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CYCLIC LATERAL LOAD
BEHAVIOUR OF PILES

by
T. MOSIKEERAN

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A Report Submitted to The Earthquake and War Damages Commission

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ABSTRACT

The non linear soil model proposed by Pender is presented in this thesis based on an initial small strain stiffness, a yield pressure and an index "n". Carter (1984) wrote a finite element program to predict the static lateral load behaviour of piles using this model and found that this simple model gave similar standard of accuracy to the responses predicted using the p-y curves. Ling (1988) verified this fact.

The author used the same model to predict the cyclic lateral load behaviour of piles modifying the finite element program written by Carter (1984). Some full scale pile test results are presented to verify the validity of the soil model. This model was found to be a valuable tool for its simplicity and accuracy for cyclic loads as well.

The permanent set created in unloading cycles were achieved by stiffening soil springs in the unloading cycles i.e. increasing yield pressure in the soil model.

The cyclic degradation of soil was considered which is a function of number of cycles and the deflection of the pile in the previous loading cycle.
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LIST OF SYMBOLS

Throughout this thesis the symbols below will have the following meaning unless otherwise specified.

- $B$: width of beam or diameter of pile
- $C_N$: SPT overburden correction factor
- $c'$: apparent cohesion in terms of effective stress
- $D$: depth below ground surface
- $D_E$: degradation factor for soil modulus
- $D_p$: degradation factor for soil yield pressure
- $d$: pile diameter
- $E_C$: Young's modulus of concrete
- $E$, $E_p$: modulus of elasticity of pile
- $E_S$: modulus of elasticity of soil
- $ER_m$: energy ratio
- $e$: void ratio
- $G$: shear modulus of soil
- $g$: gravitational constant
- $H_{ij}$: member length
- $I$, $I_p$: moment of inertia of pile
- $K$: modulus of subgrade reaction
- $K_p$: coefficient of passive earth pressure
- $k_h$: coefficient of horizontal subgrade reaction
- $k_o$: the initial small strain coefficient of subgrade reaction
- $k_s$: spring stiffness
- $k_{si}$: spring stiffness at node $i$
$k_{sj}$  
spring stiffness at node $j$

$L$  
length of pile

$L_c$  
critical depth

$M_i, M_j$  
member moments

$N, N_m$  
measure SPT blow count

$N_{60}$  
$N$ value measured in SPT test delivering 60% of free fall energy where $N_{60} = E R_m N$

$(N_{t})_{60}$  
$N_{60}$ value corrected for overburden

$N_k$  
cone factor

$N_p$  
lateral bearing capacity factor

NDF  
total number of degrees of freedom

$n$  
index for the soil model that control the nonlinearity

$n_h$  
constant of subgrade reaction

$P_{ult}, P_{ult}$  
ultimate soil pressure

$P$  
lateral soil pressure

$q_c$  
cone bearing

$s_u, c_u$  
undrained shear strength of clay

$t$  
degradation parameter

$u_i, u_j$  
member displacements

$V_i, V_j$  
member shear

$V_s$  
in-situ shear wave velocity

$X_i, X_j$  
structural displacements

$x$  
length along the pile

$y$  
lateral pile displacement

$\alpha_i, \alpha_j$  
member rotations

$\theta_i, \theta_j$  
structural rotations
$E_{50}$ strain at one-half of the maximum deviator stress in an undrained test

$\rho$ density of soil

$\nu_s$ poisson's ratio of soil

$\gamma$ total unit weight of soil

$\Delta$ lateral pile displacement

$\sigma_0$ in-situ total overburden pressure

$\sigma_0'$ effective stress

$\phi'$ angle of friction for sand

$[a]$ displacement transformation matrix

$[g]$ force transformation matrix

$[k]$ member stiffness matrix

$[R]$ applied force matrix

$[r]$ structure displacement matrix

$[S]$ internal member force

$[s]$ member displacement matrix
CHAPTER 1

INTRODUCTION

The lateral load behaviour of the piles has been of great interest of many researchers in recent years ever since it was discovered that piles can be used to carry considerable amount of lateral loads as well as the axial loads. However, there are no internationally accepted methods available to predict the pile behaviour under lateral loads.

The response of piles subject to static loading has been analysed by different approaches including linear subgrade reaction analysis (e.g. Matlock and Reese, 1960), non-linear subgrade reaction (or "p - y") analysis (e.g. Matlock, 1970; Reese et al, 1975), elastic continuum analysis (e.g. Poulos, 1971) and finite element analysis (e.g. Desai and Appel, 1976). The lateral loads on piles may be induced by earthquakes, machines, waves, blasts or impacts and may be transient or cyclic in nature. Thus research on cyclic lateral loads has been of great importance. Of the analyses of the static loading only the "p - y" analysis allows for the effects of cyclic loading. But it gives only an envelope to the behaviour of a pile under cyclic loading. It doesn't consider the effect when the number of cycles increases. Poulos (1981) gives an approximate engineering analysis of cyclic lateral loading on a pile based on the use of elastic continuum theory. Because of the complication and tedious procedures involved in the proposed methods so far, there has always been a quest for a simple soil model which predict the pile response reasonably well.

The simple soil model proposed by Pender was first analysed by Carter (1984) for static lateral loading. This was further verified by Ling (1988). Both of them unanimously agreed on the versatility and accuracy of the soil model in predicting the behaviour of pile subject to static lateral loading. The author too analysed this model for cyclic loading and found to be very effective.

There are two major phenomena which may contribute to the response of the piles subject to cyclic loads with increasing numbers of cycles. First is the phenomenon of structural "shakedown". Second is the decrease in
compressibility and strength of the soil itself due to the cyclic loading known as cyclic soil degradation. Studies on shakedown of laterally loaded piles were undertaken by Swane (1980) as reported by Poulos (1981). However, the author considered only the effects of cyclic soil degradation.

The soil model analysed in this thesis gave a reasonable standard of prediction for sand and clay and proved to be as good as the more intricate "p - y" curves. When modelling the soil, the shape of the load displacement curves is specified by three parameters; an estimate of the initial small strain stiffness of the soil, estimate of the ultimate soil pressure and a parameter n. The values of n are believed to be 1 and 0.2 for sand and clay respectively. Further, two parameters COEFF and DEGPA which can be back calculated from the response of the full scale pile tests control the response with an increasing number of cycles.

A literature review based on the methods currently available in predicting the response of laterally loaded piles especially for cyclic loading is outlined in Chapter 2. The reader is asked to look for the references provided if more details are necessary.

Chapter 3 defines the non linear soil model and explains how this is used to predict the behaviour of piles embedded in sand and/or clay. This is basically the same as the non linear soil model analysed by Carter (1984) as far as the first loading cycle is concerned. This chapter also explains how this soil model helps to handle the subsequent cycles.

Chapter 4 explains the main structure of the finite element program written by Carter (1984) which was modified by the author to handle cyclic loading. Chapter 3 and Chapter 4 mostly describes the work done by Carter (1984). More details are given in the theses by Carter (1984) and Ling (1988).

Chapter 5 analyses a series of full scale pile tests and compares with the predicted response. A discussion on the predicted response of the full scale pile tests analysed in Chapter 5 is presented in Chapter 6. The summary and conclusions are presented in Chapter 7.
Appendix A describes the graphs of predicted and recorded pile response. A copy of the finite element program and the sample input and output are given in Appendix B.
CHAPTER 2:

REVIEW

2.1 INTRODUCTION

The design approach by engineers of lateral loaded piles has been comparatively simple. However, when these methods are used for cyclic or dynamic lateral loadings their performance has often been far from satisfactory. This has been demonstrated by Margason (1975).

The design of the laterally loaded piles is governed by the deflections at working loads and the ultimate strength of the pile. This problem can be solved when the load-deflection characteristics of the pile-soil system are investigated. However, there is no internationally accepted method available so far.

Recently there has been a lot of researchers interested in lateral load behaviour of piles. Most of the research and testing has been funded by oil companies because of the large financial investments involved in offshore platforms. The behaviour of soil-pile systems is dependent on the type of loading. Davisson and Prakash (1963), Singh and Prakash (1971) and Matlock (1970) have considered other than static load tests. The various load States are named as following:

1. Static
2. Repeated (or one way cyclic)
3. Cyclic (or two way cyclic)
4. Dynamic.

These are defined in Fig. 2.1
Fig. 2.1 Definition of the various modes of loading

The traditional approaches to the lateral loading problem have ignored any effect due to the pile installation process or the existence of axial loads on the pile. However, it has been suggested by many researchers that the vertical load did not have any appreciable effects on the measured lateral deflection for a laterally loaded free-headed piles.

When it comes to a group of pile, pile interaction becomes an important phenomena and is considerably more complex. In the case where the pile cap is not in contact with the soil, the general problem of a group subjected to lateral loads can be analysed assuming that each pile in the group behaves as an individual pile unaffected by other piles in the group. A number of methods solving this particular problem exists, for example, Hrennikoff (1956), Saul (1968) and Aschenbrenner (1967).
Some other researchers like Donovan (1959), Francis (1964), Nair, Gray and Donovan (1969) analysed this problem considering the individual piles in the group as "equivalent cantilevers". The cantilever length is chosen so that when subjected to lateral loads and moments of the real pile, the equivalent cantilever will model either the deflections sustained by the real pile, or the moments.

The procedures mentioned above assume that all the piles in the group behave in the same fashion. However, pile interaction reduces the load carrying capacity of each pile especially when they are narrowly spaced. Prakash (1962) and Davisson and Salley (1970) conducted some tests on model pile groups in sand which indicated the magnitude of this interaction.

2.2 STATIC LOADING

Carter (1984) presents a review in detail for the static lateral loading behaviour of piles. However, the author wishes to give a summary of what Carter (1984) has presented in his thesis.

The methods available to predict the lateral load behaviour of piles can be divided into two categories. First is experimental methods which uses model piles and full scale piles. the author used a few full scale pile test results for the analysis in this thesis.

Analytical method available which give reasonably accurate solutions can be categorised as follows:

1. Elastic continuum methods
2. Winkler Methods

2.2.1 Model Tests

The model tests are preferred in research because it is a relatively cheap method. Further, the tests can be done in a controlled environment. The stresses and strains in the pile can be monitored with greater accuracy. The movement of the soil surrounding the pile can be observed.
The researchers are able to understand the mechanism involved in the soil pile system thoroughly.

However, care must be taken when this is used to predict the behaviour of prototype behaviour because of the large difference in dimensions. It is also difficult to have the foundation material which models the field situation. If the prototype soil is used, the model bed must be subjected to a much higher gravitational force than the prototype. One method of achieving a high stress gradient in the sand is to perform the tests inside a centrifuge.

Another method produces a high gradient by forcing water to flow down through the soil bed. However, by analysing the pile segmentally researchers have attempted to avoid the difficulties associated with testing piles at high artificial gravities.

2.2.2 Analytical Methods

2.2.2.1 Elastic Continuum Methods
The elastic continuum technique was developed to account for the continuity of the soil mass. The concept of representing the soil as an elastic continuum was proposed by Mindlin (1936) who presented solutions for the problems of the loaded beam on the soil surface. These solutions were later adapted by other researchers to investigate the response of the laterally loaded piles; Douglas and Davis (1964) and Spillers and Stoll (1964) and Poulos (1971). The elastic continuum solutions are based on the Young’s modulus of the soil. Again accuracy of this method depends on the accuracy of the soil properties. Further, this method only leads to approximate solutions because the behaviour of soil is highly non linear.

2.2.2.2 Winkler Method
This was originally proposed by Winkler (1867). Winkler suggested that the soil supporting a foundation could be considered as a series of closely spaced independent springs (Fig. 2.2). The soil pressure $p$ is related to the lateral deflection $y$ by the coefficient of subgrade reaction $k_h$.

\[ p = k_h y \quad \text{Equation 2.1} \]
Fig. 2.2 Subgrade reaction model for laterally loaded piles suggested by Winkler.

The relationship between soil pressure $p$ and deflection $y$ is highly non-linear. Reese and Matlock (1956) suggest that the adoption of a linearly increasing modulus of subgrade reaction with depth takes care of soil yield and non-linearity to a certain amount. Madhav et al (1971) have employed a solution which used an elasto-plastic Winkler model, while Kishida and Nakai (1977) have employed a method based on a bi-linear soil model. However, the $p - y$ concept originally proposed by McClelland and Focht (1956) is the most widely used method.
Fig. 2.3 Concept of $p-y$ curves

Fig. 2.3 explains the concept of $p-y$ curves. If it is possible to predict a set of $p-y$ curves, such as those shown in Figure 2.3, the pile behaviour under lateral loading can be determined by solving the differential equation.

$$EI \frac{d^4y}{dx^4} + E_s y = 0$$  \hspace{1cm} \text{Equation 2.2}

where

- $y$ = deflection
- $x$ = length along the pile
- $EI$ = flexural stiffness of pile
- $E_s$ = soil modulus
Matlock (1970) describes a procedure for constructing $p-y$ curves for soft clays which were developed from the results of full scale pile tests carried out at Lake Austin, Sabine, Texas. He found that he was best able to match the computed results by using a cubic load deflection curve. The curve passes through the origin and meets a plateau represented by the ultimate resistance of the soil. In addition the cubic is described by two points which are functions of $\varepsilon_{50}$, the strains corresponding to half the maximum principal stress difference in an unconsolidated undrained test as illustrated in Figure 2.4.

$$Y_c = 2.5\varepsilon_{50}d$$

Fig. 2.4 Construction of $p-y$ curves for soft clay below water table (after Matlock, 1970)

Reese et al (1974) describes a procedure for constructing $p-y$ curves for saturated sands based on the Mustang Island pile tests. They approximated the true non-linear response of the soil by a curve composed of three straight lines and a curve as illustrated in Figure 2.5.
Fig. 2.5 Typical family of p - y curves for sand (after Reese et al, 1974).

Reese et al (1975) describes a procedure for constructing p - y curves for stiff clays. However, the above procedures are largely based on a very limited number of pile load tests.

The subgrade reaction approach has been widely used to predict the lateral loading behaviour of piles because this enables to take care of factors such as non-linearity, variation of soil stiffness with depth and layered soil profile.

2.2.2.3 Finite Element Methods

Finite element methods are preferred by researchers because they are able to take into account the continuity of the soil, as well as incorporating a non-linear soil response. The separation occurring between the pile and the soil also can be taken care of by this technique. Desai and Appel (1976) developed a three dimensional finite element solution for the laterally loaded pile problem. Kuhlemeyer (1979), Randolph (1981) and Baguelin et al (1979) have also presented more economical solutions. Baguelin et al (1977) used a combination of both finite element method and the elastic continuum theory to examine the mechanism of the lateral reaction of piles in an
elas-to-plastic medium. Winnicki and Zienkiewicz (1979) also presented a finite element solution. In this case the soil behaviour is represented by a pseudo-visco-elastic response.

Although these methods provide the researcher with a very detailed picture of the stress field within the soil, their accuracy depends on the accuracy of the soil properties.

2.3 Cyclic Loading

The methods used to predict the response of piles subject to static lateral loading have been used to predict that of cyclic lateral loading. However, because the soil behaviour under cyclic loading is highly non-linear and not very well understood, modifications becomes necessary in every load cycle. Although researchers suggested some empirical solutions (Randolph and Wroth (1978), Prevost (1978) etc), these are only applicable to particular cases. Thus researchers still depend on full scale pile test results for the prediction of cyclic response. The elastic continuum method also was used to predict cyclic response. Poulos (1981) modified the Boundary Element Method to account for degradation of the soil properties.

Hettler and Gudehus (1980) produced several empirical expressions which predict the cyclic behaviour of piles.

Matlock and Reese et al developed p-y criteria to model cyclic response of piles in sand. Information regarding this cyclic p-y criteria are reported by Reese and Allen (1977) and Reese and Desai (1977). The cyclic response of piles in clay also has been analysed using p-y criteria. However, this modelling only predicts maximum likely pile deformations instead of considering cycle by cycle approach.

Apart from analytical methods, there are some experimental methods available to predict the cyclic response. They are full scale pile tests, model tests etc. The behaviour of a single pile and two pile groups embedded in dry sand under lateral loading is presented by Finn and Gohl (1987). The single pile was subjected to both sinusoidal and random earthquake motion;
the pile groups were subjected to sinusoidal wave motion only. These centrifugal modelling studies were carried out at the California Institute of Technology (Caltech), Pasadena, California. Ting and Scott (1984) have subjected model single piles and pile groups consisting of two and four piles to slow cyclic loading using a pneumatic piston and to dynamic (high frequency) loading using an air driven eccentric mass shaker. The model piles were placed in a saturated sand foundation.

2.3.1 Repeated loading or one way cyclic loading characteristics

Figure 2.6 indicates repeated load test results reported by Prakash et al (1971). In cohesive soils Davisson and Prakash (1963) observed that repeated loading is likely to remould the soil in the top few pile diameters by repeated shearing deformations with a resultant reduction in soil strength and stiffness. It was further observed that the repeated loading caused permanent deformations of the soil near the ground surface leaving a gap between the soil and the pile. No such gaps were seen to develop between the pile and soil in cohesionless materials.

![Graph showing cyclic load tests](image)

Fig 2.6 The "cyclic" load tests of Prakash et at (1971)
2.3.2 Two way cyclic loading characteristics

As a pile is cycled by lateral loadings, soil near the surface is molded away, a gap formed behind the pile, and water drawn into this gap. As the pile is pushed in the other direction, water and some soil are expelled, soil molded away ahead of the pile, a gap developed at depth behind the pile, and the process repeated. The upper zone of soil in which lateral stresses are not sufficient to close such gaps is termed the zone of unconfined response. The zone at depth where the overburden pressures are sufficient to close any gaps, is termed the zone of confined resistance. Figures 2.7 and 2.8 show experimentally derived lateral load \((P)\) - deformation \((Y)\) results for soil resistance in unconfined and confined response, respectively. Once the cyclic load level passes 70 to 80 percent of the static resistance of the soil, significant deterioration in resistance develops. Deterioration of resistance is achieved in a relatively few number of cycles, generally less than 10 to 20. The dog-bone shaped hysteresis loops in Figure 2.7 are caused by the gapping processes referred to in the zone of unconfined response. The hysteretic response of the soil in contact with the pile in the zone of confined response (Fig. 2.8) is similar to that derived from cyclic triaxial or simple shear tests. As in such tests, the lateral cycling of the soil decreases peak strength and stiffness and increases hysteresis.

Fig. 2.7 Cyclic Lateral Load-Displacement Characteristic of the soil in the One of unconfined response (after Bea et al, 1980).
Fig. 2.8 Cyclic Lateral Load-Displacement Characteristics of soil in zone of confined response (after Bea et al, 1980)

Matlock (1970) also suggests similar effects for two way lateral loading.

2.3.3 The Degradation of the Soil

The rapid changing response of a pile during cyclic lateral loading is produced by the degradation of the soil pile system. The degradation can take two forms. "Material" degradation causes changes in the soil properties, as evidenced by increased pore pressures, changes in the density and rotation of principal stress directions. "Mechanical" degradation is caused by plastic deformation of the soil, and occurs when residual pressures develop along the pile or separation takes place between the soil and the pile. The interaction of material and mechanical degradation during cyclic loading can increase the deflection, rotation and hence bending stresses along the pile, and thereby reduce the ultimate lateral resistance.

The degradation factors for soil modulus, $D_e$, and yield pressure, $D_p$, are defined as:
\[ D_E = \frac{E_C}{E_s} \]  \hspace{1cm} \text{Equation 2.3}

and \[ D_p = \frac{P_{yc}}{P_{ys}} \]  \hspace{1cm} \text{Equation 2.4}

where \( E_C \) = soil modulus after cyclic loading
\( E_s \) = soil modulus for static loading
\( P_{yc} \) = yield pressure after cyclic loading
\( P_{ys} \) = yield pressure for static loading

Based on the data summarized by Idriss et al. (1978), both the yield pressure degradation factor \( D_p \) and the soil modulus degradation factor \( D_E \), can be expressed as follows:

\[ D_p = D_E = N^{-t} \]  \hspace{1cm} \text{Equation 2.5}

where \( N \) = number of cycles
\( t \) = degradation parameter depending on cyclic axial strain

2.3.4 The Gapping Model

Swane and Poulos (1984) suggested a soil-pile interaction model based on subgrade reaction theory where the pile is analysed as a thin elastic beam. The soil along both sides of the pile is represented by a set of springs and friction-slider blocks, the response of each spring block system being elasto-plastic. A description of the model is shown in Figure 2.9(a).
Variations in the overall soil-pile response are made by allowing the spring stiffness $k$, and yield pressure $p_y$ in tension (that is, at the back of an advancing pile), to differ from the compression spring stiffness and yield pressure values (that is, at the front face), as shown by Swane and Poulos (1982) as reported by Swane and Poulos (1984). For piles embedded in cohesive soils, a "gap" soil model has been developed which allows separation between the soil and the back face of a pile by defining the tension springs as very weak. The lateral resistance offered by the tension springs is therefore negligible. Complete separation of the soil from both sides of a pile occurs at pile nodes where both nodal springs are in tension. The yield pressure of the tension spring is selected so that it always remains elastic, thereby allowing the location of recontact of the pile with the soil to be recovered. The behaviour of the gap model is illustrated in Figure 2.9(b).

By defining an initial distribution of in-situ lateral confining pressure along a pile, the analysis is able to simulate the progressive formation of permanent gaps during cyclic loading. The method allows the influence of different levels of confining pressure to be investigated, and represents a consistent approach which requires no empirical assumptions. The analysis is illustrated in Figure 2.10.
Fig. 2.10 The characteristic cyclic reaction-deflection behaviour of soil along a laterally loaded pile

A more detailed treatment of the method is given by Swane (1983), including a description of the computer program CYCLAP.

