\ E_h}{s + \frac{1}{2}(k_1+k_2+k_3)}\ \frac{W + n^2\ W_p}{W + W_p}$$

k_1 = 12.5 mm [6] $\qquad$ n = 0.5 [6]

k_3 = 0.0 mm [6]

CHAPTER 3

STATIC PILE CAPACITY ANALYSIS

The ability of a pile foundation to support loads without failure or excessive settlement depends on a number of factors from which we will deal with the transfer of the pile load to soil which is also termed as pile bearing capacity.

As it is generally accepted the load supported by the pile is transferred to the soil in two ways:

- Skin Resistance
- Point Resistance

Generally also the Ultimate Bearing Capacity of a pile is considered as the sum of its Point Resistance and its Skin Resistance although the validity of this assumption is open to criticism:

$$Q_{ult} = Q_p + Q_s$$

The Static Pile Capacity formulas which are derived from bearing capacity theories depend on shear strength τ_f. ($\tau_f = c' + \sigma' \tan \phi'$). In the absence of c' (soil cohesion) the soil strength ~~parameter~~ involved is the angle of internal friction ϕ'.

In sandy soils, which is the one considered in this thesis the cohesion is negligible. Because of the relatively high permeability of sands, the angle of internal friction is usually based on drained tests. The angle of internal friction is primarily affected by the density of the sand and normally varies within the limits of about 28 to 46 degrees. Approximate values of ϕ' in terms of the relative density D_r [1] are given as follows :

$\phi' = 30^o + 0.15\ D_r$ for soils with **less** than 5 percent fines

$\phi' = 25^o + 0.15\ D_r$ for soils with **more** than 5 percent fines

Values of $\phi = 25^o$ (degrees) for loose sands and $\phi = 35^o$ (degrees) for dense sands are conservative for most cases of static loading.

" The relation between the actual void ratio (e) and its limiting values, e_{max} and e_{min}, is expressed by the relative density or relative void ratio D_r " [35]:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\ \%$$

In order to make a good prediction of pile bearing capacity the best would be to be able to get results from a load test and do a comparison. This is because of the difficulties in determining the in situ soil properties.

Three different formulas have been used in determining the ultimate bearing capacity values according to:

- the Navy Foundations and Earth Structures Design Manual (Navfac DM 7.2) **[24]**
- the method proposed by Meyerhof **[24]**
- the method proposed by J.L.Briaud **[8]**

3.1 Point Resistance Formulas

The Point Resistance in its simple form, as given in Navfac DM 7.2 can be expressed as follows:

$$Q_p = P_T \, N_q \, A_T$$

in which

P_T = Effective Vertical Stress at Pile Tip [if the embedment length E_l is greater than 20D (D being the diameter of pile) P_T is constant after E_l = 20 D]

N_q = Bearing Capacity Factor (it depends on the angle of internal friction ϕ' and is chosen from Table 3.1) **[24]**

A_T = Area of Pile Tip ($\pi D^2/4$)

Table 3.1 Bearing Capacity Factors - N_q

ϕ'	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q	10	15	21	24	29	35	42	50	62	77	86	120	145

N_q values are for driven pile displacement

Point Resistance of driven piles as proposed by Meyerhof is as follows:

$$Q_p = A_T \cdot q_{ult}$$

in which

A_T = Area of Pile Tip $(\pi D^2/4)$

q_{ult} = Ultimate Point Resistance of driven pile

$$= \frac{0.4\ \overline{N}\ E_d}{D} \leq q_l \qquad \text{(in TSF)}$$

$\overline{N}$ = average Standard Penetration Resistance near pile tip (blows/ft) corrected for the effective overburden stress at pile tip, p

$= C_N \cdot N$

N = Standard Penetration Resistance near pile tip (blow/feet) is the average of N SPT values 8D above to 3D below the pile point

C_N = correction factor for $\overline{N}$

$$= 0.77 \log_{10} \frac{20}{p} \quad \text{(for } p >= 0.25 \text{ TSF)}$$

p = effective overburden stress at pile tip (TSF)

E_d = depth driven into granular bearing stratum (ft)

D = width or diameter of pile tip (feet)

q_l = limiting point resistance (TSF), equal to 4N for sand and 3N for non-plastic silt.

J.L.Briaud introduced the point resistance as follows:

$$Q_p = A_T\, q_{ult}$$

in which

A_T = Area at Pile Tip $(\pi D^2/4)$

D = Diameter of the pile

q_{ult} = ultimate point resistance of driven pile

= $19.75\ (N_{pt})^{0.36}$

N_{pt} = uncorrected average SPT blow count over a distance of 4 diameter either side of the pile point

Vesic, Meyerhof and others contributions to the point resistance of piles have brought improvements to the static formulas.

3.2 Skin Resistance Formulas

The Skin Resistance in its simple form, as given in Navfac DM 7.2 can be expressed as follows:

$$Q_s = \Sigma\ (K_{HC})(P_o)(\mathrm{Tan}\delta)(s)$$

K_{HC} = Ratio of horizontal to vertical effective stress on side of element when element is in compression (K_{HC} = 1 to 2 for sand)

P_o = Effective vertical stress over length of embedment

δ = Friction angle between pile and soil

($\delta = 3/4\ \phi'$ for concrete)

s = Surface area of pile per unit length

Skin Resistance of driven piles as proposed by Meyerhof is as follows:

$$Q_s = A_s f_s$$

in which

A_s = Pile shaft area ($\pi D E_l$)

D = Diameter of the pile

E_l = embedment length of the pile

f_s = ultimate skin resistance for driven pile (TSF)

$$= \frac{N_{side}}{50} \leq f_l$$

N_{side} = uncorrected average SPT blow count within the shaft length considered

f_l = limiting skin resistance (for driven pile, f_l=1 TSF)

J.L.Briaud, introduced the skin resistance as follows:

$$Q_s = A_s f_s$$

in which

A_s = Pile shaft area ($\pi D E_l$)

D = Diameter of the pile

E_l = embedment length of the pile

f_s = ultimate skin resistance for driven pile

$= 0.224 \, (N_{side})^{0.29}$

N_{side} = uncorrected average SPT blow count within the shaft length considered

CHAPTER 4

A LITERATURE SURVEY ON THE WAVE EQUATION PILE ANALYSIS

The impact on the end of a longitudinal rod to describe pile driving has been considered historically by many people.

In 1955, 1962 an analytical method using the wave equation concept was proposed by E.A.L.Smith **[34]**, a mechanical engineer in Raymond International Inc. (Concrete Pile Division). The pile was taken as a slender rod and when there is an impact on the rod an initial velocity is given to the pile as a boundary condition. This produces a wave that propagates through the pile and goes to the pile end. If there is a resistance at the tip of the pile this wave comes back with a twice speed. If there is no barrier the pile is pulled downwards. Due to this motion tension and compression stresses occur inside the pile. The wave equation of E.A.L.Smith **[34]**, models the pile as a series of blocks connected by a series of springs. This method is a finite-difference solution which can be done by hand but is more practical with a digital computer.

Smiths's work was one of the very first applications of the digital computer in the solution of mechanics problems. A generally usable program was prepared by researchers at Texas A&M University as a direct outgrowth of Smith's work.

Forehand and Reese (1964) **[15]** illustrated the load-carying capacity prediction potential of the wave equation using the Smith model.

Later on an updated version of the Texas A&M program was written and called TTI (Texas Transportation Institute) program. Also, the WEAP program was developed to improve the analysis of diesel driving systems. Both of these programs were written under the sponsorship of the U.S. Department of Transportation.

The Texas Transportation Institute (TTI) computer program accounts for the nonlinearity of the soil damping force with penetration velocity. **[9]**

The Case Western Reserve University (CWRU) proposed a soil model where the soil damping force was uncoupled from the soil spring force. **[9]**

Holloway showed that residual stresses remaining in the pile and soil at the end of a hammer blow can be included in the analysis and may be important. **[27], [28]**

Matlock and Foo presented a discrete algorithm that may be used for either static or dynamic predictions and included Holloway's residual stress analysis. **[21], [28]**

Jean Louis Briaud presented a method for the evaluation of shaft and end resistance of piles driven in sand considering the residual driving stresses. **[8]**

The CAPWAPC **[11]** method of analysis (Case Pile Wave Analysis Program - Continuous Version) follows a procedure similar to that described by F.Rausche, F.Moses and G.G.Goble **([29], [30], [31])**. Data is obtained from an accelerometer and strain gauge bolted to the head of the pile. The results not only include the static pile capacity and load displacement diagrams but also indicate where on the pile the soil resistance is left.