2.3.5 Methods for Determining "p - y" Curves for Cyclic Loading Analysis

2.3.5.1 Soft clays below the water surface

Matlock (1970) modified static loading "p - y" curves to enable handling cyclic loading cases. This is shown in Figure 2.11.
Fig. 2.11 Matlock's "p - y" curves construction for cyclic loading of soft clays below water table.

Up to point 'b' defined by a deflection of $3Y_{50}$, there are possible substantial deflections with no deterioration in resistance compared to the static curve. This is illustrated in Figure 2.11(a) and (b). At point 'b', the resistance under cyclic loading has reached a maximum even at greater depths. Complete loss in resistance is assumed to occur at soil surface when deflection at that point reaches $15Y_{50}$, as indicated by the point 'd' in Figure 2.11(b).

For depths other than the ground surface level and up to the critical depth $L$, the pseudo plastic resistance of the soil is considered to reduce to a level defined by equation 2.6.
\[
\frac{p}{p_u} = 0.72 \left[ \frac{L}{L_c} \right]
\]

Equation 2.6

\[L_c = \frac{6}{\gamma} \left( \frac{C_u}{C_u} + \frac{0.5}{D} \right)
\]

Equation 2.7

where \( \gamma \) = effective unit weight of the soil
\( C_u \) = the undrained cohesive strength of the soil
\( D \) = Diameter of the pile.

Figure 2.11(c) indicates the post cycle behaviour. The point ‘f’ refers where the degradation of the soil system has occurred. After any particular point, such as point ‘f’ in Figure 2.11(c), has been reached, rebound to zero resistance is assumed to occur along a line parallel to a secant through point ‘a’, i.e. fg in Figure 2.11(c). The resulting slack zone and reloading path are indicated by fgo (unloading) through ‘ogf’ towards ‘e’ (reloading). In, figure 2.11(b), the curve through ‘abh’ applies for all values of strain greater than \( 3(Y/r) \) for \( L > L_c \). Where \( L \leq L \), the curves defined by ‘abej’ and ‘abd’ apply. The nature of the family of \( p - y \) curves so constructed is indicated in Figure 2.12.

![Lateral Soil Resistance Ratio vs Lateral Pile Strain](Image)

Fig. 2.12 Family of \( p - y \) curves for a typical pile in cohesive soil.
2.3.5.2 Stiff clays

Similar techniques have been developed for stiff clays both above and below the water table. The procedures for developing $p - y$ curves are presented by Reese and Welch (1975) and Reese, Cox and Koop (1975) for stiff clays above the water table and below the water table respectively.

2.3.5.3 Sand

To handle cycle loading, modification is done to the procedure developed by Reese, Cox and Koop (1974) for static loading. The only modification to the static technique is that cyclic values for the empirical adjustment factors $A$ and $B$ are used. These are all explained by Hughes, Goldsmith and Fendall (1978).

2.3.6 Dynamic Loading

The analysis of the behaviour of the pile subject to dynamic lateral loading is much more complex and less well understood than that of static loading problems. It is generally considered that the dynamic loading analysis can be done by superimposing the following:

(a) The behaviour of the upper part of the pile where the response is affected by the presence of the free soil surface.

(b) The behaviour of the lower part of the pile where the surrounding soil mass dominates the response.

Larkin (1986), suggests that the seismic response of a pile may be considered as three separate zones such as the near surface region, the transition zone and the deep region. Larkin also suggests that it is analytically convenient to decouple the problem into two effects:

(i) A kinematic interaction effect involving the pile responding to the base rock motion with no superstructure present

(ii) An inertial interaction effect which allows for the effect the acceleration of the superstructure mass has on the pile.
In general the method of analysis for piles subject to dynamic or earthquake loads is to assume the system behaves as a damped cantilever with a single concentrated mass at the top of the pile. Novak (1974 b) has observed that in practice the damping derived from a pile-soil system is difficult to assign thus design values are usually guessed. The behaviour of such piles has been investigated, for example, by Nair (1969).

The trends in the development of analytical techniques as reported by Blaney, Kausel and Roesset (1976) have been along two principal lines.

(1) the use of a continuous elastic model (for example, Novak 1974 a and Tajimi 1969),

(2) the use of a discrete model with lumped masses, springs and dash pots (for example, Ogata and Kotsubo 1966 and Penzien et al 1964).

The continuous model has the advantage that it automatically incorporates the mass densities of the soil and pile as well as the effect of the radiation damping. The method is however restricted to linear soil behaviour.

On the other hand the discrete model encounters the difficulty of defining equivalent soil masses and fictitious dash pots to simulate radiation damping. Non-linear soil behaviour is accommodated varying the force deformation characteristics of the springs.

Blaney et al (1976) proposes a discrete model which uses a finite element technique to reproduce the soil about the pile. The lack of agreement observed in the different techniques, as indicated by Figure 2.13, serves only to highlight the need to improve our understanding of the mechanism involved.
Fig. 2.13 Comparison of Blaney and Novak's solution for a pile subject to dynamic lateral loads.

Nogami and Chen (1987) proposed a dynamic non-elastic Winkler soil model and a method to define its parameters. The model is shown in Figure 2.14.

Fig. 2.14 Proposed Dynamic Non-linear Winkler Model.
In the proposed model, the soil is represented by the non-elastic near-field and the elastic far-field element. The non-linear behaviour is considered through the non-linear near field element. The non-linear characteristics of the element can be completely defined from the "static-cyclic" p - y curve.

The accuracy of this model should be verified with full scale pile tests.
CHAPTER 3

NON LINEAR SOIL MODEL

3.1 INTRODUCTION

Over the past decade the investigation of single piles and pile groups has been the subject of several researchers. The simplest model used by most of the researchers is the Winkler model.

The resistance of the soil at any depth can be represented by a stiffness of a spring at the same depth. The original soil models proposed by Winkler and Vesic assume a linear soil response. However, the true behaviour of soil is highly non linear especially when it is loaded cyclically. The non linear p - y curves are considered to be the most intricate soil model which was produced by Reese and Matlock. However, a simple soil model was thought to produce the soil behaviour with similar accuracy to the p - y curves. This was first proposed by Pender in 1983. Carter (1984) and Ling (1988) analysed the versatility of this model for static loading. This soil model is based on an initial stiffness $k_o$ and a yield pressure, $P_{ult}$, at a particular depth along the pile. The values of $k_o$ and $P_{ult}$ can be deduced from the soil properties surrounding the pile.

3.2 DERIVATION of the SOIL MODEL

The load deflection relationship of the soil at a particular depth is described by the initial coefficient of subgrade reaction $k_o$ and yield pressure $P_{ult}$ of the soil at that depth. This is illustrated in Fig. 3.1.

Fig. 3.1 : Load Deflection Curve for the author's soil model.
The shape of the curve is given by the equation

\[ y = \frac{p}{k_0} \left[ \frac{p^{n}_{ult}}{p^{n}_{ult} - p^n} \right] \]  

Equation 3.1

where

\( y \) = the displacement at any point

\( p \) = the current contact pressure

\( n \) = an index that controls the non linearity

\( k_0 \) = the small strain coefficient of subgrade reaction

\( p^{o}_{ult} \) = the maximum soil pressure that soil can sustain

Equation (3.1) can be written as

\[ \frac{y}{p} = k_0 \left[ \frac{1}{1 - \left( \frac{p}{p^{o}_{ult}} \right)^n} \right] \]  

Equation 3.2

\[ y k_0 \left( p^{n}_{ult} - p^n \right) = p p^{n}_{ult} \]  

Equation 3.3

By differentiating this the tangent modulus can be obtained

\[ k_0 \left( p^{n}_{ult} - p^n \right) dy - n y k_0 p^{n-1} dP = p^n_{ult} dP \]

\[ k_0 \left[ 1 - \left( \frac{p}{p^{o}_{ult}} \right)^n \right] dy = \left[ 1 + n k_0 \frac{y}{p} \left( \frac{p}{p^{o}_{ult}} \right)^n \right] dP \]  

Equation 3.4

Substituting equation (3.2) into equation (3.4) gives

\[ k_0 \left[ 1 - \left( \frac{p}{p^{o}_{ult}} \right)^n \right] dy = \left[ 1 + \frac{n k_0 \left( \frac{p}{p^{o}_{ult}} \right)^n}{k_0 \left[ 1 - \left( \frac{p}{p^{o}_{ult}} \right)^n \right]} \right] dP \]

\[ k_0 \left[ 1 - \left( \frac{p}{p^{o}_{ult}} \right)^n \right]^2 dy = \left[ 1 + (n - 1) \left( \frac{p}{p^{o}_{ult}} \right)^n \right] dP \]
\[
\frac{dP}{dy} = \frac{k_o \left(1 - \left(\frac{p}{p_{ult}}\right)^n\right)^2}{1 + (n - 1) \left(\frac{p}{p_{ult}}\right)^n}
\]

Equation 3.5

tangent stiffness

The values necessary to describe this curve are initial small strain stiffness \(k_o\), the ultimate pressure \(P_{ult}\) capable of being resisted by the soil and the index \(n\). The values of \(P_{ult}\) and \(k\) can be deduced from the site investigation results around the piles. This is explained in the sections 3.3 and 3.4 of this chapter. Carter (1984) and Ling (1988) suggested a value of 0.2 and 1 for \(n\) will give the most suitable match for clays and sand respectively for static loading. The author also found that this was true for cyclic loading as well.

3.3 DETERMINATION of the ULTIMATE SOIL PRESSURE for the FIRST LOADING CYCLE

There are several papers available with different opinions about the magnitude of ultimate pressure. Still there is no universally accepted method to determine \(P_{ult}\).

3.3.1 Ultimate Soil Pressure For Cohesionless Soils

Most of the limiting pressure distributions are normally assumed to increase linearly with depth. Examples of this type of distribution are illustrated in Figure 3.2 (Broms 1964; Minikin 1950; Petrasovits and Awad 1972).

However, several non linear distributions have also been proposed. Brinch - Hansen (1961) presented a non linear distribution which is depicted in Figure 3.2 as well.
Carter (1984) and Ling (1988) found that a slope of 5K_L gave the best results for static loading. However, the author found that this couldn’t take care of the non linearity when the load is cyclic. The author suggests a slope between 3K_L and 5K_L would give the best results as far as the cyclic loading is concerned. However, this has to be confirmed by reanalysing considering yield of the pile.

3.3.2 ULTIMATE SOIL PRESSURE FOR COHESIVE SOILS

Like cohesionless soils, there are several distributions suggested by different researchers for cohesive soils. Matlock (1970) suggested that ultimate pressure distribution would increase linearly from 3S_u at the surface to 9S_u at depth. However, Reese et al (1975) suggested that less support could be provided at the surface, and greater support at depth. Stevens and Audibert (1979) further analysed these distributions and suggested that distributions presented by Reese and Matlock are underestimated. These are depicted in Fig. 3.3.
Fig. 3.3 Lateral bearing capacity factor $N_p$ with respect to normalised depth (after Stevens et al 1979)

Carter (1984) and Ling (1988) used the values of $N_p$ suggested by Stevens et al. The non-linear portion of the curve was approximated by a straight line varying from $N_p = 5$ at the surface to $N_p = 12$ at a depth of $z = 3.5 \text{ d}$.

3.4 Determination of Ultimate Soil Pressure for the Subsequent Cycles

The program written by Carter (1984) is able to handle cyclic loads as well. However, the permanent set created according to this theory is very much less than that of recorded one. To eliminate this defect, the author introduced a parameter $\text{COEFF}$. This is still not a very good solution for handling reverse cycles.

Fig. 3.4 Change in ultimate pressure for cyclic loading
The ultimate pressure $P_{ult}$ can be calculated from the method explained in Section 3.3. The pressure $P_1$, given in Fig. 3.4, can be calculated for any spring down the pile. This is possible from the Winkler's fundamental assumption

$$P = k_h y$$

Equation 2.1

Rearranging equation 3.1,

$$y = \left[ \frac{P}{k_0} \right] \left[ \frac{P_{ult}}{P_{ult} - P^n} \right]$$

$$\rightarrow P \left[ 1 + \frac{k_0 y P^{n-1}}{P_{ult}^n} \right] = k_0 y$$

Equation 3.6

The pressure $P_1$ can be solved for iteratively from equation (3.6). However, for second and subsequent unloading cycles of loading the ultimate pressure is assumed to be increased. This new value of ultimate pressure, $P_{ult}$, is equal to $P_1 + P_{ult}$ as illustrated in Fig. 3.4. However, the author found that this is smaller than it should be. Thus, the author introduced a constant parameter COEFF, to increase the value of $P_{ult}$, i.e.

$$P_{ult} = P_{ult} + COEFF \times P_1.$$  

Equation 3.7

3.5 DETERMINATION of the INITIAL SOIL STIFFNESS for the FIRST CYCLE

There are several methods available to calculate initial small strain stiffness. The accuracy of these methods are often criticized. However, differences in these methods becomes less significant compared to the inaccuracy in determining in-situ properties of soil.

The best method to determine $k_0$ is to back analyse the full scale pile tests with reasonably accurate soil data.

Terzaghi (1955) suggested that coefficient of subgrade reaction $k_h = n_h x$ where $n_h$ equals the constant of subgrade reaction as illustrated in Figure 3.5.
Fig. 3.5 Variation of Horizontal Subgrade Reaction with Depth

Load test data by Alizadeh & Davisson (1970), Robinson (1979) and Bhushan and Askari (1983) have shown that the constant of horizontal subgrade reaction, \( n_h \), is not a constant but decreases with increasing horizontal load. This has been taken care of in the author's soil model by taking the tangents of the curve at different load level.

From the elastic beam theory and Winkler model, Vesic suggested an expression for modulus of horizontal subgrade reaction \( K \) as in equation 3.8.

\[
K = \frac{0.65 E_s}{\left(1 - \gamma_s^2\right)} \left[ \frac{E_B^4}{E_p - p} \right]^{\gamma_s^2}
\]

Equation 3.8

Carter (1984) and Ling (1988) have shown that \( \left[ \frac{E_B^4}{E_p - p} \right] \) is approximately equal to 1 for most practical situations.

Thus equation 3.8 can be rewritten as

\[
K = \frac{0.65 E_s}{1 - \gamma_s^2}
\]

Equation 3.8 suggested by Vesic doesn't reveal the effect of pile width. This equation is believed to be valid only for the pile width of 1 m. Carter (1984) achieved better agreement using a linear relationship between the modulus of subgrade reaction and the width of the pile. Thus \( K \) may be expressed as
\[ K = \frac{0.65 E_s B}{1 - \gamma_s^2} \]

To correct this equation dimensionally a reference diameter \( B_{\text{ref}} \) is introduced. Thus

\[ K = \frac{0.65 E_s B}{(1 - \gamma_s^2)B_{\text{ref}}} \]

where \( B_{\text{ref}} = 1.0 \text{ m} \)

The coefficient of subgrade reaction, \( k_h \) is therefore

\[ k_h = \frac{K}{B} = \frac{0.65 E_s}{(1 - \gamma_s^2)} \quad \text{Equation 3.9} \]

Since a pile has soil in contact with both sides Bowles (1982) suggested that the modulus of subgrade reaction obtained from equation 3.9 should be doubled. Thus

\[ k_h = \frac{1.3 E_s}{(1 - \gamma_s^2)} \quad \text{Equation 3.10} \]

However, Carter (1984) and Ling (1988) found that the factor of 1.0 instead of 1.3 gave closest agreement with full scale pile test results. The author also supports this conclusion. Therefore

\[ k_h = \frac{1.0 E_s}{(1 - \gamma_s^2)} \quad \text{Equation 3.11} \]
3.5.1 Initial Coefficient of Subgrade Reaction for Cohesionless Soils

Equations which predict the shear modulus of sands, for small strains, were presented by Hardin and Richart (1963). The two equations developed for round grained materials and angular grained materials are presented below.

For round grained materials

\[
G = \frac{6906 (2.17 - e)}{1 + e} (\sigma')^{0.5} \quad \text{(kPa)} \quad \text{Equation 3.12}
\]

and for angular grained materials,

\[
G = \frac{3230 (2.97 - e)}{1 + e} (\sigma')^{0.5} \quad \text{(kPa)} \quad \text{Equation 3.13}
\]

The shear modulus of the soil is related to the elastic modulus of the soil, \(E_s\), by the following equation.

\[
E_s = 2G (1 + \gamma) \quad \text{Equation 3.14}
\]

Carter (1984) used these equations to calculate initial \(k_0\). However, the method used by Ling (1988) was found to be attractive by the author. This method is based on the proposal by Seed et al (1986).

Seed et al. presented the following equations.

\[
V_s = 220 N_{60}^{0.17} D^{0.2} \quad \text{(fps)} \quad \text{Equation 3.15}
\]

where \(V_s\) = in-situ shear wave velocity

\(N_{60}\) = N-value measured in SPT test delivering 60% of free fall energy. A summary of energy ratios for SPT procedures is presented in Table 3.1.

\(D\) = depth below surface in feet.

The shear modulus \(G_{\text{max}}\) is related to the shear wave velocity \(V_s\) by the equation

\[
G_{\text{max}} = \frac{\gamma V_s^2}{g}
\]
Hence the relationship between \( G_{\text{max}} \) and the SPT - N value reduces to

\[
G_{\text{max}} = 7.22 \, N_{60}^{0.34} \, D^{0.4} \, \rho \quad (\text{KNm}^2)
\]

Equation 3.16

where the depth \( D \) is in meters and the density \( \rho \) of soil is in \( \text{kg m}^{-3} \)

<table>
<thead>
<tr>
<th>Country</th>
<th>Hammer Type</th>
<th>Hammer Release</th>
<th>Estimated Rod Energy (Percent)</th>
<th>Correction Factor for 60 Percent Rod Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Japan*</td>
<td>Donut</td>
<td>Free-fall</td>
<td>78</td>
<td>78/60 = 1.30</td>
</tr>
<tr>
<td></td>
<td>Donut</td>
<td>Rope and pulley</td>
<td>67</td>
<td>67/60 = 1.12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>with special throw</td>
<td></td>
<td></td>
</tr>
<tr>
<td>United</td>
<td>Safety</td>
<td>Rope and pulley</td>
<td>60</td>
<td>50/60 = 1.00</td>
</tr>
<tr>
<td>States</td>
<td>Donut*</td>
<td>Rope and pulley</td>
<td>45</td>
<td>45/60 = 0.83</td>
</tr>
<tr>
<td>Argentina</td>
<td>Donut</td>
<td>Rope and pulley</td>
<td>45</td>
<td>45/60 = 0.83</td>
</tr>
<tr>
<td>China</td>
<td>Donut</td>
<td>Free-fall</td>
<td>60</td>
<td>60/60 = 1.00</td>
</tr>
<tr>
<td></td>
<td>Donut</td>
<td>Rope and pulley</td>
<td>50</td>
<td>50/60 = 0.83</td>
</tr>
</tbody>
</table>

*Non-natural: PT results have additional corrections for borehole diameter and frequency effects.
*Prudent: method in the United States today.
*Phan-type: hammers develop an energy ratio of about 60 percent.
*SACE: Seed et al. (1984).

Table 3.1 Summary of energy ratios for SPT procedures

If the water table is at the ground surface Seed et al suggests that \( G_{\text{max}} \) may be expressed approximately by

\[
G_{\text{max}} = 3609 \left( \frac{N}{1}_{60} \right)^{0.33} \left( \sigma'_0 \right)^{0.5}
\]

Equation 3.17

where

\[
\left( \frac{N}{1} \right)_{60} = \frac{N_{60}}{C_N}
\]

\( C_N \) = SPT Overburden correction factor

\( \sigma'_0 \) = effective stress in \( \text{kNm}^{-2} \)

The relationship between \( C_N \) and the effective overburden pressure shown in Figure 3.6 represents a consensus of published proposals.
Fig. 3.6 SPT Overburden correction factor

The shear modulus of the soil is related to the elastic modulus $E_s$ and the Poisson’s ratio $\gamma_s$ of the soil by

$$E_s = 2G \left[1 + \gamma_s\right]$$

The initial small strain coefficient of subgrade reaction can thus be found using the equation 3.11. Table 3.2 lists typical values of Poisson's ratio for different materials, as given by Bowles (1982).
<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, Saturated</td>
<td>0.4 - 0.5</td>
</tr>
<tr>
<td>Clay, Unsatrated</td>
<td>0.1 - 0.3</td>
</tr>
<tr>
<td>Sandy Clay</td>
<td>0.2 - 0.3</td>
</tr>
<tr>
<td>Silt</td>
<td>0.3 - 0.35</td>
</tr>
<tr>
<td>Sand, Dense</td>
<td>0.2 - 0.4</td>
</tr>
<tr>
<td>Coarse</td>
<td></td>
</tr>
<tr>
<td>Fine Grained</td>
<td></td>
</tr>
<tr>
<td>(e=0.4-0.7)</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
</tr>
<tr>
<td>Rock</td>
<td>0.1 - 0.4</td>
</tr>
<tr>
<td>Loess</td>
<td>0.1 - 0.3</td>
</tr>
<tr>
<td>Ice</td>
<td>0.36</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Table 3.2 Typical range of Poisson's ratio

Hence for the case where the water table is below the ground surface, the initial small strain coefficient of subgrade reaction is given by

\[
k_0 = 14.44 \, N_{60}^{0.34} \, D^{0.4} \, \rho \left( \frac{1 + \gamma_s}{1 - \gamma_s} \right) \, \text{kN m}^{-3}\]

Equation 3.18

for the case where the water table is at the ground surface, the initial small strain coefficient of subgrade reaction is given by

\[
k_0 = 7219 \left( N_1 \right)_{60}^{0.33} \, \left( \sigma' \right)^{0.5} \left( \frac{1 + \gamma_s}{1 - \gamma_s} \right) \, \text{kN m}^{-3}\]

Equation 3.19

3.5.2 Initial Coefficient of Subgrade Reaction for Cohesive Soils

The equation 3.11 can be used to predict \( k_0 \) for cohesive soils as well. Figure 3.7 describes a plot of shear modulus versus shear strain, as proposed by Pender et al (1983). For shear strain of less than \( 10^{-4} \), the ratio of shear modulus to undrained shear strength, \( G/S_u \), takes a value of 2000. The initial coefficient of subgrade reaction calculated using this value tends to overestimate the initial stiffness of the soil. Ling (1988) found that values between 100 and 400 of \( G/S_u \) gave good representation of recorded response. The author also supports this. The value of 100 corresponds to soft clay and the value of 400 corresponds to stiff clay.
3.6 COEFFICIENT of SUBGRADE REACTION for the SUBSEQUENT CYCLES

Long and Reese (1987) suggested that the behaviour of a pile subjected to cyclic lateral loads can be influenced significantly by the following:

(a) the accumulation of progressive permanent strains
(b) the reduction of soil modulus and strength due to cyclic loads
(c) the effect of water entering fissures within the soil and reducing the strength of the soil
(d) the removal of soil particles (scour) due to water entering a gap that forms along the side of the pile.