Chow (1981) and Smith and Chow (1982) proposed a three-dimensional (axisymmetric) finite-element model for pile-driving analysis. The pile and the surrounding soil are discretized into finite elements. **[9], [20]** It was necessary to formulate a one-dimensional wave equation model that could reflect the observed phenomena of the three-dimensional model that was suitable for routine pile-driving analysis. **[17]**

A step towards this direction was taken by Simons and Randolph (1985). Corte and Lepert (1986) also investigated the use of a soil model that uses conventional soil parameters as input parameters. However, the proposed models suffer certain limitations. **[20]**

In recent years, there has been an increase in the use of stress waves measured during driving to predict the load-carrying capacity of driven piles. The use of measured stress waves is advantageous, since the uncertainty in the amount of energy transferred to the pile is eliminated. In this method, the hammer system is not modelled. Instead, the measured force or velocity trace at the instrumented location is used as a boundary condition in a wave equation analysis. The soil resistance and distribution are varied until the predicted force or velocity (depending on which was used as a boundary condition) trace matches the measured values. This method of prediction is also possible using the new model proposed by Lee et al. (1988) **[20]**. Although stress wave measurements may be more refined than the set matching methods proposed, the instrumentation is costly and matching the stress waves using the wave equation analysis requires considerable experience. In addition, the load-carrying-capacity prediction is limited only to the instrumented piles. The metod of stress wave matching is, therefore, less versatile and less economical than the methods of set matching, since sets are recorded routinely during driving.

In the new one-dimensional wave-equation model presented by Lee et al. [20] the pile is discretized into line elements interconnected at the nodes. The major difference between this new model and Smith's model or WEAP, TTI computer programs lies in the modelling of the soil behaviour during driving. The treatment of the pile using the finite difference method, finite element method, or the method of characteristics should result in essentially similar solutions for the same soil model.

The new soil model is derived from visco-elasto-dynamic theory. For the behaviour of the soil at the pile shaft, the soil model for a harmonically vibrating rod in an infinite medium was assumed (Novak et al. 1978). The soil stiffness and radiation damping coefficients are dependent on the frequency of excitation, and they are essentially constant at high frequencies.

CHAPTER 5

WAVE EQUATION PILE ANALYSIS - FADWAVE PROGRAM

5.1 Wave Equation Analysis

The Wave Equation Analysis is based on the theory of one dimensional wave propagation. For the analysis the pile is divided into a series of masses connected by springs which characterize the pile stiffness, and dashpots which simulate the damping below the pile tip and along pile embedded length.

This method was first put into practical form by E.A.L.Smith in 1962 **[34]**. The wave equation analysis provides a means of evaluating the suitability of the pile stiffness to transmit driving energy to the tip to achieve pile penetration, as well as the ability of pile section to withstand driving stresses without damage. The results of the analysis can be interpreted to give the following :

- Equipment Compatibility : appropriate hammer size and cushion.

- Driving Stresses : plots of stress vs. set can be made to evaluate the potential for pile overstress.

- Pile Capacity : plot of ultimate pile capacity vs. set can be developed.

The soil is modeled by approximating the static resistance (quake), the viscous resistance (damping), and the distribution of the soil resistance along the pile. The assigned parameter for springs and dashpots cannot be related to routinely measured soil parameters which constitutes the major draw back of the wave equation analysis.

5.1.1 The Mathematical Model of the Wave Equation Pile Analysis

Piles can be considered to be longitudinal rods and pile driving can be thought as the impact on the end of the longitudinal rods.

According to S.Timoshenko and J.N.Goodier 's theories [39] on the propagation of waves in an elastic medium; due to impact, motion is produced in an elastic body.

In the case of impact on a pile during the pile driving process the action of the suddenly applied force is not transmitted at once to all parts of the pile. The deformations produced by the force are propagated through the pile in the form of elastic waves.

If we consider the differential equation of impact of a long slender rod subjected to side resistance **R** as shown in Fig. 5.1:

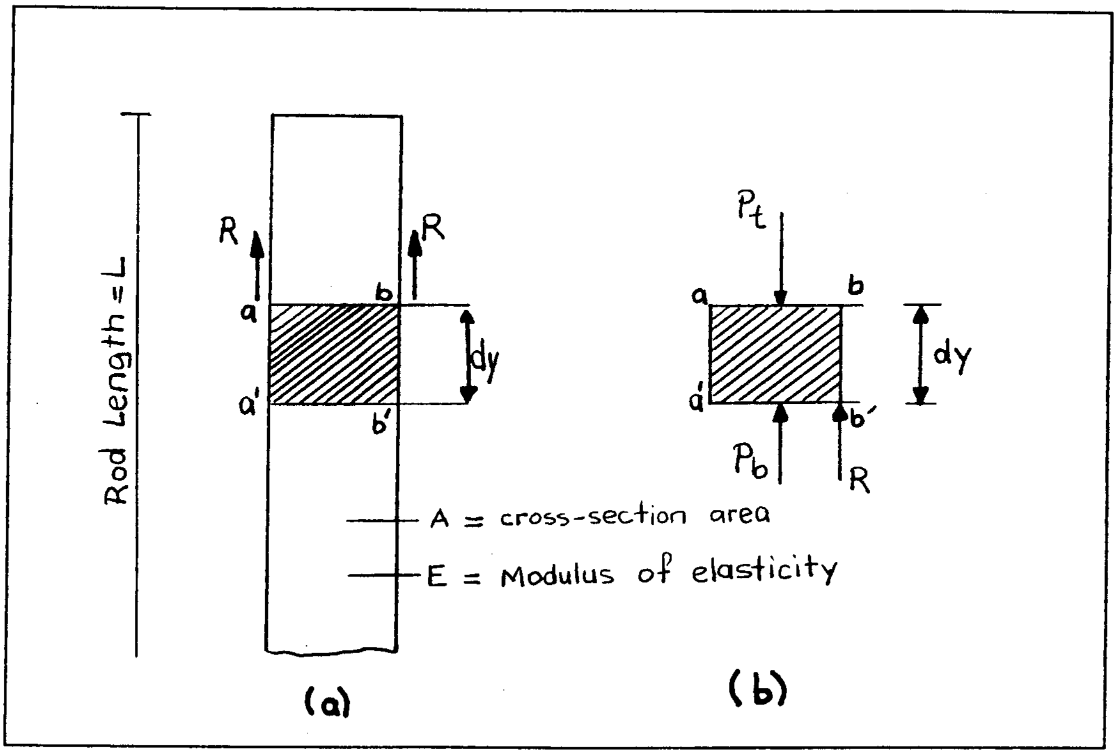

Fig. 5.1 Transmission of strain (and stress) in a long slender rod in the case of impact

As an assumption cross sections of the rod remain plane during deformation.

The unit elongation at any cross section **ab**, due to a longitudinal displacement **u**, is equal to $\partial u/\partial y$.

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

According to the Hooke 's Law we know that $\sigma = E.\varepsilon$.

$$\sigma = E.\varepsilon = E \cdot \frac{\partial u}{\partial y}$$

It is also known that $\sigma = F / A$; where A is the cross sectional area, therefore $F = \sigma.A$.

Then we can write the tensile force in the rod as :

$$P_t = F = \sigma.A = E.A.\frac{\partial u}{\partial y}$$

The force on the bottom of the rod element is:

$$Pb = E.A.\frac{\partial u}{\partial y} + E.A.\frac{\partial^2 u}{\partial y^2}$$

Considering an element of the rod between the two adjacent cross sections **ab** and **a′b′** we have the following forces as shown in Fig 5.1.(b):

The difference of forces P_{net} is:

$$P_{net} = P_b + R - P_t$$

$$= E.A.\frac{\partial u}{\partial y} + E.A.\frac{\partial^2 u}{\partial y^2} \pm R - E.A.\frac{\partial u}{\partial y}$$

$$= A.E.\frac{\partial^2 u}{\partial y^2} \pm R$$

Applying Newton 's Second Law of motion:

$$F = Pnet = m.a$$

$$= A.E.\frac{\partial^2 u}{\partial y^2} \pm R = m.a \qquad \text{(I)}$$

Taking the density ρ which is the mass per unit volume of the rod mass can be written as:

$$m = A.\rho.dx \qquad \text{(i)}$$

Due to the longitudinal displacement **u** the acceleration can be written as:

$$a = \frac{\partial^2 u}{\partial t^2} \qquad \text{(ii)}$$

Placing **i** and **ii** into **I** the equation of motion can be written as:

$$A.E.\frac{\partial^2 u}{\partial y^2} \pm R = A.\rho.dy.\frac{\partial^2 u}{\partial t^2}$$

$$\frac{E}{\rho} \frac{\partial^2 u}{\partial y^2} \pm R = \frac{\partial^2 u}{\partial t^2}$$

Letting $E / \rho = c^2$ we obtain :

$$c^2 \frac{\partial^2 u}{\partial y^2} \pm R = \frac{\partial^2 u}{\partial t^2}$$

Which is the classical one-dimensional wave equation including **R** the resistance of the surrounding medium.