The author decided to focus only on the degradation of soil since it is very difficult at this stage to analyse all those effects above.

In 1981 a program was initiated to do some soil testing on undisturbed samples obtained from Sabine and Manor sites where lateral - load pile tests were conducted. Controlled - deformation, undrained cyclic triaxial tests
were conducted on isotropically consolidated specimens to determine the stress-strain characteristics of soil subjected to cyclic loads. The effect of cyclic loads on soil behaviour is shown in Fig 3.8 as a plot of the secant modulus of the soil versus the number of cycles. The slope, $t$, of a line passing through the points is a measure of the effect of cyclic loads.

![Graph showing variation of secant modulus with number of cycles.]

**Fig. 3.8 Variation of Secant Modulus with Number of Cycles after Long and Reese (1987).**

Idriss et al (1978) first introduced the $t$ - parameter to express the effect of cyclic loads on reduction in value of secant modulus. The following equations can be used

$$E_N = E_0 N^{-t}$$

where

- $E_N$ = Secant modulus at any cycle, N.
- $E_0$ = Secant modulus at the first cycle of loading
- $N$ = number of cycles of loading, and
- $t$ = degradation parameter.

Plots of $t$ versus axial strain for Sabine and Manor clays are shown in Fig. 3.9 along with the relationship determined by Idriss et al (1978). It can be seen that the $t$ - parameter increases with the magnitude of cyclic axial strain.

Based on these facts, the author decided to calculate the initial coefficient of subgrade reaction at any cycle, $N$, according to the following equation
\[(k_0)_N = k_0 N - (\text{DEGPA} \times \text{DEFL})\]

where \((k_0)_N\) = initial coefficient of subgrade reaction at any cycle, \(N\).

\(k_0\) = initial coefficient of subgrade reaction at first cycle

\(N\) = number of cycles of loading

DEGPA = degradation parameter constant

DEFL = deflection of the pile at the previous cycle of loading

Fig. 3.9 Comparison of \(t\) - Parameters
(after Long and Reese 1987)
CHAPTER 4

PROGRAM OUTLINE

4.1 INTRODUCTION

There are several computer programs available to predict the response of laterally loaded piles. Only some of which consider non linearity of soils. The finite difference program produced by Reese is one example.

The program utilised in this thesis is originally written by Carter (1984) for static lateral loading of piles. This was verified by Ling (1988). This is a finite element program with the soil around the pile modelled as a bed of Winkler springs.

This program was modified by the author to handle varying pile properties with depth and cyclic degradation of soil. The author made use of the subroutine for linear soils to analyse the pile behaviour above the ground surface by nullifying the initial coefficient of subgrade reaction. The program was designed to handle a layered soil profile and include the presence of ground water. The user guide and a sample output is provided in the Appendices.

4.2 FINITE ELEMENT FORMATION

The pile is divided into certain number of elements. Every element has four degrees of freedom as depicted in Fig. 4.1. The soil resistance generated against the pile is represented by two springs attached at the ends of the element.
The deformation of the element is now analysed using basic structural principles. Fig. 4.2 illustrates it very clearly.
The member displacement is expressed as a function of total displacements at the nodes.

That is,

\[
[s] = [a] \cdot [r]
\]

Equation 4.1

where

- \([s]\) = member displacement matrix
- \([a]\) = displacement transformation matrix
- \([r]\) = structure displacement matrix

\((U_i, U_j, \alpha_i, \alpha_j)\) are the member displacement and rotations and \((X_i, X_j, \theta_i, \theta_j)\) are the structure displacements and rotation. By analysing the deformation, the displacement transformation matrix given below can be formulated.

\[
\begin{bmatrix}
\alpha_i \\
\alpha_j \\
U_i \\
U_j
\end{bmatrix}
= 
\begin{bmatrix}
1 & -\frac{1}{H_{ij}} & 0 & \frac{1}{H_{ij}} \\
0 & \frac{1}{H_{ij}} & 1 & \frac{1}{H_{ij}} \\
0 & 1 & 0 & 0 \\
0 & 0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
\theta_i \\
x_i \\
\theta_j \\
x_j
\end{bmatrix}
\]

Equation 4.2

The internal member forces is related to the member displacement by

\[
[S] = [k] \cdot [s]
\]

Equation 4.3

where

- \([S]\) = internal member force
- \([k]\) = member stiffness matrix
- \([s]\) = member displacement matrix

The member stiffness matrix can be obtained by analysing the behaviour of a propped cantilever.
The forces at either end of the segment are directly related to the displacement of the soil springs. Thus the following equations can be derived.

\[ V_i = k_{si} u_i \]  \hspace{1cm} \text{Equation 4.4}

\[ V_j = k_{sj} u_j \]  \hspace{1cm} \text{Equation 4.5}

Thus the member stiffness matrix can be expressed as shown in the equation

\[
\begin{bmatrix}
  M_i \\
  M_j \\
  V_i \\
  V_j
\end{bmatrix} =
\begin{bmatrix}
  4EI & 2EI & 0 & 0 \\
  H_{ij} & H_{ij} & 0 & 0 \\
  2EI & 4EI & 0 & 0 \\
  H_{ij} & H_{ij} & k_{si} & 0
\end{bmatrix}
\begin{bmatrix}
  \alpha_i \\
  \alpha_j \\
  u_i \\
  u_j
\end{bmatrix}
\]  \hspace{1cm} \text{Equation 4.6}

The externally applied forces can be obtained from the relationship

\[ [R] = [g] [S] \]  \hspace{1cm} \text{Equation 4.7}

where \[ [R] \] = applied force matrix
\[ [g] \] = force transformation matrix
\[ [S] \] = member force matrix

Using the principle of virtual work it can be shown that

\[ [g] = [a]^T \]  \hspace{1cm} \text{Equation 4.8}

Therefore

\[ [R] = [g] [S] \]
\[ = [g] [k] [s] \]
\[ = [g] [k] [a] [r] \]
\[ = [a^T k a] [r] \]  \hspace{1cm} \text{Equation 4.9}
Where \( [a\ k\ a]^T \) is referred to as the structure stiffness matrix. Having calculated the structural displacements, the member forces can be determined from

\[
[S] = [ka][r] \tag{Equation 4.10}
\]

Because of the difference in each and every element, the stiffness matrix must be calculated for each element. The global matrix for the pile is then calculated from these matrices.

The size of the global stiffness matrix is NDF x NDF. NDF is the number of degrees of freedom of the pile. However, because of the independent nature of the springs, the vast majority of these terms are equal to zero. In addition, the global matrix is symmetric, therefore, the matrix can be further reduced to NDF x the bandwidth, which is four for this problem.

Carter (1984) used the finite element method in preference to the finite difference method for a number of reasons as explained in his thesis.

The program presented by Carter (1984) allows for the displacements and/or the rotations to be set to zero, at any position down the pile. Fixed values of displacement and/or rotation can only be modelled for linear soils where the number of load iteration is one. For non-linear soils, the deformations increase by the specified value with each iteration.

### 4.3 SOIL SPRINGS

The soil around the pile is modelled as Winkler springs. Winkler springs could be used to analyse non-linear soils as well as linear soils. Although this approach doesn't consider the continuity of the soil, it has been found by Carter (1984), Ling (1988) and the author that it is as accurate as the more intricate continuum solution.
The soil resistance against each element of the pile is lumped into two springs at the ends of the element. The stiffness of each spring is a function of the element, the pile width and the coefficient of subgrade reaction at that depth. Although the spring values are calculated at the ends of an element allowing variation with depth, a linear variation between the nodes is assumed. The spring stiffness is then the average of this variation according to the equation 4.11. This is depicted in Fig. 4.3

\[ k_{si} = \left( k_{h1} + 0.25 \left( k_{hj} - k_{h1} \right) \right) \frac{Ld}{2} \]  

\text{Equation 4.11}

where

- \( k_{si} \) = spring stiffness at node \( i \)
- \( k_{ni} \) = coefficient of subgrade reaction at node \( i \)
- \( k_{nj} \) = coefficient of subgrade reaction at node \( j \)
- \( L \) = length of the element
- \( d \) = pile width.

\[ \text{Fig. 4.3 Variation in the coefficient of subgrade reaction for each element} \]

The assumption of linear variation of stiffness is compatible with the accuracy of the soil data.
This technique is effective in the following:

(1) The boundary of the soil layer is taken to be coincident with one node. The continuity of the pile deformations through the soil layer boundary is provided by the stiffness of the pile. The stiffness of the upper nodal spring is determined from the soil above the boundary.

(2) The presence of the water table is accounted for. The soil above and below water table is considered as two separate layers. The water table is specified by the user to be level with the upper boundary of a soil layer.

(3) The program can also handle a non uniform soil profile

4.4 PILE DIVISION

The pile program is written so that the pile is divided into a specified number of elements, each with four degrees of freedom. The accuracy of the program is dependent upon the size of elements used. Carter (1984) found that the smaller the length of the element the greater the accuracy of the pile response. This concept becomes very important when it comes flexible piles where the displacements arising from bending is significant. Several authors have suggested that an element length equal to half the diameter of the pile is satisfactory. Evangelista and Viggiani (1976) demonstrated clearly that increased efficiency can be achieved by using unequal elements. However, displacements of the base becomes important for short stiff piers. When a uniform soil profile is considered, the program can be used to divide the pile into a specific number of elements, so that the length increases down the pile. The division is done using a geometric series.
Pile Length = $\frac{D}{2} \sum_{n=1}^{nm} r^n$ \hspace{2cm} \text{Equation 4.12}

where \hspace{0.5cm} r = \text{constant}  \\
D = \text{diameter of the pile} \\
\text{nm} = \text{number of pile segments}

The element lengths are then given by

$L(1) = \frac{D}{2} r^1$, \hspace{0.5cm} L(2) = \frac{D}{2} r^2$, \hspace{0.5cm} L(i) = \frac{D}{2} r^i$ \hspace{0.5cm} \text{etc.}

At present, the minimum element size is set as half of the pile diameter. For layered soil profiles, the user specifies the element lengths.

4.5 APPLIED LOADINGS AND NON LINEAR BEHAVIOUR

Carter (1984) suggested that the pile above the ground surface is analysed as a simple structural column with known displacements and rotations at its base. Thus the program does not analyse the pile sections above the ground. Loads applied above the ground level are equivalent to a load and a moment at the ground surface. However, the author made use of the uniform soil layer subroutine program with no soil springs to analyse the behaviour of the pile above ground level.

The non linear soil model uses a decreasing coefficient of subgrade reaction with increasing loads. Thus the load is applied in incremental fashion. This is illustrated in Fig. 4.4.
The coefficient of subgrade reaction for the first load increment is \( k \). After the first increment has been applied and the displacements calculated, a new tangent stiffness is obtained using the equation (3.5). The next increment is then applied and the process repeated until the required final load is reached. By superimposing the incremental responses the total response of the pile is determined. Carter (1984) showed that as the loading increments becomes smaller, the response becomes more accurate.

The number of loading increments is obtained by dividing the maximum horizontal force by the loading increment size. The remainder is applied to the pile as the first load increment. However, if the remainder is less than a predetermined minimum value, the first load increment is equal to the remainder plus the loading increment. The bending moment increment is obtained by dividing the maximum bending moment by the number of loading increments obtained above. This approach ensures that the ratio of bending moment to horizontal force at each increment will be a constant.
4.6 **EFFECT OF AXIAL LOADS**

It is suggested by several authors that if axial load is less than 10\% of the buckling load, then the lateral behaviour of the pile is not affected by the axial load. In most practical situation, the axial load is very small. Thus there is no allowance made in this program for this effect.

4.7 **EFFECT OF CONSTRAINTS**

To account for constraints, such as a pile cap, the program allows for displacements or rotations to be set to zero at any position down the pile.

4.8 **CYCLIC LOADS**

The first cycle is considered as a static loading cycle as analysed by Carter (1984) and Ling (1988). Subsequent cycles are analysed in a manner explained in Chapter 3. The applied loading is provided by the user.

4.9 **PROGRAM OUT-PUT**

The program prints the rotations, deflections, moments and soil reactions at every node down the pile at iterations specified by the user. These prints can be obtained for any cycle. The soil springs, soil stiffness and band matrix also can be printed on request.

In addition, applied horizontal force, deflections and rotations at a particular node of the pile for any range of cycles can be printed.

A sample output is provided in Appendix B.
CHAPTER 5

MODELLING OF FULL SCALE PILE TESTS

5.1 INTRODUCTION

There are various analytical methods available to predict the behaviour of substructures under earthquake loading. However existence of uncertainty of soil properties becomes a problem in class 1 prediction. Hence, it is important to have some reliable full scale test results before any analytical research is carried out. In this thesis, the author has analysed 5 full scale pile test results.

This analysis is carried out to predict the pile behaviour under earthquake loading including pile head displacement, foundation stiffness and pile moment. Foundation stiffness is one of the most important parameters in the non linear soil model analysed. Estimating the ductility demand requires the estimation of displacement of substructure under earthquake loading. Precise estimation of pile head displacement will enable designers to design energy dissipating systems economically. There are only a few full scale pile tests available with detailed information.

Newmans Bridge pile test is one of the first of a number of relatively simple and cost effective load tests that were undertaken on a much wider range of soils and foundation types than previously investigated in New Zealand. A summary of pile load testing projects undertaken in New Zealand is given by Millar (1986). All the tests undertaken before Newmans Bridge test, with the exception of Mangere Bridge cylinder test, reported by Priestley (1974), have been on piles of diameters 0.6 m or less. The Mangere tests were carried out on 1.38 m diameter steel lined cylinders bored into a complex profile of soil and rock layers including marine muds, sands, tuff, clays and siltstone. Newmans Bridge pile test and Maitai River pile tests were carried out on 1.8 m diameter steel lined cylinders. Details of these tests on large diameter piles are given in reports by Phillips and Wood (1987) and (1988).

Austin pile test on stiff clay reported on the paper by Reese Cox and Koop (1975) has only limited information. As far as the cyclic loading is concerned, information in each and every cycle is important. The paper reveals only the final cycle pile head displacement. Although Arkansas River
pile test reported in the papers by Mansur et al (1970) and Alizadeh et al (1970), has the details of pile head displacement over the number of cycle, this has no detail of the permanent set undergone by the pile in each cycle. This information is very important in determining the parameter used in this work, COEFF and DEGPA, reliably.

The pile load test described in the Central Laboratories Report 5-85/17 (1985) by Jennings, Thurston, Edmonds and Millar, is quite elaborate in detail. This was also reported in a paper by Jennings (1984).

5.2 **NEWMANS BRIDGE LOAD TESTS**

Newmans Bridge is on state highway 2 about 150 km north of Wellington. The bridge is located in earthquake zone A and has been designed to resist the most severe earthquake loads specified in the Ministry of Works and Development's (MWD) "Highway Bridge Design Brief". Earthquake resistance is provided by ductility in the columns that are detailed to form plastic hinges just above the connection to the cylinders.

The load tests on the Newmans Bridge are considered to be simple and cost effective. The foundation soils types are considered to cover a wider range than previously investigated. The main objective of this test was to obtain field data on the lateral stiffness of full scale bridge foundations subjected to lateral loads that produce steel reinforcement stresses close to yield levels and to compare test results with analytical predictions.

5.2.1 **Pier Specifications**

The superstructure of the bridge is supported on two piers with single 1.4 m diameter reinforced concrete columns and hammer head type column caps. Each pier has a 1.8 m diameter bored cylinder foundation that consists of a reinforced concrete core cast in a 10 mm thick steel liner. Typical details of the structure are shown in Fig. 5.1 and Fig. 5.2.

The calculated moment capacities and equivalent horizontal forces acting at the test load application point of 7.88 m above bases of the columns were as shown in Table 6.1.
Fig. 5.1 Details of Bridge Piers.
Fig. 5.2 Typical details of Newmans Bridge
<table>
<thead>
<tr>
<th>Moment Capacity</th>
<th>Horizontal force</th>
</tr>
</thead>
<tbody>
<tr>
<td>kNm</td>
<td>kN</td>
</tr>
<tr>
<td>First Yield</td>
<td>2970</td>
</tr>
<tr>
<td>Ultimate Moment</td>
<td>4140</td>
</tr>
</tbody>
</table>

Table 5.1 Yield properties of the piers.

Foundation cylinder details are shown in Table 5.2.

<table>
<thead>
<tr>
<th>Item</th>
<th>Reduced Levels (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pier B</td>
</tr>
<tr>
<td>Top of Cylinder</td>
<td>190.68</td>
</tr>
<tr>
<td>Papa level</td>
<td>187.0</td>
</tr>
<tr>
<td>Bottom of casing</td>
<td>186.0</td>
</tr>
<tr>
<td>Bottom of cylinder</td>
<td>178.5</td>
</tr>
</tbody>
</table>

Table 5.2 Foundation cylinder details.

Transformed section properties were used to model the cylinder taking into account the stiffening effect of the 10 mm steel shell and the main longitudinal reinforcement in the cylinder. Full composite action was assumed between the steel shell and concrete. The influence of the column bars lapping into the cylinder was neglected. Cracked section transformed properties were assumed for the pier columns where cracks formed. The moment of inertias computed using these assumptions were:

\[
\text{Cylinder Moment of inertia} = 0.746 \, \text{m}^4 \\
\text{Column Moment of inertia} = 0.0612 \, \text{m}^4.
\]

A Young's Modulus of 30 GPa was assumed for the concrete in the cylinder and the column.
<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>Dark brown silty sandy TOPSOIL</td>
</tr>
<tr>
<td></td>
<td>-soft, moist.</td>
</tr>
<tr>
<td>1.00</td>
<td>SPT 100 - 1.25m : N = 6</td>
</tr>
<tr>
<td>1.40</td>
<td>Grayish brown gritty SAND</td>
</tr>
<tr>
<td></td>
<td>-light brown, matrix;</td>
</tr>
<tr>
<td></td>
<td>-quartz to feldspar,</td>
</tr>
<tr>
<td></td>
<td>-sandstone, minor</td>
</tr>
<tr>
<td>2.00</td>
<td>SPT 300 - 2.45m : N = 89</td>
</tr>
<tr>
<td></td>
<td>Gravelly to pebbly, sandstone,</td>
</tr>
<tr>
<td></td>
<td>-medium to coarse,</td>
</tr>
<tr>
<td></td>
<td>-pebbles, minor</td>
</tr>
<tr>
<td>3.00</td>
<td>SPT 400 - 3.65m : N = 89</td>
</tr>
<tr>
<td></td>
<td>Gravelly to pebbly, sandstone,</td>
</tr>
<tr>
<td></td>
<td>-coarse, minor</td>
</tr>
<tr>
<td>4.00</td>
<td>SPT 500 - 4.85m : N = 87</td>
</tr>
<tr>
<td></td>
<td>Material as above, becoming coarser with depth</td>
</tr>
<tr>
<td>4.50</td>
<td>100% water loss at 4.5m depth.</td>
</tr>
<tr>
<td>5.00</td>
<td>Grey fine grained PAPA</td>
</tr>
<tr>
<td></td>
<td>-hard, moist.</td>
</tr>
<tr>
<td>5.30</td>
<td>Material as above.</td>
</tr>
<tr>
<td>6.00</td>
<td>U.C.S. at 6.00m : 1.5MPa. In situ water content : 19.2%</td>
</tr>
<tr>
<td>6.20</td>
<td>U.C.S. at 6.20m : 1.5MPa. In situ water content : 19.6%</td>
</tr>
<tr>
<td>6.30</td>
<td>U.C.S. at 6.30m : 1.5MPa. In situ water content : 19.8%</td>
</tr>
<tr>
<td>6.40</td>
<td>U.C.S. at 6.40m : 1.5MPa. In situ water content : 19.2%</td>
</tr>
<tr>
<td>6.50</td>
<td>U.C.S. at 6.50m : 1.5MPa. In situ water content : 19.4%</td>
</tr>
<tr>
<td>6.60</td>
<td>END of BOREHOLE at 6.60m depth.</td>
</tr>
</tbody>
</table>

Fig. 5.3 Borehole Log.
Fig. 5.4 Soil and Rock Properties
5.2.2 Soil Description

Drilling was carried out at two locations, however, only one site was useful for this research. Borehole 2, near Pier C, was about 20 m from Pier B. Consequently reliable soil classification and soil test data is unavailable for soils surrounding Pier B. Details of Borehole 2 are shown in Fig. 5.3. Top of the cylinder was at the same level as the stream bed gravels.

The cylinder at Pier C passes through about 4.5 m of gravels and sands overlying papa of low to moderate strength. At the time of testing the top of the cylinders was about 1.4 m above the stream bed. Soils data from the borehole would be expected to provide a reliable description of the soils surrounding the cylinder.

A summary of the measured soil and rock properties is shown in Fig. 5.4. The gravels and sands had an average SPT value of 25. An average angle of internal friction of 37 degrees for the sands and gravels was obtained from the correlations between the SPT values and friction angles. Reliable rock strength data was unavailable for Pier B. The compression strength of complete rock layer at Pier B was assumed to be 2.0 MPa. It was also assumed that a 3.5 m layer of sands and gravels with the same stiffness properties as indicated by Borehole 2, were overlying papa.

5.2.3 Initial Coefficient of Subgrade Reaction

Initial coefficient of subgrade reaction was calculated using the following expression given by Carter (1984):

\[
k_0 = \frac{1.0 \, E_s}{1 - \gamma}, \quad E_s = 2G(1 + \gamma)
\]

\(\gamma\) was taken as 0.3

For gravels, \(G = 6906 \left(\frac{2.17 - e}{1 + e}\right)^2 \left(\sigma'_s\right)^{0.5}\)

For papa \(\frac{G}{C_u}\) was taken as 300 based on its high strength.

Usually \(\frac{G}{C_u}\) lies between 100 and 200 for clays according to Carter (1984).
Bulk density of papa was calculated using water content of the papa assuming the soil is fully saturated i.e. $S_v = 1$.

The values of e and Bulk density of gravel were estimated from SPT values.

Soil properties are summarised in Table 6.3 and the initial coefficient of subgrade reaction in depicted in Fig. 5.6. Fig. 5.5 shows the details of the piers.

5.2.4 Loading System and Instrumentation

The loading system used is described by Phillips and Wood (1987). The rig applies a horizontal force at the pier top level and pier base level by means of a tension system between adjacent piers. During the loading cycles, the load was not reduced below a "minimum load" of 2.5 kN to remove sufficient sag in the cable.

Loads applied were monitored by strain gauge type load cells. Deflections at the base of the piers were measured by means of electric dial gauges. Horizontal deflections at the tops of the piers were monitored with a theodolite at Pier B and a dumpy level at Pier C. Rotations on the top of the hammer head and at the base of the column were monitored with two force balance type accelerometers.

5.2.5 Pier Top Loading

Six cycles of loading and unloading were applied at the pier tops. In the first two cycles, the peak loads were about 50% and 90% of the maximum peak load. The maximum peak load of 325 kN was applied in the four remaining cycles. This load represents 86% of the lateral load at the pier top to produce first yield in the column reinforcement.

5.2.6 Cylinder Top Loading

Four cycles of loading and unloading were applied at the cylinder tops. The peak loads reached in successive cycles were 388, 510, 511 and 565 kN.
Fig. 5.5 Details of piers B and C

Fig. 5.6 Initial coefficient of subgrade reaction of the soil surrounding piers B and C.
### Pier B

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil Type</th>
<th>$N_s$</th>
<th>$\phi$</th>
<th>$C_u$ MPa</th>
<th>Bulk density</th>
<th>$e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5.5</td>
<td>Sand and gravels</td>
<td>25</td>
<td>35</td>
<td>-</td>
<td>20.00</td>
<td>0.65</td>
</tr>
<tr>
<td>5.5 - 12</td>
<td>Papa</td>
<td></td>
<td></td>
<td>1000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Pier C

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil Type</th>
<th>$N_s$</th>
<th>$\phi$</th>
<th>$C_u$ MPa</th>
<th>Bulk density</th>
<th>$e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 2.9</td>
<td>Sand and gravels</td>
<td>25</td>
<td>37</td>
<td>-</td>
<td>20.00</td>
<td>0.65</td>
</tr>
<tr>
<td>2.9 - 10.5</td>
<td>Papa</td>
<td></td>
<td></td>
<td>1000 -2050</td>
<td>21.11</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.3 Soil properties.
5.2.7 RESULTS

5.2.7.1 Pier Top Load Case

The recorded pier top deflections are shown in Fig. A1. Fig. A2 depicts the predicted response with the non linear soil model. Fig. A3 compares the recorded responses. Similarly Fig. A4, A5 and A6 describes the responses for Pier B.

As described in the report by Wood and Phillips a clear break in the linearity of second loading cycle curves for both piers can be seen at a load of about 160 kN. This change in stiffness corresponds to the loads producing cracking of the concrete in the columns. Back analysis showed that the cracking load corresponded to a modulus of rupture of the concrete of about 0.125 of the 28 day cylinder compression strength. Although the predicted response reasonably resembles the recorded response, as far as a macroscopic analysis is concerned, there are some discrepancies.

Cracked section properties were used for the pile and this gives rise to an overestimation of the displacement in the first cycle. Change of stiffness in the second cycle must be taken care of by changing the pile properties after it has started cracking.

Permanent set recorded in each and every unloading cycle were caused by the yield of the pile since all the measurements are done on the pile. However, prediction of permanent set is propagated by stiffening the soil springs in the unloading cycle. This is quite opposite to what happens in practice.

Further stiffening the soil springs only provide an under estimated permanent set. Therefore the author used another parameter called COEFF to increase the stiffness in the unloading cycle. However, it was difficult to increase the permanent set because the non linear model no longer changes the stiffness markedly within the load of application no matter how large the ultimate pressure is.

Therefore it was decided to reproduce the required response by degradation of the soil i.e. decreasing the initial coefficient of subgrade reaction. However, this is not necessary if the unloading cycles are handled by changing pile properties considering the yield of the pile.
This has resulted in a higher value of degradation parameter (1000) and a value of COEFF of 20 for Pier C. For Pier B, the values are 270 and 0.6. The difference is due to the higher number of soil springs compared to Pier C. Yield of the soil springs in Pier B is higher than that of Pier C.

Since the author modelled the predicted response using higher degradation, this has resulted in higher changes in deflection in last three cycles.

The author further believes that all of the sudden changes in the stiffness in the loading cycles are mainly due to cracking of the pile which had not been taken into account in the analysis.

Recorded, predicted and compared rotation of Pier C and Pier B are shown in Fig. A7 to A12 respectively. Predicted response is reasonably good. Wood and Phillips suggested that because of the small magnitude of the rotations it was not possible to differentiate between permanent rotational sets and errors caused by temperature drift in the instrumentation.

A linear increase of $P_{ult}$ gives a good representation of the ultimate resistance of the soil. But determination of $\beta$, in the equation $P_{ult} = \beta k_p g_z$ becomes important. The value of 3 for $\beta$, as proposed by Broms, seems to be an overestimation for gravels. A value of 1 for $\beta$ gives a good representation for the cyclic analysis of gravels.

The value of $n$ equal to 1.0, as proposed by Carter (1984) seems to give a good cyclic response of gravels.

Papa is thought to have a very high ultimate resistance. However a lower ultimate resistance tended to give a good representation of the cyclic response, due to the reduced amount of degradation involved and an increase in the permanent set. However, the author did not carry out the analysis beyond a reasonable ultimate resistance.
Figures A13 to A18 show the predicted, recorded and compared cylinder top displacement of Pier C and Pier B respectively. There is reasonable agreement between observed and predicted results. In the 4th and 6th cycle it is of interest to note that soil creep has given rise to increase of displacement. Since this is very difficult to model analytically, this also has been taken care of by degradation of soil.

Similarly Figures A19 to A24 describe the cylinder top rotational responses of Pier C and Pier B respectively.

5.2.7.2 Cylinder Top Load Case

This testing was done after the pier top load test of 6 loading cycles. The shapes of the loading curves may have been influenced to some extent by this. The predicted response is slightly underestimated compared with the observed values, as the author expected.

Predicted, recorded and compared cylinder top displacement are shown in Figure A25 to A30 for Pier C and B respectively.

Similarly rotational responses are depicted in Figures A31 to A36.

5.3 Maitai River Bridge Load Tests

The Maitai River study is the second in a series of tests being undertaken to provide measured stiffness parameters for a wider range of soil and foundation types than covered by previous full scale pile tests in NZ.

The Maitai River Bridge is at the mouth of the Maitai River, near Nelson city. This bridge is also located in earthquake zone A and has been designed to resist the most severe earthquake loads. The pier columns have been designed to have a relatively high resistance to lateral load. Like Newmans bridge pier tests, a tension system was used for loading purposes as a cost effective technique. By alternatively tensioning cable anchored at Piers B and E, Pier C was loaded in both directions.
5.3.1 Pier Specifications

The superstructure is supported on four piers with single 1.4 m diameter reinforced concrete columns and hammerhead type column caps. Each pier has a 1.8 m diameter cylinder foundation that consists of a reinforced concrete core cast in a 12 mm thick steel liner. The cylinders were placed by excavation and top driving of the steel shell. Typical details of the structure are shown in Fig. 5.7 and Fig. 5.8.

The calculated moment capacities and equivalent horizontal forces acting at the top of the hammer head are shown in Table 5.4.

<table>
<thead>
<tr>
<th></th>
<th>Moment Capacity kNm</th>
<th>Horizontal Forces kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Yield</td>
<td>3290</td>
<td>432</td>
</tr>
<tr>
<td>Ultimate Moment</td>
<td>4570</td>
<td>600</td>
</tr>
</tbody>
</table>

Table 5.4 Yield properties of hammer head

Foundation cylinder details are shown in Table 5.5.

<table>
<thead>
<tr>
<th>Item</th>
<th>Length m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pier B</td>
</tr>
<tr>
<td>Top of hammerhead to top of</td>
<td>7.150</td>
</tr>
<tr>
<td>cylinder: as measured</td>
<td></td>
</tr>
<tr>
<td>Top of hammerhead to top of</td>
<td>7.621</td>
</tr>
<tr>
<td>cylinder: as designed</td>
<td></td>
</tr>
<tr>
<td>Top of hammerhead to ground</td>
<td>7.15</td>
</tr>
<tr>
<td>level at time of testing</td>
<td></td>
</tr>
<tr>
<td>Length of cylinder as designed /</td>
<td>11.98</td>
</tr>
<tr>
<td>built</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.5 Foundation cylinder details

Transformed section properties were used to model the cylinder taking into account the stiffening effect of the 12 mm steel shell and the main longitudinal reinforcement in the cylinder. Full composite action was assumed between the steel shell and concrete. The influence of the column
Fig. 5.7 Typical details of Maitai River Bridge.
Fig. 5.8 Details of Bridge Piers
bars lapping into the cylinder was neglected. Cracked section properties were assumed where cracks formed. The moment of inertias computed using these assumptions were:

\[
\text{Cylinder Moment of Inertia} = 0.716 \text{ m}^4 \\
\text{Column, Uncracked Moment of Inertia} = 0.189 \text{ m}^4 \\
\text{Column, Cracked Moment of Inertia} = 0.065 \text{ m}^4
\]

Following values of Young’s moduli were obtained for Pier C

\[
\text{Cylinder Young’s modulus} = 38 \text{ GPa} \\
\text{Columns Young’s modulus} = 35 \text{ GPa}
\]

5.3.2 Soil Description

The soil was described as "unweathered to moderately weathered fine gravels in a matrix of yellowish brown silty fine sand with rare clay". In the upper layers soils was described as "medium dense" with layers towards the bottom of the cylinders described as "dense" to "very dense". The investigation showed that the foundation soil was relatively uniform over the extent of the bridge site.

An estimate of the foundation soil density void ratio and angle of internal friction was obtained from the Standard Penetration Test (SPT) results obtained during the investigation drilling. Fig. 5.9 shows the SPT test results. Fig. 5.10 shows the details of the piers.

To eliminate the stiffening effect of soil around Pier B and C columns during testing, the surrounding ground was excavated down to the cylinder tops.

5.3.3 Initial Coefficient of Subgrade Reaction, \( k_0 \)

Soil properties are summarised in Table 5.6 and initial coefficient of subgrade reaction is depicted in Fig. 5.11.
The initial coefficient of subgrade reaction - $k_0$ was calculated using the following expression:

$$k_0 = \frac{1.0 \ E_s}{1 - \gamma_s} \quad \text{and} \quad E_s = 2 \ G(1 + \gamma)$$

$\gamma$ is taken as 0.3

For Gravels, $G = \frac{6906(2.17 - e)^2}{1 + e} \ (\sigma')^{0.5}$ according to equation 3.12.

Fig. 5.9 Standard penetration test results.
Fig. 5.10 Details of the pier B

<table>
<thead>
<tr>
<th>Layer</th>
<th>$N_s$</th>
<th>$\phi$</th>
<th>Bulk density ($kg \cdot m^{-3}$)</th>
<th>$e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 - 3.1</td>
<td>25</td>
<td>35</td>
<td>19.7</td>
<td>0.69</td>
</tr>
<tr>
<td>3.1 - 7.3</td>
<td>55</td>
<td>42</td>
<td>20.4</td>
<td>0.58</td>
</tr>
<tr>
<td>7.3 - 12.0</td>
<td>60</td>
<td>44</td>
<td>20.4</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Table 5.6 Soil properties. Average value of 0.62 was used for $e$ and 20.2 for bulk density.

Fig. 5.11 Initial coefficient of subgrade reaction.
5.3.4 Loading System and Instrumentation

Considerations of transportation and cost led to a tension loading system, operating between adjacent piers, being adopted similar to the one used for Newmans bridge load tests. The initial tension load required to remove sufficient sag in the cable spans to enable the jacks to reach maximum load without stopping and packing was about 3.0 kN for each pair of cables.

Loads applied by each jack were monitored by Strain gauge type load cells. Horizontal deflections at the top of the pier were monitored with dumpy levels. The rotation at the top of Pier C was measured with a theodolite. The rotation of Pier B was measured using a dumpy level. Deflections and rotation at the tops of the cylinders at Piers B and C were measured using mechanical dial gauges. The rotation at the top of the cylinder at Pier C was also monitored with a force - balance type accelerometer.

5.3.5 Loading of Piers

Five complete cycles of loading were applied to Pier C by alternatively tensioning the cables to Piers B and E. In the first two cycles, the loads in each direction were limited to about 60% and 90% of the maximum peak test load. The maximum peak load of 325 kN was applied in each direction of the three remaining cycles. This peak value represented 75% of the theoretical load to produce first yield in the column reinforcement and is about 13% of the total dead load carried by the columns in the completed structure.

Pier E was loaded in the reverse direction by applying three cycles of loading with the jacks located at Abutment F.

The peak load applied in the first two cycles was 330 kN. In the final cycle the peak load was increased to 390 kN. To observe creep effects, this final peak load was maintained for a period of 10 minutes.
5.3.6 Results

Figure show A37 to A42 show predicted, recorded and compared response of pier top displacement and rotation for Pier C. Similarly cylinder top responses are depicted in Figures A43 to A48.

Although the foundation soil is gravel the analysis was carried out modelling the soil as a sand. \( P_{ult} = k \rho'gz \) gave a good agreement i.e. \( \beta = 1 \).

\( n = 1.0 \) was used to model the non linear behaviour of gravel.

Figures A49 to A54 show the top displacement and rotational responses for Pier B. Figures A55 to A57 show the cylinder tip displacement response for Pier B. Since stiffening the soil springs in the unloading cycles only results in underestimation of the permanent set, the author achieved the necessary permanent set by extra degradation keeping the pult the same even for unloading cycles.

It is of interest to note that increasing Pult in the unloading cycles especially for two way loading cycles especially for two way loading changes the shape of the curve in a peculiar way. COEFF = 0 gives a good response especially for the two way loading case.

A clear break in the linearity of the second loading cycle curves for both piers B and C can be seen at a load of about 260 kN. This change in stiffness occurs at the load producing cracking of the concrete in the columns. The author did not model this aspect of the pile response but this effect has been accomodated by reducing \( P_{ult} \) i.e. reducing \( \beta \). No attempt was made to model soil creep effects.

Figure A44 shows that when load is increased after the 2nd cycle, the stiffness increases. This may be due to a cyclic loading effect. When the pile is loaded cyclically the liquefaction occurring may have given rise to higher gradient of the load deflection curve i.e. higher stiffness.
5.4 CENTRAL LABORATORY PILE TESTS

In October 1981, a lateral load full scale pile test programme was conducted in a site located immediately to the South of the MWD Central Laboratories, Gracefield, Lower Hutt. These tests are presented in detail by Jennings et al (1985). Two piles were used for analysis, the East and West piles. In addition to the slow cyclic loading, dynamic loading also was imposed on the test programme. This author analysed only the slow cyclic load tests.

5.4.1 Pile Specifications

Two 450 mm diameter steel shell piles 8.25 m long were embedded 6.75 m below ground level. The arrangement is shown in Fig. 5.12. Installation was achieved by removing the soil inside the casings and sinking the pile under their own weight. Structural properties of the pile are summarised in Table 5.7.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength of West pile shell ( f_y )</td>
<td>350 MPa</td>
</tr>
<tr>
<td>Yield strength of East pile shell</td>
<td>305 MPa</td>
</tr>
<tr>
<td>Young's modulus, steel ( E_s )</td>
<td>200 MPa</td>
</tr>
<tr>
<td>Young's modulus, concrete ( E_c )</td>
<td>7.7</td>
</tr>
<tr>
<td>Cracking Moment:</td>
<td></td>
</tr>
<tr>
<td>((51 \times 51 \text{ L})) ( M_{\text{crack}} )</td>
<td>117 kNm</td>
</tr>
<tr>
<td>((64 \times 64 \text{ L})) ( M_{\text{crack}} )</td>
<td>131 kNm</td>
</tr>
<tr>
<td>Uncracked section moment in inertia:</td>
<td></td>
</tr>
<tr>
<td>((51 \times 51 \text{ L})) ( I )</td>
<td>0.00473 m$^4$</td>
</tr>
<tr>
<td>((64 \times 64 \text{ L})) ( I )</td>
<td>0.00529 m$^4$</td>
</tr>
<tr>
<td>Cracked section moment of inertia:</td>
<td></td>
</tr>
<tr>
<td>((51 \times 51 \text{ L})) ( I_{\text{cr}} )</td>
<td>0.00346 m$^4$</td>
</tr>
<tr>
<td>((64 \times 64 \text{ L})) ( I_{\text{cr}} )</td>
<td>0.00410 m$^4$</td>
</tr>
</tbody>
</table>

Table 5.7 Pile Properties
Fig. 5.12 General Arrangement and Location of Strain Gauges & Earth Pressure Cells.
Fig. 5.13 Field Investigations Data
5.4.2 Soil Description

The soils are recent, dark grey, silty sands which are loose to medium dense and the water table is about 1.5 m below the natural ground line. Table 5.8 summarises the soil properties obtained from the site investigation and laboratory testing.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.06 \text{ mm} &lt; D_{20} &lt; 0.2 \text{ mm}$</td>
<td></td>
</tr>
<tr>
<td>$U_c = \frac{D_{60}}{D_{10}}$</td>
<td>2.5 to 6</td>
</tr>
<tr>
<td>$N = 4 - 20$ (Uncorrected)</td>
<td></td>
</tr>
<tr>
<td>$c' = 0$, $\phi' = 35^\circ$ (Triaxial)</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.8 Soil Properties

Field investigation data is depicted in Fig. 5.13.

5.4.3 Initial Coefficient of Subgrade Reaction

The initial coefficient of subgrade reaction, $k_0$, is calculated using the equation (3.11)

For angular grained material

$$ G = \frac{3230 (2.97 - e)^2}{1 + e} (\sigma')^{0.5} $$

$\gamma$ is taken as 0.3

Layered soil properties are summarised in Table 5.9.

Figure 5.14 illustrates the East and West piles.

![Diagram of pile configuration](#)

Fig. 5.14 West and East pile.
<table>
<thead>
<tr>
<th>Layer</th>
<th>$N_s$</th>
<th>$\phi'$</th>
<th>$\gamma$</th>
<th>$e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 2</td>
<td>3</td>
<td>31</td>
<td>19</td>
<td>0.50</td>
</tr>
<tr>
<td>2 - 3</td>
<td>10</td>
<td>35</td>
<td>19</td>
<td>0.65</td>
</tr>
<tr>
<td>3 - 6.75</td>
<td>25</td>
<td>35</td>
<td>19</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Table 5.9 Layered soil properties

The author found two possible values of initial coefficient of subgrade reaction to predict the cyclic response. These values are given by the following equations -

$$k_o = 30028 \left(\sigma'\right)^{0.5}$$

modified $k_o = 12011 \left(\sigma'\right)^{0.5}$

A graphical representation is given in Figure 5.15.

---

Fig. 5.15 Initial coefficient of subgrade reaction.
5.4.4 Loading System and Instrumentation

Lateral loads were applied through a strut between the pile heads at a level of 1.35 m above ground level using a push/pull jacking system.

Dynamic loads were generated using a variable frequency shaking machine with contrarotating weights giving sinusoidal force output.

Loads were measured with a load cell. In addition to strain gauges, earth pressure cells and peizometers were attached to the East pile shell. Dial gauges and transducers were used to measure surface displacements. Inclinometer tubing was placed in each pile to monitor the deflected shape of the piles.

5.4.5 Loading

Having applied dynamic load on West pile, slow cyclic load was applied on both East and West piles.