The finite-element form of the above equation used in the Wave Equation Pile Analysis is:

$$D_m = 2D_m' - D_m'' + \frac{F_{am}\, g}{W_m} (DT)^2$$

The objection of using the wave equation is actually to find forces in each pile segment and the displacement (or set) of the pile point. The following equations can be used for obtaining these results:

The instantaneous element displacement D_m is:

$$D_m = D_m' + v_m \text{ (DT)}$$

The net element compression or tension movement is:

$$C_m = D_m - D_{m+1}$$

The force in segment **m** is:

$$F_m = C_m \left(\frac{A.E}{L} \right)_m = C_m K_m$$

The soil spring constant is:

$$K_m' = \frac{R_m'}{\text{quake}}$$

The side or point resistance term is obtained using damping with the side (Js) or point (Jp) value J and K:

$$R_m = (D_m - D_{sm}) K_m' (1 + J v_m)$$

The accelerating force in segment **m** is:

$$F_{am} = F_{m-1} - F_m - R_m$$

The element velocity is:

$$v_m = v_m' + \frac{F_{am} g}{W_m} \text{ (DT)}$$

5.2 FADWAVE Program - Its Use

The FADWAVE computer program ~~written~~ by J.E.Bowles is used in pile driving analysis to determine if the pile-hammer combination can be used.

When input of ultimate pile resistance R_u results in forces that are too large for the material, either the pile or the hammer must be changed.

It can be used to estimate pile driving stresses.

It can be used to estimate capacity in terms of set/blow. If the permanent set is very small it will mean that the hammer is too small or the pile is too heavy.

" This program can be used for all types of pile algorithms. The algorithm used is that given by E.A.L.Smith (1962) and is very similar to the one used by the Texas Transportation Institute and others." [7]

5.3 FADWAVE Program - Required Inputs

The hammer ram, capblock, pile cap and the pile are represented as series of weights and springs as shown in Fig. 5.2.

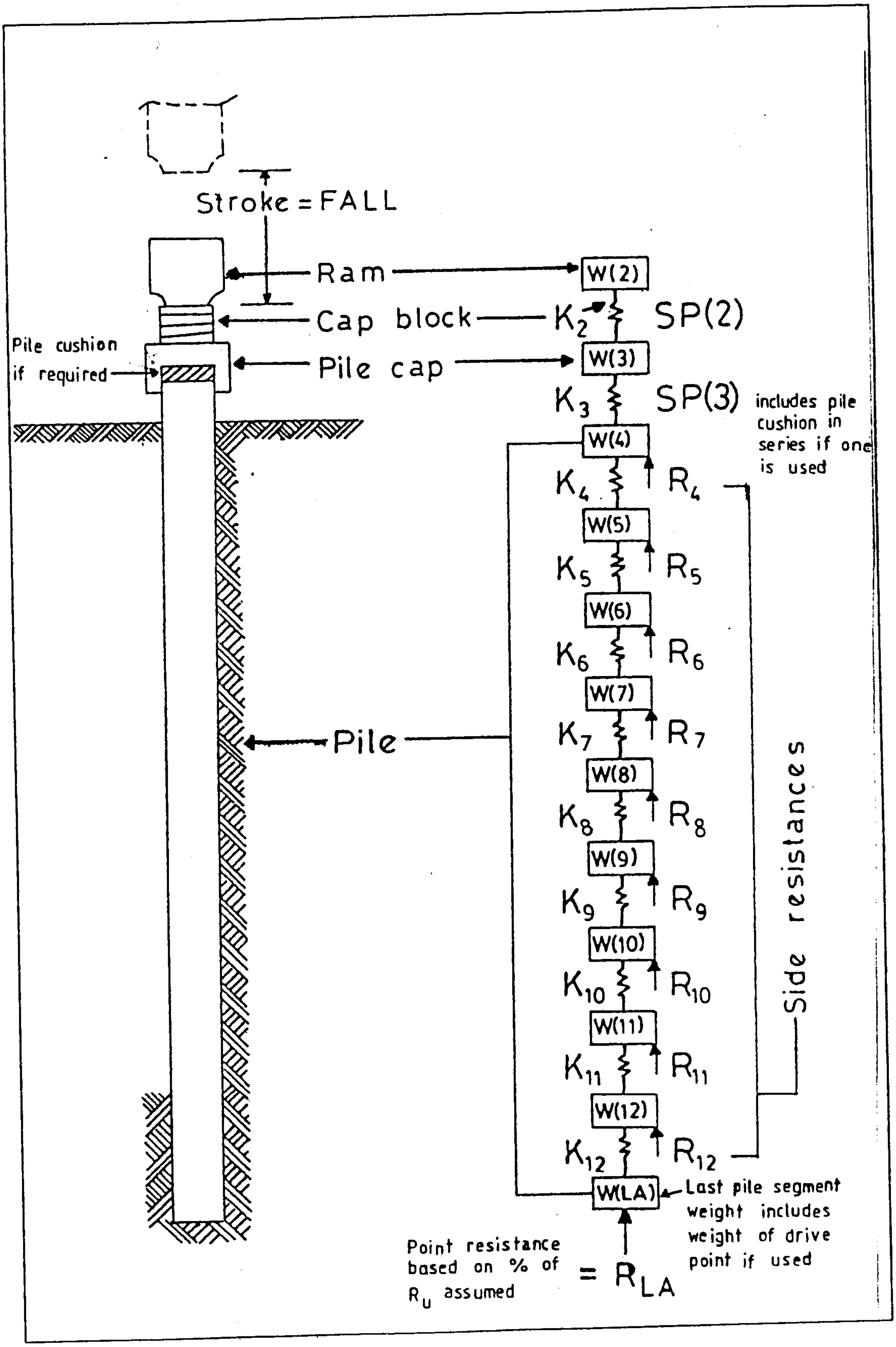

Fig. 5.2 The Wave Equation Model [4]

5.3.1 NELEM and ELEML

The input value NELEM is the number of elements including ram and the pile cap. The pile is divided to elements according to the pile length such that ELEML the element lengths are approximately 1 to 1.2 meters.

5.3.2 UNITS

The units can either be given in FPS (based on the foot of length and pound of force) or SI (based on the meter of length and the kilogram of mass) system.

5.3.3 Pile

The pile is a heavy object, but is somewhat compressible because of its length. It is therefore subject to wave action under the blow of the hammer.

This wave action can be analyzed mathematically by dividing the pile into short sections or 'unit lengths'.

- the weight of each unit length is represented by an individual weight [W(4) to W(LA) of Fig.5.3]

- the elasticity of each unit length is represented by an individual spring [K(3) to K(12) of Fig.5.3]

The motion of each weight and each spring is then calculated as though each were actually a separate and distinct object.

5.3.4 ITYPE = Pile Types

Any type of pile can be used, accordingly the program asks different input values:

- for Constant Section Piles

  WFT is the weight of pile in KIPS/FT or KN/M

  AREA is the area of pile in SQ IN. or SQ M

- for Tapered or Constant Pipe Piles

  DIAT is the top diameter, in INCH or METERS

  DIAB is the bottom diameter, in INCH or METERS

  UNITWT is the unit weight of the pile material

- Any Pile Type

  W(M+1), M = 3, NELEM the element weights

  A(M), M = 3, NELEM the element areas

5.3.5 TWALL (Wall Thickness)

Wall thickness of pipe piles should be given as an input. In the case of a solid pile or when not needed in the problem it should be given as 0.00.