5.4.6 RESULTS

5.4.6.1 Recorded Response

Direct reduction of the data to provide strains deflections and soil pressures was achieved by using appropriate calibration factors for the transducers. The pile shell strain data was used to evaluate a number of profiles for the piles below ground level. The integration and differentiation was carried out using finite difference techniques by Jennings.

Pile top displacement and rotation was assumed to be zero. This might be a reason for the discrepancies involved in prediction. Moment and shear profiles needed to be smoothed to produce a more realistic profile.
5.4.6.2 Predicted Response

Figures A58 and A59 show the predicted ground line deflection profile with the $k$ calculated by the usual method and with the modified $k_0$. Figure A60 shows the recorded response. Figures A61 and A62 show the comparison of predicted and recorded ground line displacement for the East pile.

It is evident from Figures A61 and A62 that modified soil stiffness best represents the in situ soil behaviour except for last two cycles. Figure A61 shows that the last two cycles are well modified using the $k_0$ calculated by the usual method.

Similarly rotational profiles are depicted in Figures A63 to A65.

Figures A66 to A71 show the similar responses for West pile.

Figures A72 to A99 show the moment, rotation, deflection responses along the East pile for a particular lateral load.

The values of $n=1$ and $\beta=3.7$ gave a reasonable resemblance with the recorded response. However, the author has no explanation why the predicted rotational responses are underestimated.

5.5 AUSTIN PILE TEST

The site is located at a place 5 miles away from Northeast of Austin, Texas, adjacent to US Highway 290. The surface soils consist of stiff preconsolidated clays of marine origin.

On December 15, 1966, a pit 45 ft wide by 50 ft long was excavated at the test site. A plan of the test area is shown in Fig. 5.16 with the locations of all the soil borings. Boring 5 and 6 are shown in Fig. 5.17 and 5.18 respectively. After the excavation was completed the pit was flooded with water to saturate the near surface clays and simulate conditions that would exist in clays on the ocean floor. This initial inundation was done some five months before lateral loading of test piles.
Fig. 5.16 Drawing of test area showing locations of borings and piles
Fig. 5.17 Log of boring 5

Fig. 5.18 Log of boring 6
5.5.1 Specifications

Two nominally 24 inch diameter piles, instrumented for measuring bending moments, were driven into stiff clay and subjected to cyclic lateral loading. A nominally 6 inch diameter pile was also instrumented for measuring bending moments. It was driven at the same site, loaded, pulled, redriven and reloaded. Short term static and cyclic loading was employed on both 24 inch diameter and 6 inch diameter piles. The water table was maintained a few inches above the ground surface during the testing program.

For the tests in stiff clays, it was necessary to increase the existing 3/8 inch wall thickness in the top of 23 foot of each 24 inch diameter piles. This was accomplished by wrapping the pile with two 5/8 inch thick wrappers, attached by circumferential weld at the flange of the test pile and at the bottom of the wrap section, and by two longitudinal welds.

The bulk density of the soil is believed to be 18 kNm$^{-3}$. The undrained shear strength profile with depth is depicted in Figure 5.19. The test set up is shown in Fig. 5.20.

---

Fig. 5.19 Composite Soil Profile
5.5.2 Results

Using the values of $\gamma = 0.2$ and 400 for $\frac{G}{S_u}$, the author was able to get a good representation of in situ stiff clay.

Figures A100 to A102 illustrate the surface displacement responses. Initial coefficient used for the analysis is depicted in Figure 5.21.

![Diagram](image)

**Fig. 5.20** Test setup 6 in. diameter test pile

![Graph](image)

**Fig. 5.21** Initial coefficient of subgrade reaction
The author couldn’t make any comment on the permanent set created in cyclic loading because no such recorded data is available for analysis. Degradation parameter DEGPA used for the cyclic analysis is 0.6.

5.6 ARKANSAS PILE TESTS

The lateral load tests on piles performed at sites on the Arkansas River downstream from Pine Bluff, Ark., were carried out by the U.S. Army Engineer Districts of Little Rock, Ark. and Tulsa, Okla., Corps of Engineers. The purposes of this testing programme include the lateral loading - deformation behaviour for individual vertical and batter piles and the effect of repetitive loading.

The pile test analysed here in were made at a site of Lock and Dam 4. A complete description of the soil conditions at Lock and Dam 4 is given by Mansur et al (1970). Figure 5.22 shows the locations of the borings and test piles. The details of the borings are shown in Figure 5.23. The standard penetration resistance SPT value of the upper 60 ft of the deep sand stratum generally increases with depth but averaging 32 blows per foot prior to site excavation. After the 7 m excavation necessary for penetration of the site was completed, the average penetration resistance as determined by the 200 series borings, decrease to about 27 blows per foot. Laboratory test on the sand stratum gives a submerged unit weight of sand of 1000 kgm$^{-3}$ and internal angle of friction of 31° to 35°.
Fig. 5.22 Plan of borings, piezometers and test piles
(after Mansur et al, 1970)
Site preparations included excavation of an area 54 m by 50 m down to elevation 178 (7 m depth), to stimulate the subgrade and overburden stress conditions which would prevail during construction and operation of the locks and dams. The ground water table generally varied from 0.5 m to 0.9 m below the bottom of the excavation during pile driving and testing. The author only analysed pile No. 6. Lateral loads were applied at the ground line for pile No. 6 in 3 equal increments. Properties of this pile is given in Table 5.10.


<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Type</th>
<th>Size (m)</th>
<th>Length (m)</th>
<th>Modulus (GPa)</th>
<th>Mom. Inertia (m^4)</th>
<th>Installation Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Steel H</td>
<td>0.36</td>
<td>12.2</td>
<td>200</td>
<td>0.00031</td>
<td>Driven</td>
</tr>
</tbody>
</table>

Table 5.10 - Properties of pile No. 6

Properties of the soil profile is given in Table 5.11

<table>
<thead>
<tr>
<th>Elevation (m)</th>
<th>Density (kg m(^{-3}))</th>
<th>(N_m)</th>
<th>(C_N)</th>
<th>(E_{m})</th>
<th>((N_i)_{60})</th>
<th>(\phi')</th>
<th>(\gamma)</th>
<th>(k_o) (MN m(^{-3}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 0.4</td>
<td>2200</td>
<td>15</td>
<td>1.6</td>
<td>1.0</td>
<td>24</td>
<td>34</td>
<td>0.3</td>
<td>0 - 65</td>
</tr>
<tr>
<td>0.4 - 0.8</td>
<td>2200</td>
<td>15</td>
<td>1.6</td>
<td>1.0</td>
<td>24</td>
<td>34</td>
<td>0.3</td>
<td>65 - 91</td>
</tr>
<tr>
<td>0.8 - 1.5</td>
<td>2200</td>
<td>15</td>
<td>1.6</td>
<td>1.0</td>
<td>24</td>
<td>35</td>
<td>0.3</td>
<td>91 - 125</td>
</tr>
<tr>
<td>1.5 - 2.5</td>
<td>2200</td>
<td>15</td>
<td>1.6</td>
<td>1.0</td>
<td>24</td>
<td>35</td>
<td>0.3</td>
<td>125 - 161</td>
</tr>
<tr>
<td>2.5 - 4.0</td>
<td>2200</td>
<td>15</td>
<td>1.6</td>
<td>1.0</td>
<td>24</td>
<td>35</td>
<td>0.3</td>
<td>161 - 204</td>
</tr>
<tr>
<td>4.0 - 12</td>
<td>2200</td>
<td>35</td>
<td>1.3</td>
<td>1.0</td>
<td>24</td>
<td>35</td>
<td>0.3</td>
<td>253 - 440</td>
</tr>
</tbody>
</table>

Table 5.11 - Properties of the soil profile (The Arkansas Pile Test: Pile No. 6)

The initial coefficient of subgrade reaction, \(k_o\), was calculated using the equation (3.11). The predicted and recorded response of the pile No. 6 is shown in Figure A103. Like Austin pile tests lack of data about the permanent set created while repetitive loading against number of cycles leads to unreliable values of COEFF and DEGPA.
CHAPTER 6

DISCUSSION ON THE PREDICTED RESPONSE

Modelling of non-linear behaviour of piles subjected to static and cyclic lateral loading has been of great interest to researchers in the recent years. Various approaches have been adapted by different researchers of which the Winkler model is the simplest. The different type of method are characterised by the differences in the treatment of the soil medium in the analysis.

Elastic Continuum methods as proposed by Mindlin (1936) have been used by various authors for soil-pile interactions problems. However, this has limited applications because of the assumptions made for this analysis. The soil is assumed to be elastic in this analysis. Poulos (1973) suggested modifying the method by changing Young's modulus of the soil, $E_s$. Non-linear soil response has also been included. Poulos (1981) made modification to allow degradation of the soil properties. However, non-linearity of the soil was modelled very well by p-y curves. Matlock (1970) first introduced the concept of p-y curves and methods for their development. This technique accommodates degradation of soil too in case of cyclic loadings. Due to the tedious procedures involved in the p-y curves technique, it has always been important to find a simple and reasonably accurate soil model. The non-linear soil model analysed in this research is one of those solutions.

The non-linear soil model originally proposed by Pender (1984) was used by Carter (1984) and Ling (1988) to analyse static loading behaviour of piles and proved to be very effective. The author also was successful in predicting the cyclic loading behaviour of the piles. However, more comparison of full scale pile tests must be carried out to find the probable range of values for the parameters used in this research.

Analysis for gravels has been considered similar to sand. The shear modulus of the sand is calculated from equation 3.12 which was also used by Carter. The initial coefficient of subgrade reaction $k_0$ was calculated from the equation
\[ k_0 = 1.0 \frac{E_s}{\left(1 - \nu_s^2\right)} \text{ where } E_s = 2G(1 + \nu) \]

However, the author found it simple to use the equations 3.16 and 3.17 in determining initial coefficient of subgrade reaction for cohesionless soils. Arkansas river pile test was analysed by the author using these equations. These equations are very useful where there are SPT results available.

The shear modulus for clays was calculated from undrained shear strength. Carter found that the values of G/S for soft clays lie between 100 and 200. The author found that the value of $G/S_0$ gave a good representation for the Austin stiff clay but found it difficult to estimate the value of $G/S_0$ for papa found in Newmans Bridge Tests. A value of 1500 was assumed based on its higher strength. The same problem was encountered in the papa found in Waitai River Bridge Test.

A linear increase in the value of Pult with depth provided a good representation of the ultimate resistance of the soil i.e. $Pult = \beta K \rho'gz$ for cohesionless soils, and $\beta = 5$ gave a good representation. This was found true especially for sand. However, the gravels found in Waitai River site were not satisfied by these values. They rather required smaller values. The soil model analysed in this research gave a good representation for static load tests with the value of $\beta = 5$. However, in Waitai River Bridge Test, the author used a value $\beta = 1$ for a good representation. This could have been avoided if the yield of the pile was considered. The author reduced the value of $\beta$ for the non linearity occurring due to the yield of the pile. The Central Lab test results also showed that the value of 3.7 for $\beta$ gave a good representation. The values of $\beta$ greater than 5 tend to give a linear load displacement response. The smaller values of $\beta$ tend to give rather a non-linear response which is very common in cyclic loading. The author suggests a value between 3 and 5 for $\beta$ as far as "CLASS A" prediction is concerned. However, yield of the pile also must be taken into consideration before this is adopted.
The hyperbolic non-linear soil model with an \( n \) value of 1.0 suggested by Pender gave the closest match for sand and gravels. However, this was unsuitable to model clayey soils. For clays, \( n = 0.2 \), was found to give the best match.

The values of the parameter COEFF, used in this research are another important consideration for cyclic loading. The value expected by the author was 1.0. However, closest match was obtained with a value of 0 for two way loading. For one way loading, this value ranges from 0.1 to 20. The reverse cycles are controlled by the value of COEFF. The greater the value of COEFF the greater the value of Pult for reverse cycles. This leaves a permanent set when the pile is totally unloaded. A value of 1.0 underestimates the permanent set. The author suggests a value of 10 of COEFF for one way loading to give the closest match.

In the case of the Austin Pile Test where there is a large number of cycles involved, a large value of COEFF could overestimate the ultimate permanent set. A value of COEFF = 0.1 gave the closest match.

Newmans Bridge and Maitai River Bridge tests only employ a few cycles and the permanent set is very much higher in these cases. The author increased the value of COEFF to 20 to obtain the closest match.

A drawback in this technique is that yield of the pile is not considered properly. A non-linear pile model should be introduced to accomodate the permanent set created by the pile. That is the reason why there is difference in values of COEFF. To obtain more realistic values of COEFF, yield of the pile should be taken care of especially for the reverse cycles.

The Austin pile test reported by Reese Cox and Koop (1975), doesn’t have any information about the permanent set over number of cycles. This is a drawback although this is the test involved with a large number of cycles.

The degradation parameter DEGPA, used in this research, determines how much degradation occurs in the soil in each and every cycle. However, Newman’s Bridge Test and Maitai River Bridge test results show some sudden changes in stiffness at a load of about 160 kN. This change in stiffness
corresponds to the loads producing cracking of the concrete in the columns. The author model this feature by using a higher value of DEGPA and lower value of Pult i.e. lower value of $\beta$. But the author suggests that the consideration of the yield of the pile would be the best solution. However, cracked section properties of the concrete were used to avoid under estimation of ultimate response. The degradation of the soil in the loading cycles also left some additional permanent set in the unloading cycles since the unloading cycles still use the initial coefficient of subgrade reaction which is higher than that of loading cycles.

The value of DEGPA varies from 5 to 300. In case of the Austin pile test where there are a large number of cycles involved, a large value of degradation parameter like 300 gives an overestimation of ultimate displacement. A value of 0.6 gives a closest match. The silty sand found in Central Laboratory pile test has a value of 5.0 for DEGPA giving the closest match. When the modified initial coefficient of subgrade reach is used, the value of DEGPA needs to be only 1.0 to give the closest match. Maitai River Bridge Test soil layers consisting of papa and gravel have values of 170 and 220 for piers C and B respectively. Newmans Bridge site soils around pier B have values of DEGPA 270 and 350 for pier top and cylinder top loading cases respectively. However soils surrounding pier C require a value of DEGPA of 1000. The difference is due to the higher number of soil springs in pier B compared to pier C.

The author didn’t analyse the effect the pile width has on the amount of degradation of the soil. The creep effect of the soil also was not taken into consideration although this effect was evident in some of the recorded responses in Maitai River Bridge & Newmans Bridge Tests.

The non-linear soil model has thus been proved to be very effective for cyclic loading as well. By changing values of $k$, $Pult$, $n$, COEFF and DEGPA, the author was able to model cyclic lateral loading behaviour of the piles. However, many more comparisons must be made to determine the probable values of COEFF and DEGPA especially.
Because of the lack of understanding of the mechanism involved within soil layers, it is very important to back analyse the full scale pile tests. However, there are only a few tests available with detailed information. To analyse the effect of the number of cycles, it is desirable to have detailed information about the response over higher number of cycles. Although the Austin pile test involves a larger number of cycles, each and every cycle is not documented, only final response is available. It would be more rewarding to analyse if detailed full scale pile test results are found. Maitai River pile test and Newman's Bridge Test have got detailed information over every cycle. However, the number of cycles involved are only a few. The author suggests carrying out some detailed full scale pile tests with repeated constant lateral load until failure. This will enable analysis of the effect of number of cycles.

The reverse cycles must be modelled considering the gap formed behind the pile and yield of the pile. This will enable determination of realistic values of COEFF and DEGPA.

The effect of installation of piles wasn't considered by the author. Research is also necessary to predict the cyclic response based on effective stress parameters. Furthermore research should be done on dynamically loaded full scale pile tests.
CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 SUMMARY

It is the researchers' desire to provide a simple soil model to predict lateral response of a pile. The non-linear behaviour of soil is very difficult to predict. However, the non-linear soil model analysed in this research has proved to be a valid tool to predict static and cyclic lateral response of a pile. Although use of the p - y curve method introduced by Matlock is very accurate, a simpler model is preferred because p - y criteria involve tedious procedures. There are many other simple soil models available. However, their accuracy is very limited.

The non-linear soil model originally proposed by Pender was used by Carter (1984) for static loading and was found to be an excellent tool in predicting the lateral response of piles. Carter wrote a finite element program allowing the soil to behave non-linearly. This was further verified by Ling (1988). Ling also concluded that the soil model gave excellent agreement with recorded response. This was possible by varying the initial small strain stiffness of the soil, $k_0$, the ultimate soil resistance $P_{ult}$, and the index $n$.

Although the author adapted the method suggested by Carter to determine initial coefficient of subgrade reaction for cohesionless soils found in some of the sites, the author found it easy to use the method suggested by Ling (1988).

The author modified the program written by Carter. The original program could not accommodate a varying moment of inertia of pile with depth. This modification was extremely important in a test like Newmans Bridge Test.
Although the Carter’s program was able to handle cyclic loads, the author verified that the permanent set after complete unloading was very much underestimated. Unloading cycles correspond to an increased value of Pult i.e. soil springs are stiffened for the unloading cycles. The author introduced a stiffening coefficient COEFF which increases the value of Pult linearly. This is the parameter which controls the unloading cycles. If the gap formed behind the pile and yield of the pile are considered more realistic values of COEFF can be obtained.

The author made another major modification which helped to model the lateral response of the pile in each and every cycle. A reduction in initial coefficient of subgrade reaction was allowed considering degradation of the soil. The degradation in every load cycle depends on the number of cycles, deflection undergone by the pile in the previous cycle and degradation parameter DEGPA. Since yield of the pile was not considered by the author, any response due to the yield of the pile also was taken care of by the parameter DEGPA.

Pender suggested the non-linear soil model from the results derived for sand which showed a hyperbolical response with \( n = 1 \). Thus Carter also never attempted to analyse the full scale pile test results for cohesionless soils for values of \( n \) other than 1. However, the author modified the program for cohesionless soils to handle any value of \( n \). But back analysis showed that the best match takes place when \( n = 1 \).

7.2 SUGGESTIONS FOR FUTURE WORK

1. The aim of this thesis was to see if the non-linear soil model, which gave an excellent match with recorded response for static loading, is applicable for cyclic loading. Although the non-linear soil model used in this research proved to be very satisfactory, much more comparisons will need to be made before it can be used for prediction of lateral response. Thus, the most probable values of \( k_0 \), Pult, \( n \), COEFF and DEGPA should be found.
2. So far the non-linear soil model has not been used for CLASS "A" prediction. The prediction should be attempted before the pile test is carried out.

3. The methodology adapted for unloading cycles should be changed. The gap formed behind the pile should be considered. This might be a better solution rather than the introduction of the stiffening coefficient COEFF.

4. Yield of the pile should be considered especially in the loading cycles. This would enable avoidance of extra allowance in DEGPA.