5.3.6 Pile Driver Ram

The hammer ram is ordinarily a short, heavy, rigid object that can be represented by a single weight without elasticity. [W(2) of Fig.5.3]

5.3.7 Cap Block

To avoid pile damage a pile cap block is generally inserted between the ram and pile cap. Termed 'Dolly' in England the cap block is a short springy object of wood, plastic, or similar material which is comparatively light and which may therefore be represented by a spring. [K(2) of Fig.5.3]

5.3.8 Pile Cap (or Follower or Helmet)

Like the ram of the hammer, the pile cap is ordinarily a short heavy rigid object that can be represented by a single weight without elasticity. [W(3) of Fig.5.3]

5.3.9 Cushion Blocks (Pile Cushion)

If a precast concrete pile is driven, a cushion block (called 'Head Packing' in England) must be placed between the pile cap and the pile head so as to protect the concrete from shattering. It may be used to even out the contact surface and reduce driving stresses in the pile. In this case, spring K(3) [of Fig.5.3] would represent the pile cushion which is soft material such as wood planking 3 to 6 in. thick.

5.3.10 Height of fall of the ram of the pile hammer

This is either given or back-computed as $h = E_h/W_r$. This is needed to obtain the velocity of the pile cap at DT=1 (instant of impact), which is computed as $v_1 = \sqrt{e_h(2gh)}$

5.3.11 Modulus of elasticity of pile

Modulus of elasticity of the pile used should be given which is taken as 29430 MPa for this thesis..

5.3.12 Parameters related to soil properties :

- Quake (maximum amount of elastic soil deformation)
- Point Damping Constant, J_p
- Side Damping Constant, J_s ($J_s = J_p / 3$) [34]

J.E.Bowles [4] states that quake and damping are not very critical and that Q=0.1 to 0.15 and J_p=0.15 to 0.30 (FPS) are acceptable.

The lower limits which are the values proposed by E.A.L.Smith [34] are taken in the thesis.

5.3.13 Pile hammer efficiency

Values of hammer efficiency depend on the condition of the hammer and capblock and possibly the soil (especially for diesel hammers). In the absence of known values the following may be taken as representative of hammers in reasonably good operating condition:

Type	Efficiency e_h
Drop hammers	0.75-1.00
Single-acting hammers	0.75-0.85
Double-acting or differential	0.85
Diesel hammers	0.85-1.00

Pile hammer efficiency 1.0 is taken in this thesis.

5.3.14 Time interval of computations (DT)

The system is considered in a series of separate time intervals DT chosen sufficiently small that the stress wave should just travel from one element into the next lower element during DT. Practically this is not possible, and DT is taken as a value which usually works as in the following table:

Element material	Length, m	Trials DT
Steel	2.4-3.1	0.00025
Wood	2.4-3.1	0.00025
Concrete	2.4-3.1	0.00033

For shorter lengths, DT should be made correspondingly smaller. The DT value used in this thesis is 0.00020. The actual time DT can be approximately computed as

$$DT = C \sqrt{\frac{W_m L}{A E g}}$$

where C is 0.5 to 0.75 [6].

5.3.15 Coefficient of restitution for pile cap block and Coefficient of restitution for pile cushion

Representative values of coefficient of restitution is as shown in the following table:

Material	n
Micarta	0.85
Broomed wood	0
Wood piles (nondeteriorated end)	0.25
Compact wood cushion on steel pile	0.32
Compact wood cushion over steel pile	0.40
Steel-on-steel anvil on either steel or concrete pile	0.50
Cast-iron hammer on concrete pile without cap	0.40

Coefficient of restitution for pile cap block = 0.80 and Coefficient of restitution for pile cushion = 0.50 are the values used in this thesis.

5.3.16 RUTOT

It is suggested to take the ultimate load capacity obtained from a pile load test and give this an input to the program.

Pile load tests are made to determine the ultimate failure load of a pile or a group of piles to determine if the pile or pile group is capable of supporting a load without excessive or continuous settlement.

Since pile load tests are too expensive and maintaining necessary things for these tests are not very easy they are done in very big projects.

In this thesis ultimate load capacity values are obtained using both dynamic and static formulas. The input values to the FADWAVE program are the ones obtained from static pile capacity formulas.

5.3.17 Side Resistances - R(I) I=1,NELEM

The soil resistances (side resistances) can either be given as an input or can be computed in the program on a pro-rata basis according to the given RUTOT. The side resistance to be given as an input to the first segment is taken equal to zero (0) because of the wiping action. The side resistance input to the FADWAVE program are calculated using static formulas.

5.3.18 PER

On the bottom element the soil resistance (R_{LA} of Fig. 5.3) is actually the sum of the soil side resistance and point resistance calculated by the STATIC formulas. Therefore while giving the percent (PER) value one should be very careful. Accordingly the formula for the PER is :

$$PER = \frac{\text{Point Resistance + Soil Resistance Of The Last Element}}{\text{RUTOT (estimated ultimate pile resistance)}}$$

CHAPTER 6

SAMSUN AKM CASE STUDY

6.1 Subsurface Exploration

" Adequate subsurface exploration must precede the design of pile foundations. Investigations must include the following:

- Geological section showing pattern of major strata and presence of possible obstructions, such as boulders, buried debris, etc.

- Sufficient test data to estimate strength and compressibility parameters of major strata.

- Determination of probable pile bearing stratum. " [24]

6.1.1 Topographical & Geomorphological Features Of The Site

The site is a land reclaimed from the sea during the Samsun Harbour Construction in the 1950's.

Before the construction of the Samsun Harbour, the present construction site for the Samsun Atatürk Kültür Merkezi (Samsun AKM) building was situated beyond the water edge at about 9.6 meter water depth.

After the construction of the quay wall of the Samsun Harbor the materials dredged from the sea were used to backfill the landside of the quay wall.

Presently the ground surface level of the construction site is about 1.5 m above the sea level. **[41]**

6.1.2 Borings

Six (6) boreholes of 4 inch diameter were drilled with rotary drilling between July 10 and July 26, 1987 together with undisturbed Sampling and Standard Penetration Tests (SPT). The ground surface elevations vary within minimum 10 cm and maximum 40 cm. The datum level is taken at ground surface. Depths of the boreholes are as shown in Table 6.1 **[41]** given below :

Table 6.1 Depths of boreholes

borehole number	depth (m)	ground water level (m)
1	25.00	1.30
2	25.00	1.10
3	25.00	1.20
4	25.00	1.50
5	25.00	1.30
6	25.00	1.40

6.1.3 Field Test

As a field test a Standard Penetration Test was carried out of which the N (number of blows for a penetration of 300 mm after 15 mm initial drive) are as shown in Table 6.2 given below [41] :

Table 6.2 N (SPT) profiles

Boring No.	SPT No.	Depth (m)	N (SPT) # of blows
S-1	SPT-1	1.50 - 1.95	16
S-1	SPT-2	3.00 - 3.45	16
S-1	SPT-3	4.50 - 4.95	15
S-1	SPT-4	6.00 - 6.45	20
S-1	SPT-5	7.50 - 7.95	29
S-1	SPT-6	9.00 - 9.45	38
S-1	SPT-7	10.50 - 10.79	> 50
S-1	SPT-8	12.00 - 12.43	> 50
S-1	SPT-9	15.50 - 15.95	20
S-1	SPT-10	16.50 - 16.95	34
S-1	SPT-11	18.00 - 18.45	36
S-1	SPT-12	19.50 - 19.95	49
S-1	SPT-13	21.00 - 21.45	59
S-1	SPT-14	22.50 - 22.95	51
S-1	SPT-15	24.50 - 24.95	56

Table 6.2 (Continued)

Boring No.	SPT No.	Depth (m)	N (SPT) # of blows
S-2	SPT-1	1.50 - 1.95	19
S-2	SPT-2	3.00 - 3.45	24
S-2	SPT-3	4.50 - 4.95	23
S-2	SPT-4	6.00 - 6.45	11
S-2	SPT-5	7.50 - 7.95	37
S-2	SPT-6	9.00 - 9.45	51
S-2	SPT-7	10.50 - 10.79	> 50
S-2	SPT-8	12.00 - 12.45	36
S-2	SPT-9	15.00 - 15.45	61
S-2	SPT-10	16.50 - 16.95	68
S-2	SPT-11	18.00 - 18.45	48
S-2	SPT-12	19.50 - 19.95	47
S-2	SPT-13	21.00 - 21.45	50
S-3	SPT-1	1.50 - 1.95	24
S-3	SPT-2	3.00 - 3.45	12
S-3	SPT-3	4.50 - 4.95	30
S-3	SPT-4	6.00 - 6.45	8
S-3	SPT-5	7.50 - 7.95	18
S-3	SPT-6	9.00 - 9.45	10
S-3	SPT-7	10.50 - 10.95	76
S-3	SPT-8	12.00 - 12.45	54
S-3	SPT-9	13.50 - 13.95	28
S-3	SPT-10	15.00 - 15.45	85
S-3	SPT-11	16.50 - 16.95	59
S-3	SPT-12	18.00 - 18.45	39
S-4	SPT-1	1.50 - 1.95	16
S-4	SPT-2	3.00 - 3.45	17
S-4	SPT-3	4.50 - 4.95	18
S-4	SPT-4	6.00 - 6.45	8
S-4	SPT-5	7.50 - 7.95	25
S-4	SPT-6	9.00 - 9.45	31
S-4	SPT-7	10.50 - 10.95	26
S-4	SPT-8	12.00 - 12.45	18
S-4	SPT-9	13.50 - 13.95	24
S-4	SPT-10	15.00 - 15.45	77
S-4	SPT-11	16.50 - 16.95	64
S-4	SPT-12	18.00 - 18.45	56

Table 6.2 (Continued)

Boring No.	SPT No.	Depth (m)	N (SPT) # of blows
S-5	SPT-1	1.50 - 1.95	9
S-5	SPT-2	3.00 - 3.45	10
S-5	SPT-3	4.50 - 4.95	14
S-5	SPT-4	6.00 - 6.45	15
S-5	SPT-5	7.50 - 7.95	61
S-5	SPT-6	9.00 - 9.45	67
S-5	SPT-7	10.50 - 10.95	72
S-5	SPT-8	12.00 - 12.45	45
S-5	SPT-9	13.50 - 13.95	12
S-5	UD-1	15.00 - 15.40	
S-5	UD-2	19.20 - 19.50	
S-5	SPT-10	19.50 - 19.95	87
S-6	SPT-1	3.00 - 3.45	17
S-6	SPT-2	4.50 - 4.95	50
S-6	SPT-3	7.50 - 7.95	40
S-6	SPT-4	10.50 - 10.95	80
S-6	SPT-5	12.00 - 12.45	39
S-6	SPT-6	13.50 - 13.95	52
S-6	UD-1	15.00 - 15.50	
S-6	SPT-7	15.50 - 15.95	36
S-6	SPT-8	16.50 - 16.95	34
S-6	UD-2	17.50 - 18.00	
S-6	SPT-9	18.00 - 18.45	46

As all piles of the pile driving test programme are embedded in the upper sand layer, a statistical analysis can be done to see if 6 boring samples is enough to say that the layer between 0 and 9 m ~ 10 m is the same all throughout the foundation area.

The One Way Statistical Analysis of Variance [40] has been made to see if observed differences among the six (6) boring sample means can be attributed to chance or whether they are indicative of actual differences among the corresponding population means in terms of N (SPT) values ?

According to the ANOVA table the null hypothesis is tested with a significance level Alpha = 0.005 using the Percentage Points of the F distribution Table.
Since $F_0 = 0.94$ does not exceed $F_{0.95} = 2.57$,

$$F_0 = \mathbf{0.94} < F_{0.95} = \mathbf{2.57}$$

the null hypothesis can not be rejected.

Therefore we can say that the six samples come from the same population.

6.1.4 Idealized Soil Profile

The idealized soil profile - averaging the results of all the boreholes - is as shown in Table 6.3:

Table 6.3 Idealized Soil Profile

layer no.	depths (m)	thickness (m)	description	average N (SPT) # of blows
1	0.00-0.80	0.80	coarse gravel	-
2	0.80-9.60	8.80	fine silty SAND, grey erratic seams of coarse sand and shells medium dense	24
3	9.60-15.80	6.20	gravelly SAND, yellow erratic gravel layers and fossilized material very dense	45
4	15.80-25.00	9.20	CLAYSTONE	52

As it can be seen the old sea level is 9.60 meters below the construction surface level.

Tips of piles driven during the pile driving tests reached refusal within the upper grey sand backfill layer.

6.1.5 Laboratory Tests

Results of laboratory tests carried out on undisturbed samples and on samples obtained with split spoon sampler from Standard Penetration Tests are as shown in Table 6.4 given below:

Table 6.4 Laboratory Test Results

SAMPLE		Wn	ATTERBERG LIMITS			SIEVE ANALYSIS	
Sample No.	Depth (m)	W n %	LL	PL	PI	(+4 no) %	(-200 no) %
SPT-1	1.50- 1.95	40.9		N.P.		0.8	48.0
SPT-3	4.50- 4.95	27.1		N.P.		.0	19.1
SPT-5	7.50- 7.95	18.3		N.P.		5.0	48.1
SPT-7	10.50-10.79	18.5		N.P.		4.3	40.5
SPT-8	12.00-12.43	19.0		N.P.		5.0	41.0
SPT-9	15.50-15.95	19.3	51.3	24.3	27.0	.0	96.8
SPT-11	18.00-18.45	26.1	59.3	23.6	35.7	1.0	95.8
SPT-13	21.00-21.45	24.3	62.2	22.1	40.1	.0	99.2
SPT-15	24.50-24.95	25.2	59.7	22.7	37.0	.0	99.0
SPT-1	1.50- 1.95	18.7		N.P.		2.8	20.1
SPT-3	4.50- 4.95	21.4		N.P.		0.4	19.5
SPT-5	7.50- 7.95	18.9		N.P.		5.1	14.8
SPT-7	10.50-10.79	20.8		N.P.		2.5	17.4
SPT-9	15.00-15.45		57.7	24.8	32.9	0.1	97.1
SPT-10	16.50-16.95	28.8	58.3	18.3	40.0	0	96.4
SPT-11	18.00-18.45	25.1	59.8	23.9	35.9	0	96.8
SPT-12	19.50-19.95	23.4	58.3	22.9	35.4	0	97.0
SPT-13	21.00-21.45	25.0	60.6	24.8	35.8	0	97.9
SPT-1	1.50- 1.95	25.3		N.P.		2.9	36.6
SPT-3	4.50- 4.95	10.6		N.P.		0.3	35.5
SPT-5	7.50- 7.95	18.7		N.P.		0.7	38.0
SPT-7	10.50-10.95	22.0		N.P.		2.0	43.1
SPT-9	13.50-13.95	24.0		N.P.		2.6	43.7
SPT-11	16.50-16.95	20.7		N.P.		7.5	21.7
SPT-1	1.50- 1.95	23.9		N.P.		1.3	24.9
SPT-3	4.50- 4.95	27.4		N.P.		1.9	23.8
SPT-5	7.50- 7.95	20.0		N.P.		0.4	14.3
SPT-7	10.50-10.95	23.5		N.P.		3.7	13.0
SPT-2	3.00- 3.45	22.2		N.P.		5.3	21.9
SPT-4	6.00- 6.45	20.2		N.P.		12.9	26.4
SPT-6	9.00- 9.45	20.0		N.P.		10.0	27.0
SPT-8	12.00-12.45	18.2		N.P.		0	18.2
UD-1	15.00-15.45	24.4	57.0	23.0	34.0	0	98.5
UD-2	19.20-19.50	27.1	55.8	24.9	30.9	0	97.6
SPT-10	19.50-19.95	20.0	56.4	25.1	31.3	0	97.0
SPT-4	10.50-10.95	19.3		N.P.		7.1	12.6
SPT-6	13.50-13.95	18.7		N.P.		8.7	14.0
UD-1	15.00-15.50	24.9	56.0	26.0	30.9	0	98.2
SPT-8	16.50-16.95	26.0	59.0	25.8	33.2	0	98.1
UD-2	18.00-18.45	27.8	57.4	25.8	31.6	0	96.9

6.1.6 Engineering Characteristics of The Upper Sand Layer

6.1.6.1 Unit Weight

The unit weight values used in the thesis is assumed as follows:

γ_{dry} = 1.67 ton/m^3

γ_w = 1.00 ton/m^3

γ_{sat} = 2.06 ton/m^3

γ_{sub} = 1.06 ton/m^3

6.1.6.2 Angle of Internal Friction

As the content of fines are greater than 5 percent for both the grey silty fine sand and yellow gravelly sand, the following expression proposed by Meyerhof [1] was used to evaluate the average value of the angle of internal friction:

$$\phi' = 25 + 0.15\ D_r$$

The angle of internal friction values are as follows:

$\phi' = 34^o$ for the grey silty fine sand layer

$\phi' = 38^o$ for the yellow gravelly sand layer

The relative densitie (D_r) values used in the thesis is:

Dr = 0.58 (1.5 m to 9.5 m)

Dr = 0.86 (9.5 m to 18 m)