5. Changes in moment of inertia due to yielding of the pile should be allowed.

6. This non-linear soil model should be tested for pile groups.

7. Full scale pile test results for the dynamically loaded piles must be back analysed.
REFERENCES


APPENDIX A

GRAPHS OF PREDICTED
AND RECORDED RESPONSE

The following will apply for the figures A72 to A99:-

- Recorded

- - Predicted

(with modified $k_0$)
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER C)

PIER TOP LOAD CASE

Fig. A(1) - Predicted Response
Fig. A(2) - Deflections at Pier C top (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED SURFACE DISPLACEMENT

PILE TOP LOAD TEST AT HERMANS BRIDGE (PIER C)

Fig. A(3) - Analysis of the End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER B)

PIER TOP LOAD CASE

Fig. A(4) - Predicted Response
Fig. A(5) - Deflections at Pier B Top (Recorded Response)
Fig. A(6) - Analysis of the End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER C)

PIER TOP LOAD CASE

Fig. A(7) - Predicted Response
Fig. A(8) - Pier C Top Rotation (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
PILE TOP LOAD TEST AT NEWMANS BRIDGE (PIER C)

Fig. A(9) – Analysis of the End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER B)

PIER TOP LOAD CASE

Fig. A(10) - Predicted Response
Fig. A(11) - Pier B Top Rotation
(Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
PIER TOP LOAD TEST AT NEWMANS BRIDGE (PIER B)

Fig. A(12) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER C)

PIER TOP LOAD CASE

Fig. A(13) - Predicted Response
Fig. A(14) - Deflections at Pier C Base
(Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
PIER TOP LOAD TEST AT NEWMANS BRIDGE (PIER C)

Fig. A(15) - Analysis of End Points of Each Loading Cycle
Fig. A(17) - Deflections at Pier C Base (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
PIER TOP LOAD TEST AT NEWMANS BRIDGE (PIER B)

Fig. A(18) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER C)

PIER TOP LOAD CASE

Fig. A(19) - Predicted Response
Fig. A(20) - Pier C Column Base Rotation (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
PIER TOP LOAD TEST AT NEWMANS BRIDGE (PIER 0)

Fig. A(21) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER B)

PIER TOP LOAD CASE

Fig. A(22) - Predicted Response
Fig. A(23) - Pier B Column Base Rotation (Recorded Response)
COMPARISON OF PREDICTED WITHRecorded RESPONSE
PIER TOP LOAD TEST AT NemHANS Bridge (PIER B)

Fig. A(24) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER C)

CYLINDER TOP LOAD CASE

Fig. A(25) - Predicted Response
Fig. A(26) - Deflection of Pier C Cylinder Top (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE

CYLINDER TOP LOAD TEST AT NEWMANS BRIDGE (PIER C)

Fig. A(27) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER B)

CYLINDER TOP LOAD CASE

Fig. A(28) - Predicted Response
Fig. A(29) - Deflection of Pier B Cylinder Top (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE

CYLINDER TOP LOAD TEST AT NEWMANS BRIDGE (PIER B)

Fig. A(30) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER C)

Fig. A(31) - Predicted Response
Fig. A(32) - Pier C Cylinder Top Rotation (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
CYLINDER TOP LOAD TEST AT NEWMANS BRIDGE (PIER C)

Fig. A(33) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT NEWMANS BRIDGE (PIER B)

CYLINDER TOP LOAD CASE

Fig. A(34) - Predicted Response
Fig. A(35) - Pier B Cylinder Top Rotation (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
Cylinder Top Load Test at Newmans Bridge (Pier B)

Fig. A(36) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT MAITAI RIVER

PIER TOP LOAD TEST

Fig. A(37) - Predicted Response
Cycles 1 to 3

Cycles 3 to 5

Fig. A(38) - Deflections at Top of Pier C (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED SURFACE DISPLACEMENT

PILE TOP LOAD TEST AT HAITAI RIVER (PIER C)

Fig. A(39) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT MAITAI RIVER
PIER TOP LOAD TEST

Fig. A(40) - Predicted Response
Fig. A(41) - Rotations at Top of Pier C (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED PIER TOP ROTATION

PILE TOP LOAD TEST AT MAITAI RIVER (PIER C)

Fig. A(42) - Analysis of End Points of Each Loading Cycle
Fig. A(43) - Predicted Response
Cycles 1 to 3

Cycles 3 to 5

Fig. A(44) - Deflections of Cylinder at Pier C (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
PIER TOP LOAD TEST AT MAITAI RIVER (PIER C)

Fig. A(45) - Analysis of End Points of Each Loading Cycle
Fig. A(46) - Predicted Response
Fig. A(47) - Rotations of Cylinder at Pier C
(Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED CYLINDER TOP ROTATION

PILE TOP LOAD TEST AT MATTAI RIVER (PIER C)

Fig. A(48) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT MAITAI RIVER (PIER B)

PIER TOP LOAD TEST

Fig. A(49) - Predicted Response
Cycles 1 to 5

Fig. A(50) - Deflections at Top of Pier B
(Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE

PIER TOP LOAD TEST AT MAITAI RIVER (PIER B)

Fig. A(51) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT MAITAI RIVER (PIER B)

PIER TOP LOAD TEST

Fig. A(52) - Predicted Response
Cycles 2 and 3

Cycles 4 and 5

Fig. A(53) - Rotations at Top of Pier B (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED PIER TOP ROTATION

PILE TOP LOAD TEST AT MAITAI RIVER (PIER B)

Fig. A(54) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT MAITAI RIVER (PIER B)

Fig. A(55) - Predicted Response
Fig. A(56) - Deflections of Cylinder at Pier B (Recorded Response)
COMPARISON OF PREDICTED WITH RECORDED RESPONSE
PIER TOP LOAD TEST AT WAITAI RIVER (PIER B)

Fig. A(57) - Analysis of End Points of Each Loading Cycle
LATERAL LOAD PILE TEST AT CENTRAL LAB (EAST PILE)

Fig. A(58) - Predicted Response
LATERAL LOAD PILE TEST AT CENTRAL LAB (EAST PILE)

PILE TOP LOAD TEST

Fig. A(59) - Predicted Response with Modified Soil Stiffness
Fig. A(60) - East Pile Measured Ground Line Deflection
COMPARISON OF PREDICTED WITH RECORDED SURFACE DISPLACEMENT

PILE TOP LOAD TEST AT CENTRAL LAB (EAST PILE)

Fig. A(61) - Analysis of End Points of Each Loading Cycle
COMPARISON OF PREDICTED WITH RECORDED RESPONSE (MODIFIED Ko)
PILE TOP LOAD TEST AT CENTRAL LAB (EAST PILE)

Fig. A(62) - Analysis of End Points of Each Loading Cycle
Fig. A(64) - Predicted Response with Modified Soil Stiffness
Fig. A(65) - East Pile Measured Ground Line Rotation
Fig. A(66) - Predicted Response with Modified Soil Stiffness
LATERAL LOAD PILE TEST AT CENTRAL LAB (WEST PILE)

PILE TOP LOAD TEST

Fig. A(67) - Predicted Response
Fig. A(68) - Deflection Profile

WEST PILE GROUND LINE DEFLECTION (MEASURED)
LATERAL LOAD PILE TEST AT CENTRAL LAB (WEST PILE)

PILE TOP LOAD TEST

Fig. A(69) - Predicted Response with Modified Soil Stiffness
LATERAL LOAD PILE TEST AT CENTRAL LAB (WEST PILE)

PILE TOP LOAD TEST

Fig. A(70) - Predicted Response
Fig. A(71) - West Pile Measure Ground Line
Fig. A(72) MOMENT PROFILE

Fig. A(73) MOMENT PROFILE (CALCULATED FROM STRAIN GAUGE DATA)
Fig. A(74)  MOMENT PROFILE

Fig. A(75)  MOMENT PROFILE
Fig. A(76) MOMENT PROFILE

Fig. A(77) MOMENT PROFILE
Fig. A(78) MOMENT PROFILE

Fig. A(79) MOMENT PROFILE
Fig. A(80) MOMENT PROFILE

Fig. A(81) MOMENT PROFILE
Fig. A(82) DEFLECTION PROFILE

Fig. A(83) ROTATION PROFILE
Fig. A(84) DEFLECTION PROFILE

Fig. A(85) ROTATION PROFILE
Fig. A(86) DEFLECTION PROFILE

Fig. A(87) ROTATION PROFILE
Fig. A(88) DEFLECTION PROFILE

Fig. A(89) ROTATION PROFILE
Fig. A(90) DEFLECTION PROFILE

Fig. A(91) ROTATION PROFILE
Fig. A(92) - DEFLECTION PROFILE

Fig. A(93) - ROTATION PROFILE
Fig. A(94)  DEFLECTION PROFILE

Fig. A(95)  ROTATION PROFILE
Fig. A(96) DEFLECTION PROFILE

Fig. A(97) ROTATION PROFILE
Fig. A(98) DEFLECTION PROFILE

Fig. A(99) ROTATION PROFILE
LATERAL LOAD PILE TEST AT AUSTIN
PILE TOP LOAD TEST

Fig. A(100) - Predicted Response
LATERAL LOAD PILE TEST AT AUSTIN

PILE TOP LOAD TEST

Fig. A(101) - Predicted Response
CYCLIC LATERAL LOADING BEHAVIOUR OF PILLES AT AUSTIN

- Recorded graph of load vs sur. disp. (cyclic): ④
- Recorded graph of load vs sur. disp. (static): ①

- Predicted graph of load vs sur. disp. (static): ②
- Predicted graph of load vs sur. disp. (cyclic): ③

Fig. A(102) = Comparison of Predicted and Recorded Responses
LATERAL LOAD PILE TEST AT ARKANSAS

Fig. A(103) - Deflection Profile
APPENDIX B
B1 USER GUIDE

B.1.1 Input Information

The EXEC that has been used to run the Program is:

PILEX EXEC

FILEDEF 1 DISK INPUT DATA A1 (PERM BLOCK 80
FILEDEF 3 DISK OUTPUT DATA A1 (PERM BLOCK 80
LOAD CYCLIC (START

The data, and the order in which it is input, is outlined as follows. (A sample input is provided in Section B.4)

READ (1, ) TITLE
  (e.g. Pile Test No. 1)

READ (1, ) UT1, UT2, UT3, UT4, UT5, UT6, UT7
  FORMAT (7 (A4, 6X))

(for metric units:
  m, mm, kN, kN-m, kPa, kN/m³, kN/m)

READ (1, ) LIST, IPRT, ISEG, NPRT

  LIST = 1, if band matrix is to be printed
  IPRT = 1, to print soil springs and stiffnesses
  ISEG = 1, for Program to read member lengths
  NPRT = every n'th iteration to be printed

READ (1,*) NDF, NM, NSPX, NSLAY, NWTBLE

  NDF = number of degrees of freedom (2 * NM) + 2
  NM = number of pile members
  NSPX = number of specified pile deformations
  NSLAY = number of soil layers
  NWTBLE = Layer No. whose upper boundary is level with the water table.

READ (1,*) E, WIDTH, WATWGT

  E = modulus of elasticity for the pile [MPa]
  WIDTH = width of the pile [m]
  WATWGT = weight of water [kN/m³]

The next statement inputs the properties of each soil layer. This in line is repeated for each soil layer and the pile section in it.

READ (1,*) LAYTYP(IS), NMPLAY(IS), ELKWGT(IS), WDTLAY(IS), XII(IS)

  LAYTYP(IS) = type of soil in layer No IS
    = 1 for linear soil
    = 2 for non-linear soil
    = 3 for non-linear sand
NMPLAY(IS) = number of members in layer IS [kN/m³]
BLKWGT(IS) = bulk weight of soil in layer IS [m]
WDTLAY(IS) = width of soil layer IS [m²]
XII(IS) = moment of inertia for the pile section in layer IS [m]

If the member lengths are to be input manually then
READ (1,*) (M(J), J = 1, NM)

M(J) = Length of member J [m]

Otherwise, if the Program is to divide the pile, then
READ (1,*) PILENG

PILENG = total length of the pile [m]

If certain pile deformations are specified, i.e. NSPX is greater than zero, then the specified displacement must be input. Each degree of freedom must be specified on a new line.
READ (1,*) IDOFX(N), XSPEC(N)

IDOFX(N) = the degree of freedom that is specified
XSPEC(N) = the specified deformation

The force increments are now input,
READ (1,*) NAPPLD, DELTBM, DELTHF, DEGPA, COEFF, COEFFI, IS

NAPPLD = number of loading cycles [kNm]
DELTBM = bending moment increment [kNm]
DELTHF = lateral force increment [kNm]
DEGPA = degradation parameter constant
COEFF = ultimate pressure coefficient
COEFFI = 0 [coefficient which changes the initial k for unloading cycles]
IS = number of cycle for which output is printed.

followed by the loading sequence. Each load combination must be input on a new line.
READ (1,*) PLSQCE(1,1), PLSQCE(1,2)

PLSQCE(1,1) = externally applied bending moment [kNm]
PLSQCE(1,2) = externally applied lateral force [kNm]

The soil model parameters are input last. One line of input is required for each soil layer, and the information on each line depends on the soil type. The input statements for the three soil subroutines are outlined below.
For Linear Soils:

READ (1,*) AC, BC, CC

\[
Sk = AC + BC \cdot X^{CC}
\]

\(k_c\), the coeff. of subgrade reaction \([\text{kN/m}^3]\)

AC, BC, CC = constants which describe the change in \(k_c\) with depth

X = depth within the soil layer, \(X = 0\) at the upper boundary of each soil layer \([\text{m}]\)

For Non-Linear Clays:

READ (1,*) AC, BC, CC, EC, FC, EXPON(IC)

\[
\text{ORIGK(IC, J)} = AC + BC \cdot X^{CC}
\]

\(\text{ORIGK(IC, J)}\) = the initial coeff. of subgrade reaction \([\text{kN/m}^3]\)

X = depth within the soil layer \([\text{m}]\)

CCLAY = EC + FC \cdot X = undrained cohesion \([\text{kPa}]\)

EXPON(IC) = the exponent, \(n\), for the author's soil model

For Non-Linear Sands:

READ (1,*) AC, BC, CC, BETC, FACT, ANG, EXPON(IC)

\[
\text{ORIGK(IC, J)} = AC + BC \cdot X^{CC}
\]

ANG = friction angle for the sand \([\text{degrees}]\)

BETA = BETC + FACT \cdot X, the factor for the ultimate soil pressure

EXPON(IC) = the exponent, \(n\), for the author's soil model (a value of 1 is suggested for sand).
B.1.2 Error Messages

To assist the user, several error messages have been included in the Program. If any of these errors are encountered, the Program will halt execution automatically.

"INCORRECT INFORMATION ABOUT THE NUMBER OF MEMBERS IN EACH LAYER HAS BEEN PROVIDED. EXECUTION HAS TERMINATED."

The sum of the members in each layer must equal the total number of members, NM, specified by the user.

"WHEN THE SOIL PROFILE IS NOT UNIFORM, THE SEGMENT LENGTHS MUST BE INPUT MANUALLY. EXECUTION HAS TERMINATED."

If layered soil profile is used, the member lengths must be input manually to ensure that the soil boundaries are coincident with the pile nodes.

"THE NUMBER OF MEMBERS IS GREATER THAN THIS PROGRAM PERMITS. EXECUTION HAS TERMINATED."

The Program can be used to automatically divide the pile into segments of unequal length. However, the smallest element length that is permitted is half of the pile diameter. This error message will occur if the pile length, when divided by the number of members, is less than half the diameter of the pile.
**LIST OF PROGRAM VARIABLES**

- **ALI**: the number of loading increments required for each load cycle
- **ANG**: the friction angle for the sand
- **BLKNGT(I)**: the bulk weight of the soil, in the I’th layer
- **BM**: the first, bending moment increment
- **BNINCR**: the subsequent, bending moment increments
- **BMMAX**: the maximum bending moment that must be applied for each load cycle. This is equal to the difference between the maximum bending moment for the last load cycle and the bending moment for the present load cycle
- **BKTOT**: the bending moment that is currently being applied to the pile
- **COEFF**: the ultimate pressure coefficient
- **COEFF1**: 0, the coefficient which changes the initial $k_o$ for unloading cycles
- **DEGR**: degradation parameter constant
- **DELTHF**: the horizontal force increment
- **DELTM**: the bending moment increment
- **DSPL(I)**: the pile deformations as a result of the applied load at the start of each load cycle, $DSPL(I) = 0$
- **DSPMAX(I)**: the pile deformations at the maximum load of the previous load cycle
- **DSPSUM(I)**: the total deformations of the pile
- **E**: the modulus of elasticity of the pile
- **EA(I, J)**: the member displacement transformation matrix, [a]
- **EFFWGT(I)**: the effective weight of soil in the I’th layer
- **EGK(A,J)**: the structure stiffness matrix, [g] [k] [a]
- **EK(I,J)**: the member stiffness matrix, [k]
- **EKA(I,J)**: [k] [a]
- **EXPON(I)**: the exponent, $n$, (used by the author’s soil model) for the I’th soil layer
- **F(I)**: the internal member forces developed at the nodes (calculated for each force increment)
- **FSUM(I,J)**: the total member forces at the nodes
- **H(I)**: the length of the I’th pile element
- **HF**: the first, horizontal force increment
- **HFINC**: the subsequent, horizontal force increments
- **HFTOT**: the horizontal force that is currently being applied to the pile
- **IABC**: after the soil properties have been input, IABC is set to 2. The read statements will then be ignored for the subsequent loops
- **IC**: the number of the soil layer
- **IDOF**: a counter for the degree of freedom
- **IDOFX(I)**: the degrees of freedom which have specified displacements
- **IPRT**: I, if the soil springs and stiffnesses are to be printed
- **IS**: the cycle number for which the output is printed
- **ISEG**: I, if the member lengths are input manually
- **ISOCE**: the number of the loading increments necessary to reach the maximum load
- **ITERC**: the iteration counter
- **LAVTYP(I)**: the type of soil model which is to be used to model the layer
- **LIST**: I, if the band matrix is to be printed
- **MM**: the width of the band matrix, 4
- **NAPPLD**: the number of loading cycles that will be applied to the pile
NBAND = 4, the width of the band matrix
NDF = the number of degrees of freedom of the pile
NDFPI = the number of degrees of freedom + 1
NM = the number of pile members
NMCO = a member number counter
NMDMY = a dummy variable
NMP1 = the number of pile members + 1
NMPL = the number of pile members in the current layer
NMPLOY(I) = the number of pile elements in the I’th soil layer
NNSUM = a variable which checks that the sum of the members in each
layer is equal to NM
NN = the number of degrees of freedom
NPE(I,J) = the degrees of freedom associated with the I’th member
NPRT = a variable which dictates how often the member forces and
deformation are to be printed
NSLAY = the number of soil layers
NT = the number of elements in the band matrix
ORIGK(I,J) = the initial small strain coefficient of subgrade reaction at
the nodes in the I’th soil layer for the first cycle of
loading
ORIGN(I,J) = the initial coefficient of subgrade reaction at the nodes in
the I’th soil layer for cyclic loads
P(I) = the incremental load matrix which also doubles as the
deformation matrix
PILENG = the length of the pile
PLNM = a constant
PLSQCE(I,1) = the externally applied bending moment for the I’th load cycle
PLSQCE(I,2) = the externally applied horizontal force for the I’th load
cycle
PRSOLD(I,J) = the maximum soil pressure around the pile for the last load
cycle
PRTC = the print counter. If PRTC=0, then the member forces and
deformation are printed
PULT(I,J) = the current ultimate soil pressure, being used by the soil
model
PULTON(I,J) = the initial ultimate pressure of the soil, at the nodes in the
I’th soil layer
RLI = the number of loading increments required for each load cycle
RNM = the number of pile members
RNMP1 = the number of pile members + 1
SK(I,1) = the coeff. of subgrade reaction at the top of the I’th member
SK(I,2) = the coeff. of subgrade reaction at the bottom of the I’th
member
SSK(I,1) = the stiffness of the soil spring at the top of the I’th member
SSK(I,2) = the stiffness of the soil spring at the bottom of the I’th
member
STIFF(I) = the band matrix
UT1, UT2, UT3, UT4, UT5, UT6, UT7 are the units, either metric or imperial
WATWGT = the weight of water
WDTLAY(I) = the thickness of the I’th soil layer
WIDTH = the width of the pile
XI(I) = the moment of inertia of the I’th pile element
XII(IS) = the moment of inertial of the pile in the soil layer IS
XSPEC(I) = the values of the specified displacements
Z(I) = the depth of node I beneath the ground surface.
FINITE ELEMENT PROGRAM DESIGNED TO ANALYSE LATERALLY LOADED PILES

THIS PROGRAM IS SET UP TO HANDLE BOTH STATIC AND CYCLIC LOADING

IT IS BASED ON THE WINKLER SOLUTION WITH THE SOIL BEING MODELLED AS A BED OF INDEPENDANT SPRINGS. EACH ELEMENT HAS TWO SPRINGS, ONE AT EITHER END

THE LOADS ARE APPLIED TO THE TOP OF THE PILE, IN THE FORM OF A LATERAL FORCE AND A BENDING MOMENT

THE RATIO OF BENDING MOMENT TO HORIZONTAL FORCE IS KEPT CONSTANT THROUGHOUT THE APPLICATION OF LOAD

THE APPLIED BENDING MOMENT AND HORIZONTAL FORCE IS INCREASED BY A CONSTANT STEP SIZE, UNTIL THE MAXIMUM DESIRED LOAD IS REACHED

FOR ANY GIVEN SET OF APPLIED LOADS, THE PROGRAM CALCULATES THE ROTATIONS, DISPLACEMENTS, BENDING MOMENTS AND SOIL REACTIONS ALONG THE LENGTH OF THE PILE

THE PILES MUST BE OF CONSTANT WIDTH WITH A SINGLE VALUE FOR THE MOMENT OF INERTIA

THE SOIL SURFACE IS LEVEL WITH THE TOP OF THE PILE ( AT NODE 1 )

THE DIFFERENT SOIL MODELS ARE ATTACHED TO THE PROGRAM AS SUBROUTINES

BECAUSE OF THE INCREMENTAL LOADING, THE PROGRAM CAN HANDLE BOTH LINEAR AND NON-LINEAR SOILS

NO ALLOWANCE HAS BEEN MADE FOR THE EFFECT OF VERTICAL LOAD ON THE LATERAL RESPONSE OF THE PILE

VARIABLE IDENTIFICATION

LIST = 1 TO LIST BAND MATRIX
IPRT = 1 TO PRINT SOIL SPRINGS AND STIFFNESS
ISEG = 1 FOR PROGRAM TO READ MEMBER LENGTHS
NPRT = COUNTER TO INDICATE HOW REGULARLY THE DISPLACEMENTS SHOULD BE PRINTED
(1 FOR EVERY ITERATION, 2 FOR EVERY SECOND, ETC)
NDF = NO OF DEGREES OF FREEDOM
NM = NO OF PILE ELEMENTS (MEMBERS)
NSPX = NO OF SPECIFIED PILE MOVEMENTS
NSLAY = NO OF SOIL LAYERS
NWTLBE = SOIL LAYER WHOSE UPPER BOUNDARY IS COINCIDENT WITH THE WATER TABLE
E = MODULUS OF ELASTICITY OF THE PILE, KSF OR KPA
XI(I) = MOMENT OF INERTIA, FT**4 OR M**4
WIDTH = PILE WIDTH, FT OR M
PILENG = PILE LENGTH, FT OR M
WATWGT = WEIGHT OF WATER, KCF OR KNCM

CYC00010
CYC00020
CYC00030
CYC00040
CYC00050
CYC00060
CYC00070
CYC00080
CYC00090
CYC0100
CYC0110
CYC0120
CYC0130
CYC0140
CYC0150
CYC0160
CYC0170
CYC0180
CYC0190
CYC0200
CYC0210
CYC0220
CYC0230
CYC0240
CYC0250
CYC0260
CYC0270
CYC0280
CYC0290
CYC0300
CYC0310
CYC0320
CYC0330
CYC0340
CYC0350
CYC0360
CYC0370
CYC0380
CYC0390
CYC0400
CYC0410
CYC0420
CYC0430
CYC0440
CYC0450
CYC0460
CYC0470
CYC0480
CYC0490
CYC0500
CYC0510
CYC0520
CYC0530
CYC0540
CYC0550
CYC0560
CYC0570
CYC0580
H(I) = LENGTH OF PIECE ELEMENT I, FT OR M
Z(I) = DEPTH OF NODE I, BELOW THE GROUND SURFACE
IDOFX(I) = DEGREE OF FREEDOM WITH KNOWN DISPLACEMENT
XSPEC(I) = VALUES OF KNOWN DISPLACEMENTS,
           FT, M, OR RADIANS
LAYTYP(I) = TYPE OF SOIL IN LAYER I
NMPLAY(I) = NO OF PIECE ELEMENTS IN LAYER I
WDTLAY(I) = THICKNESS OF SOIL LAYER I
BLKWGT(I) = BULK WEIGHT OF SOIL IN LAYER I
EFFWGT(I) = EFFECTIVE WEIGHT OF SOIL IN LAYER I
NAPPLD = NUMBER OF APPLIED LOADING CYCLES
P(1) = LOAD MATRIX
P(2) = LOAD MATRIX