6.1.6.3 Vertical Effective Stress Diagram

Typical vertical effective stress diagram is as shown in Fig. 6.1

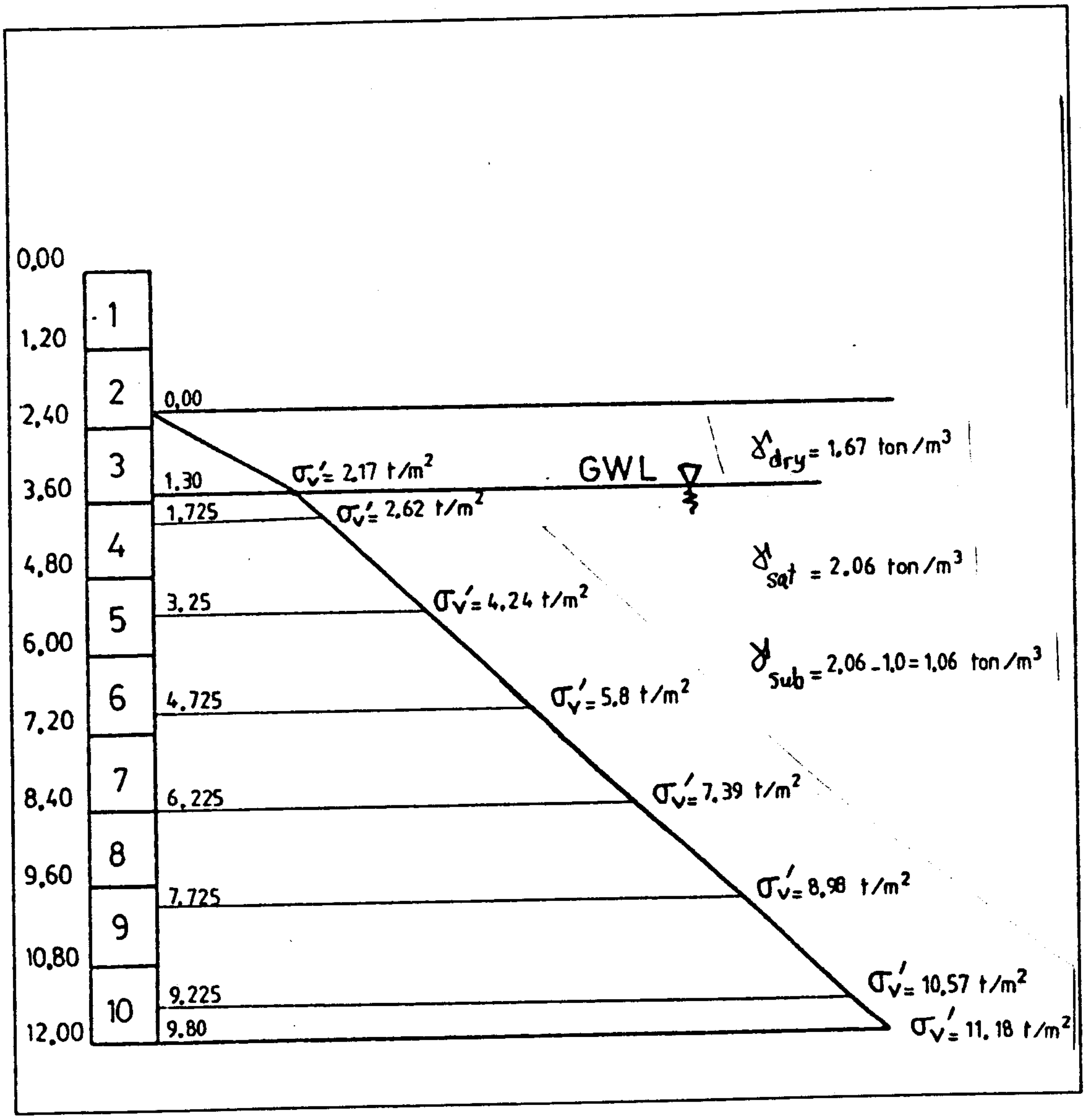

Fig. 6.1 Effective Stress Diagram

6.2 Type and dimensions of the piles

Characteristics of the centrifugally spun reinforced concrete piles used in the ~~driving~~ tests are as follows:

cross sectional area	: 942.5 sq.cm.
cross section	: circular
outer diameter	: 40 cm
wall thickness	: 10 cm
pile length	: 12.0 m
pile weight	: 2800 kg

6.3 Characteristics of the diesel pile driving hammers used in the pile driving tests

Two different diesel pile driving hammers have been used in the pile driving tests, characteristics of which are given as follows:

	Delmag D22-13	**Delmag D-15**
energy (N-m)	67000	37500
ram weight (N)	~ 22000	~ 15000
weight of pile cap (N)	5000	5000

CHAPTER 7

PILE DRIVING TESTS WITH DELMAG D22-13 DIESEL PILE HAMMER

Eleven (11) pile driving tests were carried out with Delmag D22-13 diesel pile hammer, field records of which are as shown:

test no.	date	embedment depth (m)	set (mm/blow)
1	20/4/1988	9.80	4.167
2	"	9.80	4.167
3	"	10.40	1.099
4	"	9.70	0.685
5	"	9.10	7.692
6	21/4/1988	8.80	2.679
7	"	9.40	2.941
8	"	9.50	4.762
9	"	7.00	6.024
10	"	8.00	5.747
11	22/4/1988	8.50	4.255

Nine (9) out of the eleven (11) piles driven with Delmag D22-13 diesel pile hammer were damaged during the pile driving tests, due to overdriving.

The mean and standard deviation of the sets recorded are respectively 4.0198 mm and 2.0916 mm.

The range of set values with % 75 reliability are within 2.54 mm and 5.5 mm limits.

The actually measured set value of 4.17 mm during the pile driving tests has been accepted in the computation of ultimate pile bearing capacity with dynamic formulas.

7.1 Ultimate pile capacity using DYNAMIC formulas

Janbu, Gates and Hiley dynamic formulas have been used for the calculations of the Ultimate Pile Bearing Capacities as shown in the following Table 7.1.

Table 7.1 Dynamic Formula Results (D22-13)

Pu (Ultimate Pile Bearing Capacity)		
Janbu	P_u = 3100 KN	(316 m.tons)
Gates	P_u = 1404 KN	(143 m.tons)
Hiley	P_u = 2156 KN	(220 m.tons)

7.2 Ultimate load capacity obtained with STATIC pile formulas without consideration of residual loads

The three (3) Static Pile Formulas used in this study for the skin friction, point resistance and ultimate load capacity calculations are as follows :

- the method of analysis given in NAVFAC DM 7.2
- Meyerhof method of analysis
- J.L.Briaud method of analysis

The predicted point resistance, skin friction and Ultimate Pile Bearing Capacities by the three (3) Static Formulas are as shown in Table 7.2.

Ultimate pile load capacity computations for the method of analysis given in NAVFAC DM 7.2 are made for three (3) values of K_{HC} which is the coefficient of horizontal earth pressure in compression.

Table 7.2 Static Formula Results (D22-13)

Static Formulas	Q_s (m.tons)	Q_p (m.tons)	Q_{ult} (m.tons)
Navfac for KHC=1.0	32.78	96.31	129.0
KHC=1.5	49.17	96.31	145.5
KHC=2.0	65.56	96.31	161.9
Meyerhof	57.72	157.12	214.8
J.L.Briaud	67.67	106.43	174.1

7.3 Residual loads calculated using the method of analysis proposed by J.L.Briaud [8]

The skin friction, point resistance and ultimate load capacity results obtained from the Static Pile Formulas (given in Table 7.2) were taken as input for calculating the Residual loads using the method proposed by J.L.Briaud. The results can be seen in the following Table 7.3

Table 7.3 Residual Load Results (D22-13)

Static Formulas	RESIDUAL LOAD (m tons)
Navfac for KHC=1.0	66.09
KHC=1.5	62.13
KHC=2.0	58.28
Meyerhof	106.82
J.L.Briaud	65.54

7.4 Application of the FADWAVE Computer Program to piles driven with the Delmag D22-13 Diesel Pile Hammer

The following input values have been given :

Pile type: Constant cross section	
Number of elements including Ram and Pile Cap	= 12
Pile element length	= 1.2 m
Weight of pile	= 2.746 KN/M
Pile cross sectional area	= 0.0942 m^2
Weight of pile driver ram	= 21.582 KN
Weight of pile cap	= 5.34 KN
Wall thickness	= 0.1 m
Height of ram fall	= 3.0 m
Pile hammer efficiency	= 1.0
Weight of driving point	= 0.0 KN
Damping constant for side resistance, SJ	= 0.164
Damping constant for point resistance, PJ	= 0.492
Spring constant of capblock	= 350000 KN/M
Spring constant of pile cushion	= 105000 KN/M
Modulus of elasticity of pile	= 29430 MPa
Coefficient of restitution for pile cap block	= 0.80
Coefficient of restitution for pile cushion	= 0.50
Time interval of computations, DT	= 0.00020 sec
Soil quake	= 2.54 mm

The average set, maximum and minimum forces had been obtained as outputs by giving the RUTOT and PER values calculated by using the STATIC pile formulas to the program.

The program was run for all methods firstly considering no residual loads and secondly for calculations done including the residual loads.

7.4.1 FADWAVE outputs without considering residual loads

The calculated RUTOT (Ultimate Pile Resistance) and PER (percentage of RUTOT carried by pile point) values using three (3) Static Pile Formulas and the Average set obtained from the program is as shown in Table 7.4.

Table 7.4 Output set with no residual loads (D22-13)

Static Formulas		RUTOT (kn)	PER	AVERAGE SET (mm/blow)
Navfac for	KHC=1.0	1266.41	0.796	12.593
	KHC=1.5	1427.20	0.728	11.129
	KHC=2.0	1587.99	0.674	9.721
Meyerhof		2107.58	0.793	5.546
J.L.Briaud		1707.92	0.701	8.609

The MAXIMUM FORCE and MINIMUM FORCE values obtained as an output from the FADWAVE computer program (according to the given inputs) together with the calculated MAXIMUM STRESS and MINIMUM STRESS values are as shown in Table 7.5.

Table 7.5 Output forces with no residual loads (D22-13)

Static Formulas		MAX FORCE (kn)	MAX STRESS (kg/cm^2)	MIN FORCE (kn)	MIN STRESS (kg/cm^2)
Navfac for	KHC=1.0	2819.69	304.97	- 34.94	- 3.78
	KHC=1.5	2879.14	311.40	- 17.94	- 1.94
	KHC=2.0	2931.11	317.02	0.00	0.00
Meyerhof		3357.21	363.10	0.00	0.00
J.L.Briaud		3021.65	326.80	0.00	0.00

K_{HC} is the coefficient of horizontal earth pressure in compression.

7.4.2 FADWAVE outputs considering residual loads

The calculated RUTOT (Ultimate Pile Resistance) and PER (percentage of RUTOT carried by pile point) values using three (3) Static Pile Formulas and the Average set obtained from the program is as shown in Table 7.6.

Table 7.6 Output set with residual loads (D22-13)

Static Formulas		RUTOT (kn)	PER	AVERAGE SET (mm/blow)
Navfac for	KHC=1.0	1914.72	0.555	7.039
	KHC=1.5	2036.75	0.529	6.115
	KHC=2.0	2159.71	0.505	5.210
Meyerhof		3155.48	0.550	0.109
J.L.Briaud		2350.87	0.511	3.886

The MAXIMUM FORCE and MINIMUM FORCE values obtained as an output from the FADWAVE computer program (according to the given inputs) together with the calculated MAXIMUM STRESS and MINIMUM STRESS values are as shown in Table 7.7.

Table 7.7 Output forces with residual loads (D22-13)

Static Formulas	**MAX FORCE (kn)**	**MAX STRESS (kg/cm^2)**	**MIN FORCE (kn)**	**MIN STRESS (kg/cm^2)**
Navfac for KHC=1.0	3007.14	325.24	0.00	0.00
KHC=1.5	3041.24	328.93	0.00	0.00
KHC=2.0	3069.63	332.00	0.00	0.00
Meyerhof	3245.72	351.04	0.00	0.00
J.L.Briaud	3125.22	338.01	0.00	0.00

K_{HC} is the coefficient of horizontal earth pressure in compression.

CHAPTER 8

PILE DRIVING TESTS WITH DELMAG D-15 DIESEL PILE HAMMER

Nine (9) pile driving tests were carried out with Delmag D-15 diesel pile hammer, field records of which are as shown:

test no.	date	embedment depth (m)	set (mm/blow)
1	22/4/1988	9.30	1.786
2	"	9.00	1.953
3	23/4/1988	9.70	0.417
4	"	9.10	0.962
5	"	9.20	0.862
6	"	9.10	0.847
7	"	9.70	0.729
8	"	9.40	1.282
9	"	9.30	0.855

None of the 9 piles driven with Delmag D-15 diesel pile hammer were damaged during the pile driving tests, due to overdriving.

The mean and standard deviation of the sets recorded are respectively 1.077 mm and 0.5038 mm.

The range of set values with % 75 reliability are within 0.5373 mm and 1.6167 mm limits.

The actually measured set value of 0.962 mm during the pile driving tests has been accepted in the computation of ultimate pile bearing capacity with dynamic formulas.

8.1 Ultimate pile capacity using DYNAMIC formulas

Janbu, Gates and Hiley dynamic formulas have been used for the calculations of the Ultimate Pile Bearing Capacities as shown in the following Table 8.1.

Table 8.1 Dynamic Formula Results (D-15)

Pu (Ultimate Pile Bearing Capacity)		
Janbu	P_u = 2593 KN	(264 m.tons)
Gates	P_u = 1426 KN	(145 m.tons)
Hiley	P_u = 1424 KN	(145 m.tons)

8.2 Ultimate load capacity obtained with STATIC pile formulas without consideration of the residual loads

The three (3) Static Pile Formulas used in this study for the skin friction, point resistance and ultimate load capacity calculations are as follows :

- the method of analysis given in NAVFAC DM 7.2
- Meyerhof method of analysis
- J.L.Briaud method of analysis

The predicted point resistance, skin friction and Ultimate Pile Bearing Capacities by the three (3) Static Formulas are as shown in Table 8.2.

Ultimate pile load capacity computations for the method of analysis given in NAVFAC DM 7.2 are made for three (3) values of K_{HC} which is the coefficient of horizontal earth pressure in compression.

Table 8.2 Static Formula Results (D-15)

Static Formulas	Q s (m.tons)	Q p (m.tons)	Qult (m.tons)
Navfac for KHC=1.0	29.06	96.31	125.4
KHC=1.5	43.60	96.31	139.9
KHC=2.0	58.13	96.31	154.4
Meyerhof	53.60	157.12	210.7
J.L.Briaud	62.83	106.43	169.2

8.3 Residual loads calculated using the method of analysis proposed by J.L.Briaud [8]

The skin friction, point resistance and ultimate load capacity results obtained from the Static Pile Formulas (given in Table 8.2) were taken as input for calculating the Residual loads using the method proposed by J.L.Briaud. The results can be seen in the following Table 8.3

Table 8.3 Residual Load Results (D-15)

Static Formulas	RESIDUAL LOAD (m.tons)
Navfac for KHC=1.0	65.22
KHC=1.5	61.50
KHC=2.0	57.88
Meyerhof	104.86
J.L.Briaud	64.33

8.4 Application of the FADWAVE Computer Program to piles driven with the Delmag D-15 Diesel Pile Hammer

The following input values have been given :

Pile type : Constant cross section	
Number of elements including Ram and Pile Cap	= 12
Pile element length	= 1.2 m
Weight of pile	= 2.746 KN/M
Pile cross sectional area	= 0.0942 m^2
Weight of pile driver ram	= 14.715 KN
Weight of pile cap	= 5.34 KN
Wall thickness	= 0.1 m
Height of ram fall	= 2.5 m
Pile hammer efficiency	= 1.0
Weight of driving point	= 0.0 KN
Damping constant for side resistance, SJ	= 0.164
Damping constant for point resistance, PJ	= 0.492
Spring constant of capblock	= 350000 KN/M
Spring constant of pile cushion	= 105000 KN/M
Modulus of elasticity of pile	= 29430 MPa
Coefficient of restitution for pile cap block	= 0.80
Coefficient of restitution for pile cushion	= 0.50
Time interval of computations, DT	= 0.00020 sec
Soil quake	= 2.54 mm

The average set, maximum and minimum forces had been obtained as outputs by giving the RUTOT and PER values calculated by using the STATIC pile formulas to the program.

The program was run for all methods firstly considering no residual loads and secondly for calculations done including the residual loads.

8.4.1 FADWAVE outputs without considering residual loads

The calculated RUTOT (Ultimate Pile Resistance) and PER (percentage of RUTOT carried by pile point) values using three (3) Static Pile Formulas and the Average set obtained from the program is as shown in Table 8.4.