USE CONSISTENT UNITS FT, M; K/SQ FT, KPA; KCF, KN/CU M

MISCELLANEOUS PROGRAMMING INSTRUCTIONS

SOIL LAYERS ARE NUMBERED DOWN FROM THE SURFACE
SOIL SPRING FOR END NODES SAME AS INTERIOR NODES

UT1 = FT OR M
UT2 = IN OR MM
UT3 = KIPS OR KN
UT4 = K-FT OR KN-M
UT5 = KSF OR KPA
UT6 = KCF OR KNCM
UT7 = K-FT OR KN/M

INITIALISATION SECTION

IMPLICIT DOUBLE PRECISION (A-H, O-Z)
DIMENSION BLKWGT(10), DSPL(140), DSPMAX(140), DSPSUM(140), EA(4, 4),
1      EGKA(4, 4), EFFWGT(10), EK(4, 4), EKA(4, 4), EXPON(10), F(4),
1      FSUM(140, 2), H(70), IDOFX(20), LAYTYP(10), NMPLAY(10), NPE(70, 4),
1      ORIGK(10, 40), PLSQCE(1000, 2), PRSOLD(10, 1000), PULT(10, 1000),
1      PULTN(10, 1000), SK(70, 2), SSK(70, 2), TITLE(20), WDTLAY(10),
1      XSPEC(20), Z(71), DEFACT(5000), HLOAD(5000), ORIGN(10, 40), DSPCY(140),
1     XI(1000), XII(1000)
COMMON/CONE/P(140), STIFF(800)

INPUT PIECE SPECIFICATIONS

READ (1, 102) TITLE
FORMAT (2OA4)
READ (1, 104) UT1, UT2, UT3, UT4, UT5, UT6, UT7
FORMAT (7A4, 6X)
READ (1, *) LIST, IPRT, ISEG, NPRT

READ (1, *) NDF, RM, NSPE, NSLAY, NWTABLE
READ (1, *) E, WIDTH, WATWGT

INPUT SOIL PROFILE

------------------
NMSUM = 0
DO 110 IS = 1, NSLAY
   READ(1, *) LAYTP(IS), NMPLAY(IS), BLKWGT(IS), WDTLAY(IS), X2(IS)
   CYCO1170
   IF(IS.LT.NWTBGL) GO TO 106
   CYCO1180
   EFFWGT(IS) = BLKWGT(IS) - WATWG
   CYCO1190
   GO TO 108
   CYCO1200
   EFFWGT(IS) = BLKWGT(IS)
   CYCO1210
   NMSUM = NMSUM + NMPLAY(IS)
   CYCO1220
   108 CONTINUE
   CYCO1230
   IF(NMSUM.EQ.NM) GO TO 114
   CYCO1240
   WRITE(3, 112)
   CYCO1250
   112 FORMAT(/, 5X, 'INCORRECT INFORMATION ABOUT THE NUMBER OF ',
       CYCO1260
       'MEMBERS',/ , 7X, 'IN EACH LAYER HAS BEEN PROVIDED. ',
       CYCO1270
       '/ , 5X, 'EXECUTION HAS TERMINATED.')
       CYCO1280
   GO TO 300
   CYCO1290
   CY
   PRINT HEADINGS
   --------------------
   CY
   WRITE(3, 116) TITLE
   CY
   114 WRITE(3, 116) TITLE
   CY
   116 FORMAT('1', '/ , 20A4,')
   CY
   WRITE(3, 118)
   CY
   118 FORMAT(/ , T5, 'SOLUTION FOR LATERALLY LOADED PILE' / , T5,
      CY
      34('=', ')
      CY
   INITIALISE VARIABLES AND SET ARRAYS TO ZERO
   -------------------------------------------
   CY
   119 NBAND = 4
   CY
   IABC = 1
   CY
   PILENG = 0.
   CY
   NMP1 = NM + 1
   CY
   NDFP1 = NDF + 1
   CY
   DO 120 I = 1, NMP1
      CY
      H(I) = 0.
      CY
      Z(I) = 0.
      CY
      SSK(I, 1) = 0.
      CY
      SSK(I, 2) = 0.
      CY
      SK(I, 1) = 0.
      CY
      SK(I, 2) = 0.
      CY
   120 CONTINUE
   CY
   DO 122 I = 1, NDFP1
      CY
      P(I) = 0.
      CY
      DSPL(I) = 0.
      CY
      DSPSUM(I) = 0.
      CY
      FSUM(I, 1) = 0.
      CY
      FSUM(I, 2) = 0.
      CY
   122 CONTINUE
   CY
   READ OR CALCULATE SEGMENT LENGTHS
   -------------------------------
   CY
   IF(NSLAY.GT.1.AND.ISEG.NE.1) GO TO 124
   CY
   GO TO 128
   CY
124 WRITE(3,126)
126 FORMAT(1X,'WHEN THE SOIL PROFILE IS NOT UNIFORM, THE ',
1 'SEGMENT LENGTHS',1X,'MUST BE INPUT MANUALLY',
1 '/5X,'EXECUTION IS TERMINATED')
1 GO TO 300
C
128 IF(ISSEG.EQ.1) GO TO 134
C
READ(1,*) PILENG
IF(NM.EQ.1) GO TO 138
RN8=DFLOAT(NM)
PLNM=2.*PILENG/RN8
IF(PLNM.GE.WIDTH) GO TO 132
C
WRITE(3,130)
130 FORMAT(1X,'THE NUMBER OF MEMBERS IS GREATER THAN THIS',
1 ' PROGRAM PERMITS.',1X,'EXECUTION HAS TERMINATED')
1 GO TO 300
C
C SUBROUTINE TO DIVIDE PILE INTO SEGMENTS
132 CALL DIVIDE(H,NM,NMP1,PILENG,RN8,WIDTH)
1 GO TO 140
C
134 READ(1,*) (H(J),J=1,NM)
DO 136 IC=1,NM
PILENG=PILENG+H(IC)
136 CONTINUE
GO TO 140
C
138 H(1)=PILENG
C
C COMPUTE THE DEPTH AT EACH NODE
C ---------------------------------
C
140 DO 142 IH = 1,NM
1 JH = IH+1
2 Z(JH) = Z(IH)+H(IH)
142 CONTINUE
C
C PRINT PILE SPECIFICATIONS AND SOIL DATA
C ---------------------------------------
C
144 WRITE(3,144) NM,NDF,E,UTS,WIDTH,UT1,PILENG,UT1,
1 NSLAY,DLFAC,NWTABLE
146 FORMAT(1X,'THE PILE IS DIVIDED INTO ',I3,' MEMBERS',1X,}
1 '6X,'NUMBER OF DEGREES OF FREEDOM = ',I3,1X,5X,
1 'MODULUS OF ELASTICITY OF THE PILE = ',E10.5,1X,A4,1X,}
1 '7X,'WIDTH OF THE PILE = ',F10.3,A4,1X,8X,}
1 'LENGTH OF THE PILE = ',F10.3,A4,1X,5X,}
1 'THE SOIL IS DIVIDED INTO ',I3,' LAYERS',1X,6X,}
1 'GROUND LINE DEFLECTION FACTOR, DFLAC = ',F6.3,1X,}
1 '7X,'THE WATER IS COINCIDENT WITH THE UPPER ',}
1 'BOUNDARY OF LAYER ',I2,1X)
C
WRITE(3,146) UT1,UT1
146 FORMAT(1X,'MEMNO',6X,'DOF1',2X,'DOF2',2X,'DOF3',2X,}
1 'DOF4',8X,'LENGTH',8X,'DEPTH',1X,47X,'(',A4,')',}
1 7X,('',A4,')')
C
K = 1
DO 148 I = 1, NM
NPE(I, 1) = K
NPE(I, 2) = K + 1
NPE(I, 3) = K + 2
NPE(I, 4) = K + 3
K = K + 2
148 CONTINUE
C
DO 152 I = 1, NM
IP1 = I + 1
WRITE(3, 150) I, (NPE(I, J), J = 1, 4), H(I), Z(IP1)
150 FORMAT(16, 4X, 4I6, 4X, F10.3, 3X, F10.3)
152 CONTINUE
C
PRINT SOIL PROFILE
---------------------
C
WRITE(3, 154) UT1, UT5, UT6
154 FORMAT(/, 5X, 'SOIL PROFILE', //, 5X, 12 ('-'), //, 1X, 'LAYER NO. ',
3X, 'NO MEMBERS', 3X, 'MODEL NO', 4X, 'WIDTH', 8X,
1 'BULK WT', 6X, 'EFFECT WT', //, 4IX, ('', A4, ')',
10X, ('', A4, ')', '10X, ('', A4, ')')
C
DO 158 IN = 1, NSLAY
WRITE(3, 156) IN, NMPLAY(IN), LAYTYP(IN), WDTLAY(IN),
1 BLDWTG(IN), EFFWTG(IN)
156 FORMAT(7X, I2, 11X, I2, 9X, I2, 4X, F5.2, 9X, F7.4, 9X, F7.4)
158 CONTINUE
C
READ AND WRITE SPECIFIED BOUNDARY CONDITIONS
---------------------------------------------
C
IF(NSPX.LE.0) GO TO 168
C
DO 160 N=1, NSPX
READ(1, *) IDOFX(N), XSPEC(N)
160 CONTINUE
C
WRITE(3, 162)
162 FORMAT(/, 5X, 'SPECIFIED BOUNDARY CONDITIONS', //,
5X, 29 ('-'), //)
WRITE(3, 164) (IDOFX(I), I=1, NSPX)
164 FORMAT(/, 5X, 'THE DEGREES OF FREEDOM WHOSE DISPLACEMENTS',
1 ' ARE SPECIFIED:', //, 10I8, //)
WRITE(3, 166) (XSPEC(I), I=1, NSPX)
166 FORMAT(5X, 'THE SPECIFIED VALUES OF DISPLACEMENT ARE:',
1 //, 10F8.5, //)
C
READ AND WRITE THE LOADING SEQUENCE TO BE APPLIED
-----------------------------------------------
C
READ(1, *) NAPPLD, DELTB, DELTHF, DEGPA, COEFF, COEFF1, IS
READ(1, *) ((PLSQCE(I, J), J=1, 2), I=1, NAPPLD)
C
WRITE(3, 170) NAPPLD, DELTB, UT4, DELTHF, UT3, UT4, UT3
170 FORMAT(/, 5X, 'LOADING SEQUENCE TO BE APPLIED', //, 5X,
C
CYC02330
CYC02340
CYC02350
CYC02360
CYC02370
CYC02380
CYC02390
CYC02400
CYC02410
CYC02420
CYC02430
CYC02440
CYC02450
CYC02460
CYC02470
CYC02480
CYC02490
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CYC02780
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CYC02810
CYC02820
CYC02830
CYC02840
CYC02850
CYC02860
CYC02870
CYC02880
CYC02890
CYC02900
30('-'),//,7X,'NO OF APPLIED LOADS =',I3,/, CYCO2910
9X,'BENDING MOMENT INCREMENT =',F10.4,1X,A4,/, CYCO2920
9X,'HORIZONTAL FORCE INCREMENT =',F10.4,1X,A4,/, CYCO2930
10X,'MOMENTS',10X,'LAT. FORCES',//, CYCO2940
13X,'(',A4,')',14X,'(',A4,')',//, CYCO2950
WRITE(3,172)((PLSQCE(I,J),J=1,2),I=1,NAPPLD) CYCO2960
FORMAT(7X,F10.3,10X,F10.3) CYCO2970
C
DC 176 I=1,NSLAY
DO 174 J=1,40
PRSLD(I,J)=0.
174 CONTINUE
176 CONTINUE
C
BMTOT = 0.
HFTOT = 0.
C
DEF(I)=0
HLOAD(I)=0
II=2
C
MAIN PROGRAMMING LOOP
------------------------
C
DO 250 ISQCE=1,NAPPLD
C
WRITE(*,*),ISQCE
C
X=9
PR=MOD(ISQCE,2)
IF(PR.EQ.1)GO TO 178
DC 177 I = 1,NDF
DSPCY(I)=DSPL(I)
177 CONTINUE
C
178 XX=0
C
DO 179 I=1,NDF
DSPMAX(I) = DSPL(I)
DSPL(I) = 0.
179 CONTINUE
C
DETERMINATION OF THE NUMBER OF LOADING INCREMENTS REQUIRED
----------------------------------------------------------
C
BNMAX = PLSQCE(ISQCE,1) - BMTCT
HMX = PLSQCE(ISQCE,2) - HFTOT
IF(HMX.EQ.0.)GO TO 186
RLI = (HMX/DETHF)
ALI = ABS(RLI)
NLI = DINT(ALI)
HFINC = HMX/ALI
HF = HMX - NLI*HFINC
IF = DINT(HF)
IF(IFH.FT.2.AND.IFH.LT.2)GO TO 180
NLI = NLI+1
GO TO 182
180 HF = HF + HFINC
182 IF(BM.MAX.EQ.0.) GO TO 184
BM.INCR = BM.MAX/ALI
BM = (HF/HF.MAX)*BM.MAX
GO TO 194
C
184 BM = 0.
BM.INCR = 0.
GO TO 194
C
186 IF(BM.MAX.EQ.C.) GO TO 190
HF.INCR = 0.
HF = 0.
RLI = BM.MAX/DELT.BM
ALI = ABS(RLI)
NLI = DINT(ALI)
BM.INCR = BM.MAX/ALI
BM = BM.MAX - NLI*BM.INCR
IBM = DINT(IBM)
IF(IBM.GT.-1.AND.IBM.LT.1) GO TO 188
NLI = NLI + 1
GO TO 194
188 BM = BM + BM.INCR
GO TO 194
C
190 WRITE(3,192) ISQ.CE
FORMAT(/, 'THE LOADS REMAINED UNCHANGED FOR LOAD CYCLE ',
1 'NO', I4, '///)
GO TO 290
C
194 ITERC = 0
C
C VARIABLE INITIALISATION
-----------------------
C
196 II = 0.
ITERC = ITERC + 1
PRTC = MOD(ITERC, NPRT)
C
DO 198 I = 1, NDF
P(I) = 0.
198 CONTINUE
C
C LOAD MATRIX FORMATION
-----------------------
C
198 IF(ITERC.NE.1) GO TO 200
P(1) = BM
P(2) = HF
BMTOT = BMTOT + BM
HFTOT = HFTOT + HF
GO TO 202
C
200 P(1) = BM.INCR
P(2) = HF.INCR
BMTOT = BMTOT + BM.INCR
HFTOT = HFTOT + HF.INCR
C
202 IF(PRTC.NE.0.AND.ITERC.LT.NLI) GO TO 205
IF (ISQCE.EQ.18) WRITE (3, 204) ITERC, ISQCE, BMTOT, UT4, HITOT, UT3
204   FORMAT (/42X, 'LOAD STEP ', I3, 3X, 'CYCLE NUMBER ', I4,
     $ 1   /5X, 'BENDING MOMENT ', F10.4, 1X, 'HORIZONTAL FORCE ', F10.4, 1X, 'A4', /)
     $ 
C 
C COMPUTATIONAL PORTION
C 
C ------------------------
C 
C CALCULATION OF THE SOIL STIFFNESS AT THE NODES
C 
C -------------------------------------------
C 
C 205   NMCO = 1
C  
C     DO 216 IC = 1, NSLAY
C         NMPL = NMPLAY(IC)
C     
C         IF (LAYTP(IC).EQ.1) GO TO 206
C         IF (LAYTP(IC).EQ.2) GO TO 208
C         IF (LAYTP(IC).EQ.3) GO TO 210
C         IF (LAYTP(IC).EQ.4) GO TO 212
C 
C     SUBROUTINE TO DEAL WITH LINEAR SOIL
C 
C 206   IF (IABC.NE.1) GO TO 214
C         CALL LINEAR(H, SK, IC, NMCO, NMPL, XI, XII)
C         GO TO 216
C 
C         SUBROUTINES DESIGNED TO MODEL NON-LINEAR SOIL BEHAVIOUR
C 
C 208   CALL MPCLAY
C         1 (DSPL, DSPMAX, EXPON, H, ORIGK, FSOLD, PULT, PULTIN,
C            SK, WDTLAY, Z, DSPCY, XI, ORIGN, DEGPA, ISQCE, IABC, IC, ITERC, NMCO,
C            NMPL, WIDTH, PRT, COEFF, XII, COEFF1)
C         GO TO 216
C 
C 210   CALL MPSAND(DSPL, DSPMAX, EFFWGT, H, ORIGK, FSOLD, PULT, PULTIN,
C            1 NDF, DSPCR, PRT, DEGPA, ISQCE, SK, WDTLAY, IABC, IC, ITERC, NMCO, NMPL,
C            XI, WIDTH, COEFF, XII, COEFF1)
C         GO TO 216
C   
C 212   CALL -------
C         GO TO 216
C 
C 214   NMCO = NMCO+NMPL
C 
C 216   CONTINUE
C         IABC = 2
C 
C         CALCULATION OF THE SOIL SPRINGS AT THE NODES
C 
C -------------------------------------------
C 
C    DO 220 K = 1, NM
C         SSK(K, 1) = H(K)*WIDTH*(3.*SK(K,1)+SK(K,2))/8.
C         SSK(K, 2) = H(K)*WIDTH*(SK(K,1)+3.*SK(K,2))/8.
C 220   CONTINUE
C 
C PRINT THE SOIL STIFFNESS AND THE SOIL SPRINGS
C ---------------------------------------------------------------
C IF(IPRT.NE.1)GO TO 230
C WRITE(3,224)UT6,UT7
C FORMAT(//,3X,'MEMNO',12X,'SOIL STIFFNESS',17X,'SOIL SPRINGS'
     1 ,/33X,(',A4,')',23X,(',',A4,')')
C 224 DO 228 K = 1,NM
C WRITE(3,226)K,SK(K,1),SK(K,2),SSK(K,1),SSK(K,2)
C 226 FORMAT(T5,T5,3X,2F12.3,6X,2F12.3)
C 228 CONTINUE
C C
C NT = NDF*4
C DO 232 I = 1,NT
C STIFF(I) = 0.
C 232 CONTINUE
C C ENTRY POINT TO RECONSTRUCT THE EKA MATRIX
C ---------------------------------------------------------------
C C I = 0
C I = I + 1
C DO 240 K = 1,4
C DO 238 J = 1,4
C EA(K,J) = 0.
C 238 CONTINUE
C 240 CONTINUE
C EA(1,1) = 1.
C EA(1,2) = -1./H(I)
C EA(1,4) = 1./H(I)
C EA(2,2) = -1./H(I)
C EA(2,3) = 1.
C EA(2,4) = 1./H(I)
C EA(3,2) = 1.
C EA(4,4) = 1.
C C DO 244 K = 1,4
C DO 242 J = 1,4
C EK(K,J) = 0.
C 242 CONTINUE
C 244 CONTINUE
C EK(1,1) = 4.*E*XI(I)/H(I)
C EK(1,2) = .5*EK(1,1)
C EK(2,1) = EK(1,2)
C EK(2,2) = EK(1,1)
C EK(3,3) = SSK(I,1)
C EK(4,4) = SSK(I,2)
C C CONSTRUCTION OF THE KRA MATRIX, FOR EACH ELEMENT
C ---------------------------------------------------------------
C C DO 250 K = 1,4
C DO 246 J = 1,4
C EKA(K,J) = 0.
C DO 246 L = 1,4
C EKA(K,J) = EKA(K,J) + EK(K,L)*EA(L,J)
C 246 CONTINUE
C 250 CONTINUE
CONTINUE

IF(II.GT.0) GO TO 274

CONSTRUCTION OF THE GKA MATRIX (EGKA), FOR EACH ELEMENT

DO 256 K = 1, 4
     DO 254 J = 1, 4
         EGKA(K, J) = 0.
     DO 252 L = 1, 4
         EGKA(K, J) = EGKA(K, J) + EA(L, K) * EKA(L, J)
  252 CONTINUE
  254 CONTINUE
  256 CONTINUE

FORMATION OF THE BANDED PORTION OF THE GLOBAL MATRIX

CALL BANDM(EGKA, NPE, STIFF, I)

IF(I.LT.NM) GO TO 236
WHEN I-NM BAND MATRIX HAS BEEN FORMED

IF(NSPX.LE.0) GO TO 258

MODIFICATION OF THE BAND MATRIX AND THE P MATRIX FOR BOUNDARY
CONDITIONS

CALL MODIF(IDOFX, P, STIFF, XSPEC, NDF, NSPX)

IF(LIST.LE.0) GO TO 266
WRITE(3, 260)
  260 FORMAT(/, 5X, 'THE BAND MATRIX ADJUSTED FOR ',
          'BOUNDARY CONDITIONS', /)
     N1 = 1
     N2 = 4
     DO 264 I = 1, NDF
         WRITE(3, 262) I, (STIFF(JJ), JJ=N1, N2)
     262 FORMAT(3X, I3, 4(X, F10.0))
     N1 = N2 + 1
     N2 = N2 + 4
     IF(N2.GT.NT) N2 = NT
  264 CONTINUE