Table 8.4 Output set with no residual loads (D-15)

Static Formulas	RUTOT (kn)	PER	AVERAGE SET (mm/blow)
Navfac for KHC=1.0	1229.94	0.819	7.067
KHC=1.5	1372.50	0.757	5.965
KHC=2.0	1515.06	0.707	4.894
Meyerhof	2067.16	0.808	1.563
J.L.Briaud	1660.44	0.720	3.903

The MAXIMUM FORCE and MINIMUM FORCE values obtained as an output from the FADWAVE computer program (according to the given inputs) together with the calculated MAXIMUM STRESS and MINIMUM STRESS values are as shown in Table 8.5.

Table 8.5 Output forces with no residual (D-15)

Static Formulas	MAX FORCE (kn)	MAX STRESS (kg/cm^2)	MIN FORCE (kn)	MIN STRESS (kg/cm^2)
Navfac for KHC=1.0	2466.36	266.75	-189.21	-20.46
KHC=1.5	2449.73	264.95	-159.44	-17.24
KHC=2.0	2429.83	262.80	-131.41	-14.21
Meyerhof	2830.26	306.11	0.00	0.00
J.L.Briaud	2514.23	271.93	-37.86	-4.09

K_{HC} is the coefficient of horizontal earth pressure in compression.

8.4.2 FADWAVE outputs considering residual loads

The calculated RUTOT (Ultimate Pile Resistance) and PER (percentage of RUTOT carried by pile point) values using three (3) Static Pile Formulas and the Average set obtained from the program is as shown in Table 8.6.

Table 8.6 Output set with residual loads (D-15)

Static Formulas		RUTOT (kn)	PER	AVERAGE SET (mm/blow)
Navfac for	KHC=1.0	1869.79	0.570	2.349
	KHC=1.5	1975.78	0.546	1.699
	KHC=2.0	2082.87	0.525	1.198
Meyerhof		3095.94	0.563	0.000
J.L.Briaud		2291.52	0.526	0.740

The MAXIMUM FORCE and MINIMUM FORCE values obtained as an output from the FADWAVE computer program (according to the given inputs) together with the calculated MAXIMUM STRESS and MINIMUM STRESS values ~~are as~~ shown in Table 8.7.

Table 8.7 Output forces with residual loads (D-15)

Static Formulas		**MAX FORCE (kn)**	**MAX STRESS (kg/cm^2)**	**MIN FORCE (kn)**	**MIN STRESS (kg/cm^2)**
Navfac for	KHC=1.0	2369.69	256.30	-235.14	-25.43
	KHC=1.5	2377.70	257.16	-198.96	-21.52
	KHC=2.0	2389.81	258.47	-189.83	-20.53
Meyerhof		2515.67	272.08	-330.50	0.00
J.L.Briaud		2437.62	263.64	-118.77	-12.85

K_{HC} is the coefficient of horizontal earth pressure in compression.

CHAPTER 9

EVALUATION AND INTERPRETATION OF THE RESULTS

9.1 Effect of the PER input value on the Set values output from the FADWAVE program

The FADWAVE program was run for different PER values in decreasing order and it was observed that the Average Set output values also decrease. The tests were done for the input RUTOT value obtained from the Janbu dynamic formula. The results can be seen in Table 9.1.

Table 9.1 Output set values for the input RUTOT calculated by the Janbu formula

Pile Driving Hammer Delmag D22-13				
DT (sec)	PER	RUTOT (kn)	EFF	SET (mm/blow)
0.0002	0.80	3100.11	1.00	0.692
0.0002	0.65	3100.11	1.00	0.395
0.0002	0.60	3100.11	1.00	0.304
0.0002	0.55	3100.11	1.00	0.218

Table 9.1 (Continued)

Pile Driving Hammer Delmag D-15				
DT (sec)	PER	RUTOT (kn)	EFF	SET (mm/blow)
0.0002	0.80	2592.50	1.00	0.135
0.0002	0.65	2592.50	1.00	0.006
0.0002	0.60	2592.50	1.00	0.005
0.0002	0.55	2592.50	1.00	0.005

9.2 Effect of the Residual Loads on the Set values output from the FADWAVE program

The averages of the set values obtained from the FADWAVE program without considering and considering residual loads were compared with the set values observed in the field. The results can be seen in Table 9.2.

Table 9.2 Comparison of computed set values with field set values

Pile Driving Hammer Delmag D22-13			
	PER	Computed Set (mm/blow)	Observed Set (mm/blow)
No Residual	0.738	9.520	4.167
With Residual	0.530	4.47	4.167
The PER and Computed Set values are the averages of the values in Table 7.4 (for no residual) and Table 7.5 (with residual).			

Table 9.2 (Continued)

Pile Driving Hammer Delmag D-15			
	PER	Computed Set (mm/blow)	Observed Set (mm/blow)
No Residual	0.762	4.678	0.962
With Residual	0.546	1.197	0.962

The PER and Computed Set values are the averages of the values in Table 8.4 (for no residual) and Table 8.5 (with residual).

It can be seen that the set values obtained when considering residual loads are closer to the observed set values obtained in the field.

9.3. Maximum Driving Stresses

No appreciable change occurs to maximum driving stresses when:

- Residual Loads are considered (Quake = 2.54 mm)
- For quake values varying between Q = 1.3 mm - 5.1 mm

The averages of the Maximum Stresses obtained as output from the FADWAVE program for pile driving hammers Delmag D22-13 and Delmag D-15 are compared. It can be seen from Table 9.3 that the stresses obtained for Delmag D22-13 are greater than Delmag D-15.

Table 9.3 Comparison of the maximum stresses

	Averages Of The Maximum Stresses (kg/cm^2) (No Residual Load)	
Delmag D22-13	324.658	(average of Table 7.5)
Delmag D-15	274.508	(average of Table 8.5)

CHAPTER 10

CONCLUSIONS

" With developments in foundation technique the field of application has also increased. It has become essential to choose the right type of pile and method of driving. To get the pile into the ground quickly and safely it is necessary to be able to predict, on the basis of the results of soil research, the driving process and to simulate the possibly extreme driving stresses and penetration speeds with sufficient reliability. " **[43]**

The Wave Equation Analysis gives a more reliable quantitative output regarding the bearing capacity estimation and driveability analysis.

The output from a wave equation computer program can also be viewed as qualitative information about the point force as well as the forces acting in the pile elements along the pile shaft.

10.1 Effect of adding the residual loads to the Ultimate Bearing Pile Capacity computed with static formulas to obtain the (RUTOT) as input value required by the Wave Equation Pile Analysis (FADWAVE Program).

As shown in section 9.2 introducing the total of Ultimate Bearing Capacity computed with Static Formulas and Residual Loads as input (RUTOT) value into the Wave Equation Pile Analysis, provides set values output from the FADWAVE PROGRAM with a better estimate for the set values as measured in the field during pile driving tests. But in this case RUTOT input value does not affect appreciably the maximum driving stresses during the pile driving operation. (Tables 7.4 - 7.7 and 8.4 - 8.7)

As a consequence it seems logical to accept that the results obtained with dynamic formulas on the basis of set values measured in the field, are inclusive of the residual loads.

In other words, deducting the residual loads from the gross value of the Ultimate Bearing Capacity as obtained from dynamic formulas computed on the basis of set values measured in the field during pile driving tests, offers a better estimate for the Ultimate Pile Bearing Capacity.

10.2 Integrity of piles driven in sand

Magnitude of driving stresses produced during the pile driving operation is a measure of the driving forces to which the pile body can stand without structural damage.

As also shown in the preceding section, by increasing the FADWAVE program RUTOT (Ultimate Bearing Capacity) input value the resulting differences in the driving stresses do not exceed 1 or 2 percent.

The driving stress values do not change as long as the RUTOT, Cross Sectional and Energy values are the same. The only thing that changes in this case is the Set values obtained from the FADWAVE PROGRAM.

A parametric study with varying RUTOT input value including Residual Loads, enables the engineer to take a decision regarding the upper limits of the Ultimate Bearing Capacity which can be secured without running in the risk of producing driving stresses detrimental to the structural integrity of the pile.

10.3 Decision about of the optimum embedment length of precast concrete piles in sand.

Usually decision about the length of the precast concrete piles based on the degree of experience of the engineer is a guesswork at best in this field.

The Wave Equation Analysis FADWAVE program offers the possibility of making a parametric study for driving a pile of known length and cross section using a pile driving hammer of given capacity by limiting the maximum driving stresses.

The wave equation is, in J.E.Bowles 's opinion, the best means currently available to obtain driving stresses, both compression and tension for concrete piles. The maximum value of P_u (or the force on a pile element) can be selected from a printout of forces vs. time. This value depends on the amount of side resistance assumed.

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