REDUCTION OF THE BAND MATRIX

CALL SYMSOL(NBAND, NDF)

CALCULATION OF THE TOTAL DISPLACEMENTS

DO 268 I = 1, NDF
     DSPL(I) = DSPL(I) + P(I)
     DSPANM(I) = DSPANM(I) + P(I)
  268 CONTINUE
CONTINUE

PRINT THE OUTPUT HEADINGS

IF(PRTC.NE.0.AND.ITERC.LT.NLI) GO TO 272
IF(ISQCE.EQ.IS) WRITE(3,270)UT4,UT3,UT1

270 FORMAT(///,10X,'INTERNAL FORCES AND ASSOCIATED MEMBER',
1 'DISPLACEMENTS',/10X,51('-'),/3,T3,'DOF',
1 '','T13,'MOMENTS',4X,'ROTATIONS',12X,
1 'SOIL REACTIONS',5X,'DEFLECTIONS',/15X,'(',A4,
1 ')',7X,'(RADS)',20X,'(',A4,',')',10X,'(',A4,')',/)

II = 1
IDOF = 1

REUSING THE ELEMENT DATA TO RECOMPUTE THE EKA MATRIX TO

GO TO 234

CALCULATION OF THE FORCE MATRIX FOR EACH ELEMENT

DO 278 NN = 1,4
   F(NN) = 0.
   DO 276 KK = 1,4
      NS = NPE(I,KK)
      F(NN) = F(NN) + EKA(NN,KK)*P(NS)
      CONTINUE
   CONTINUE

PRINT THE MOMENTS, ROTATIONS, SOIL REACTIONS AND DEFLECTIONS

IK = IDOF + 1
FSUM(IDOF,1) = FSUM(IDOF,1) + F(1)
FSUM(IDOF,1) = FSUM(IDOF,1) + F(3)
FSUM(IDOF,2) = FSUM(IDOF,2) + F(4)
IF(PRTC.NE.0.AND.ITERC.LT.NLI) GO TO 282
IF(IDOF.EQ.1) DEF(III) = 1000*DSPSUM(IK)
HLOAD(III) = HPTOT
IF(ISQCE.EQ.IS) WRITE(3,280) IDOF,FSUM(IDOF,1),DSPSUM(IDOF),
1 FSUM(IDOF,1),FSUM(IDOF,2),DSPSUM(IK)
280 FORMAT(T3,I3,F14.3,F13.5,4X,'|',4X,2F10.3,F13.5)
282 IDOF = IDOF + 2

IF(I.LT.NM) GO TO 236

IK = IDOF+1
FSUM(IDOF,1) = FSUM(IDOF,1) + F(2)
IF(PRTC.NE.0.AND.ITERC.LT.NLI) GO TO 286
IF(ISQCE.EQ.IS) WRITE(3,284) IDOF,FSUM(IDOF,1),DSPSUM(IDOF),
1 DPSUM(IK)
284 FORMAT(T3,I3,F14.3,F13.5,4X,'|',24X,F13.5,/)
290 CONTINUE
    WRITE (3, 293) UT2, UT3
293 FORMAT (/, 'VARIATION OF PILE TOP DISPLACEMENT WITH', C YCS06400
    1 1X, 'HORIZONTAL LOAD', '/', 2X, 55 ('-'), '/', C YCS06410
    1 2X, 'DISPLACEMENT', 6X, 'LOAD', '/', 6X, ('A4', '),', C YCS06420
    1 8X, ('A4', ')')
    WRITE (3, 294) (DEF(JJJ), HLOAD(JJJ), JJJ=1, (III-1))
294 FORMAT (F13.5, F13.2)
C
C 300 STOP
END

C =============
C SUBROUTINE BANDM(EGKA, NPE, STIFF, I)
C =============
C SUBROUTINE TO DEVELOP THE BAND MATRIX FROM GLOBAL MATRICES
C
C IMPLICIT DOUBLE PRECISION (A-H,O-Z)
DIMENSION EGKA(4, 4), NPE(70, 4), STIFF(800)
C
DO 402 K = 1, 4
    NS1 = (NPE(I, K) - 1) * 4
    DO 400 J = 1, 4
        IF (NPE(I, J) .LT. NPE(I, K)) GO TO 400
        NS2 = NPE(I, J) - NPE(I, K) + 1
        STIFF(NS1 + NS2) = STIFF(NS1 + NS2) + EGKA(K, J)
400 CONTINUE
402 CONTINUE
RETURN
END

C ================
C SUBROUTINE DIVIDE(H, NM, NMPL, PILENG, RN, WIDTH)
C ================
C SUBROUTINE TO DIVIDE PILE INTO NM NUMBER OF PILE ELEMENTS,
C WITH INCREASING LENGTH DOWN THE PILE
C
C IMPLICIT DOUBLE PRECISION (A-H,O-Z)
DIMENSION H(70)
C
CONST = 2. * PILENG/WIDTH
ROLD = CONST/RNM
RNMP1 = DFLOAT(NMP1)
C
450 RNEW = (RNM**(ROLD**NMPL) - CONST) / (RNMP1**(ROLD**NM) - CONST - 1.)
REM = DABS(ROLD - RNEW)
IF (REM .LE. 0.00001) GO TO 452
ROLD = RNEW
GO TO 450
C
452 DO 454 I = 1, NM
    H(I) = WIDTH * (RNEW**I) / 2.
454 CONTINUE
C
RETURN
END
C
SUBROUTINE LINEAR(H, SK, IC, NMCO, NMPL, XI, XII)
SUBROUTINE TO CALCULATE THE LINEAR SOIL MODULUS

IMPLICIT DOUBLE PRECISION (A-H,O-Z)
DIMENSION H(70), SK(70,2), XI(1000), XII(1000)

READ(1,*)AC,BC,CC

X=0.
IF(BC.EQ.0.)GO TO 502

DO 500 K = 1,NMPL
   SK(NMCO,1) = AC + BC*X**CC
   X=X+H(NMCO)
   SK(NMCO,2) = AC + BC*X**CC
   NMCO = NMCO+1
500 CONTINUE
GO TO 506

DO 504 K = 1,NMPL
   XI(NMCO)=XII(IC)
   SK(NMCO,1) = AC
   SK(NMCO,2) = AC
   NMCO = NMCO+1
504 CONTINUE

RETURN
END

SUBROUTINE MODIF(IDOFX,P,STIFF,XSPEC,NDF,NSPX)

SUBROUTINE TO ADJUST THE BAND MATRIX AND P-MATRIX FOR BOUNDARY
CONDITIONS, ZERO OR SPECIFIED DISPLACEMENTS AND ROTATIONS

IMPLICIT DOUBLE PRECISION (A-H,O-Z)
DIMENSION IDOFX(20), P(140), STIFF(800), XSPEC(20)

DO 556 IZ = 1,NSPX
   NPZI = IDOFX(IZ)
   LL = (NPZI-1)*4 + 1
   STIFF(LL) = 1.0
   P(NPZI) = XSPEC(IZ)
   DO 550 IL = 2,4
      NPZP = NPZI + IL - 1
      IF(NPZP.GT.NDF)GO TO 550
      P(NPZP) = P(NPZP) - STIFF(LL+IL-1)*XSPEC(IZ)
550 CONTINUE

DO 552 IN = 2,4
   NPZM = NPZI - IN + 1
   NPZ1 = (NPZI-IN)*4 + IN
   IF(NPZM.LE.0.)GO TO 552
   P(NPZM) = P(NPZM) - STIFF(NPZ1)*XSPEC(IZ)
552 CONTINUE

DO 554 K = 2,4
   STIFF(LL + K-1) = 0.0
   K1 = NPZI - K

IF(K1.LT.0) GO TO 554
   KZ = K1*4 + K
   C
554 CONTINUE
556 CONTINUE
C
RETURN
C
END
C
SUBROUTINE MPCLAY(DSPL, DSPMAX, EXPON, H, ORIGK, PRSOLD, PULT, PULTIN,
1     SK, WDTLAY, Z, DSPCY, XI, ORIG, DEGPA, ISQCE, IABC, IC, ITERC, NMCO, NMPL,
1     WIDTH, FRT, COEFF, XIII, COEFF1)
C
------------------------------------------------------------------
C
IMPLICIT DOUBLE PRECISION (A-H, O-Z)
C
DIMENSION DSPL(140), DSPMAX(140), EXPON(10), H(70), ORIGK(10,40),
1     PRSOLD(10,1000), PULT(10,1000), PULTIN(10,1000), SK(70,2),
1     WDTLAY(10), Z(71), DSPCY(140), XI(1000), ORIG(10,40), XII(1000)
C
IF(IABC.NE.1) GO TO 607
C
READ (1,*)IC,AC,BC,CC,EC,FC,EXPON(IC)
C
X = 0.
C
SURCHG = 0.
C
NMPL1 = NMPL + 1
C
ICML = IC - 1
C
ICOUNT = NMCO
C
CALCULATION OF THE INITIAL ULTIMATE PRESSURE AND SOIL STIFFNESS
C
------------------------------------------------------------------
C
600 DO 605 J=1,NMPL1
C
   CCONSUN = EC + FC*X
   PCONST = 5. + ((Z(ICOUNT)/WIDTH) + 2.)
   IF(PCONST.LT.12.)PCONST = 12.
   PULTIN(IC,J) = (PCONST*CONSUN)
   PULT(IC,J) = PULTIN(IC,J)
   IF(BC.EQ.0.) GO TO 602
   ORIGK(IC,J) = AC + BC * X**CC
   GO TO 604
   ORIGK(IC,J) = AC
604 X = X + H(ICOUNT)
   ICOUNT = ICOUNT + 1.
   CONTINUE
C
605 DO 606 J = 1,NMPL1
C
   ORIG(IC,J) = ORIGK(IC,J)
   CONTINUE
C
GO TO 618
C
CALCULATION OF THE NEW ULTIMATE PRESSURE FOR EACH LOAD CYCLE
C
------------------------------------------------------------------
C
607 IF(ITERC.GT.1) GO TO 618
C
NMPL1 = NMPL + 1
C
NMDMY = 2*NMCO
C
DO 608 J = 1,NMPL1
C
   DEFN = DABS(DSPCY(NMDMY))
   IF(FRT.EQ.1) ORIG(IN(J)) = (DFLOAT(ISQCE)*
   1   *(NEGPA*DEFN)) * ORIG(IN(J))
   CONTINUE
C
GO TO 618
C
}
IF (PRT.EQ.0) ORIGIN(IC,J) = (DFLOAT(ISQCE) * ORIGIN(IC,J)
  1   * (-COEFF1*DEFN) * ORIGK(IC,J)
  NNDMY = NNDMY+2
  608 CONTINUE
  609 EXP = EXPON(IC)
  NMPL = NMPL+1
  NNDMY = 2*NMC0
  DO 616 I = 1,NMPL
    DEFL = DABS (DSPMAX(NNDMY))
    IF (ORIGIN(IC,L).EQ.0.0 OR PULT(IC,L).EQ.0.0 OR DEFL.EQ.0.0)
      1   GO TO 612
    PCONST = ORIGIN(IC,L)*DEFL
    PSOLD = PCONST
    PRESS = PSOLD - ((PCONST - PCONST*((PSOLD/PULT(IC,L))**EXP) -
      1   PSOLD)/(EXP*PCONST*((PSOLD**EXP - 1.0)/
      1   (PULT(IC,L)**EXP) - 1.0))
    REM = DABS (PRESS-PSOLD)
    IF (REM.LE.0.001) GO TO 614
    PSOLD = DABS (PRESS)
    GO TO 610
  612 PRESS = 0.
  614 TEMP = PRESS-PSOLD(IC,L)
  PSOLD(IC,L) = DABS (TEMP)
  DEFN = DABS (DSPCY(NNDMY))
  PULT(IC,L) = PULTIN(IC,L)+COEFF*PSOLD(IC,L)
  IF (PRT.EQ.1) PULT(IC,L) = PULTIN(IC,L)
  NNDMY = NNDMY+2
  616 CONTINUE
C
C   CALCULATION OF THE SOIL STIFFNESS
C   -------------------------------
C
C 618 X = 0.
  EXP = EXPON(IC)
  WRITE(3,*),EXP
  DO 632 K = 1,NMPL
    KA = 2*NMC0
    KP = K
    DO 630 IK = 1,2
      XI(NMC0) = XI2(IC)
      IF (ORIGIN(IC,KP).EQ.0.0 OR PULT(IC,KP).EQ.0.0) GO TO 626
      DEFL = DABS (DSPFL(KA))
      IF (DEFL.EQ.0.0) GO TO 622
      PCONST = ORIGIN(IC,KP)*DEFL
      PSOLD = PCONST
      PRESS = PSOLD - ((PCONST - PCONST*((PSOLD/PULT(IC,KP))**EXP) -
        1   PSOLD)/(EXP*PCONST* (PSOLD**EXP - 1.0)/
        1   (PULT(IC,KP)**EXP) - 1.0))
      REM = DABS (PRESS-PSOLD)
      IF (REM.LE.0.001) GO TO 624
      PSOLD = DABS (PRESS)
      GO TO 620
    630 CONTINUE
  632 SK(NMC0,IK) = ORIGIN(IC,KP)
  GO TO 628
C
C   WRITE(3,*),IC,KP,EXP,ORIGIN(IC,KP),PRESS,PULT(IC,KP)
  624 SK(NMC0,IK) = ORIGIN(IC,KP)*((1.0 - ((PRESS/PULT(IC,KP))**EXP))
    **2) * (1.0 + (EXP-1.0)*((PRESS/PULT(IC,KP))**EXP))
    1  GO TO 628
626     SK(NMCO,IK)=0.
628     KP = KP + 1.
       KA = KA + 2
630     CONTINUE
       NMCO = NMCO + 1
632     CONTINUE
C
RETURN
END
C
SUBROUTINE MPSAND(DSPI, DSPMAX, EFFWTG, H, ORIGK, PRSOLD, PULT, PULTIN,
1      INDF, DSPCY, PRS, DEGPA, ISQCE, SK, WDTLAY, IABC, IC, ITERC, NMCO, NMPL,
1      XI, WIDTH, COEFF, XII, COEFF1)
C
-------------------------------------------------------------
C
IMPLICIT DOUBLE PRECISION (A-H, O-Z)
DIMENSION DSP1(140), DSPMAX(140), EFFWTG(10), H(70), ORIGK(10, 40),
1      DSPCY(140), PRSOLD(10, 40), PULT(10, 40), PULTIN(10, 40),
1      ORIG(10, 40), SK(70, 2), WDTLAY(10), EXPON(10), XI(1000), XII(1000)
C
IF(IABC.NE.1)GO TO 660
READ (1,*)AC, BC, CC, BETC, FACT, ANG, EXPON(IC)
C
X = 0.
SURCHG = 0.
NMPL1 = NMPL+1
ICM1 = IC-1
ICOUNT = NMCO
EFFEW = EFFWTG(IC)
    RDS = (ANG/180.) * 3.141592654
    CONS = DSIN(RDS)
    PK = (1.+ CONS)/(1.- CONS)
C
C CALCULATION OF THE SURCHARGE
-------------------------------------------------------------
C
IF(IC.EQ.1)GO TO 652
DO 650 L=1,ICM1
    SURCHG = SURCHG + (EFFWTG(L) * WDTLAY(L))
650     CONTINUE
C
C CALCULATION OF THE INITIAL ULTIMATE PRESSURE AND SOIL STIFFNESS
-------------------------------------------------------------
C
652     DO 658 J=1,NMPL1
       BETA = BETC + FACT*X
       PULTIN(IC,J) = BETA*PK*(EFFEW*X+ SURCHG)
       PULT(IC,J) = PULTIN(IC,J)
       IF(BC.EQ.0)GO TO 654
       ORIGK(IC,J) = AC + BC * X**CC
       GC TO 656
654     ORIGK(IC,J) = AC
656     X = X + H(ICOUNT)
       ICOUNT = ICOUNT + 1.
658     CONTINUE
DO 659 J=1,NMPL1
       ORIGN(IC,J)=ORIGK(IC,J)
659     CONTINUE
GO TO 669

C
CALCULATION OF THE NEW ULTIMATE PRESSURE FOR EACH LOAD CYCLE

C

660 IF (ITERC.GT.1) GO TO 669
NMPL1=NMPL+1
NMDMY=2*NMCO
DO 661 J=1,NMPL1
DEFN=DABS (DSPCY (NMDMY))
PRT=MOD (ISQCE,2)
IF (PRT.EQ.1) ORIGN(IC,J) = (DFLOAT(ISQCE)**(-DEGPA*DEFN)) * ORIGK(IC,J)
IF (PRT.EQ.0) ORIGN(IC,J) = (DFLOAT(ISQCE))
1   *(COEFF1*DEFN)) * ORIGK(IC,J)
NMDMY=NMDMY+2
661 CONTINUE
EXP=EXPON(IC)
NMPL1 = NMPL + 1
NMDMY = 2*NMCO
PRT=MOD (ISQCE,2)
DO 667 L=1,NMPL1
DEFL = DABS (DSPMAX(NMDMY))
IF (ORIGN(IC,L).EQ.0..OR.PULT(IC,L).EQ.0..OR.DEFL.EQ.0.)
   GO TO 663
PCONST=ORIGN(IC,L)*DEFL
PSOLD=PCONST
PRESS=PSOLD-((PCONST-PCONST*((PSOLD/PULT(IC,L))**EXP))-1.
1     PSOLD) /((-EXP*PCONST*((PSOLD**EXP-1.)) / (PULT(IC,L)**EXP))-1.))
REM=DEFL-PRESS
IF (REM.LE.0.001) GO TO 664
PSOLD=DEFL
PRESS=PSOLD
663 PRESS = 0.
664 TEMP = PRESS-PSOLD(IC,L)
PSOLD(IC,L) = DABS (TEMP)
DEFN=DABS (DSPCY (NMDMY))
PULT(IC,L) = PULTIN(IC,L)+COEFF*
1
PSOLD(IC,L)
IF (PRT.EQ.1) PULT(IC,L) = PULTIN(IC,L)
666 NMDMY = NMDMY + 2
667 CONTINUE

C
CALCULATION OF THE SOIL STIFFNESS

C

669 X = 0.
EXP=EXPON(IC)
DO 676 K = 1,NMPL
KA = 2*NMCO
KP = K
DO 675 IK=1,2
XI(NMCO)=XII(IC)
IF (ORIGN(IC,K).EQ.0..OR.PULT(IC,K).EQ.0.) GO TO 673
DEFL = DABS (DSPL(KA))
IF (DEFL.EQ.0) GO TO 671
PCONST=ORIGN(IC,K)*DEFL
PSOLD=PCONST
PRESS=PSOLD-((PCONST-PCONST*((PSOLD/PULT(IC,K))**EXP))-1.

20
1 PSOLD) / (-EXP*PCONST* ((PSOLD**((EXP-1.))/ (PULT(IC,KP)**EXP)) -1.)) CYC09870
   REM=DABS(PRESS-PSOLD)
   IF(REM.LE.0.001) GO TO 672
   PSOLD=DABS(PRESS)
   GO TO 670
671 SK(NMCO,IK)=ORIGIN(IC,KP)
   GO TO 674
672 SK(NMCO,IK)=(ORIGIN(IC,K)*((1.-(PRESS/PULT(IC,KP))**EXP)) **2)/((1.+(PRESS/PULT(IC,KP))**EXP)) GO TO 674
673 SK(NMCO,IK)=0.
674 KP = KP + 1.
   KA = KA + 2
675 CONTINUE
   NMCO = NMCO + 1
676 CONTINUE
C
   RETURN
END
C
C SUBROUTINE SYMSOL(NM,NN)
C
C SUBROUTINE TO REDUCE THE BAND MATRIX
C
C IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C COMMON/ONE/P(140),STIFF(800)
C DIMENSION A(200,4),B(140)
C EQUIVALENCE (B(1),P(1))
C
C   NJA=1
C   DO 702 JA=1,NN
C      DO 700 IA=1,4
C         A(JA,IA)=STIFF(NJA)
C         NJA=NJA+1
C   700 CONTINUE
C   702 CONTINUE
C
C REDUCE THE STIFFNESS MATRIX
C
C   DO 704 N=1,NN
C      DO 712 L=2,MM
C         C=A(N,L)/A(N,1)
C         I=N+L-1
C         IF(NN-I) 712,706,706
C      J=0
C      DO 708 K=L,MM
C         J=J+1
C         A(I,J)=A(I,J) -C*A(N,K)
C      708 CONTINUE
C      710 A(N,L)=C
C   712 CONTINUE
C   714 CONTINUE
C
C REDUCE LOAD VECTOR
C
C
716 DO 724 N=1,NN
   DO 720 L=2,MM
      I=N+L-1
      IF(NN-I) 722,718,718
518 B(I)=B(I)-A(N,L)*B(N)
720 CONTINUE
722 B(N)=B(N)/A(N,1)
724 CONTINUE
C
C BACK SUBSTITUTE
C
C
N=NN
726 N=N-1
   IF(N) 728,734,728
728 DO 732 K=2,MM
      L=N+K-1
      IF(NN-L) 732,730,730
730 B(N)=B(N)-A(N,K)*B(L)
732 CONTINUE
   GO TO 726
C
734 RETURN
END
### SAMPLE INPUT

**PREDICTION OF MAITAI RIVER PILE TEST RESULTS**

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B5 SAMPLE OUTPUT

PREDICTION OF MAITAI RIVER PILE TEST RESULTS

SOLUTION FOR LATERALLY LOADED PILE

THE PILE IS DIVIDED INTO 44 MEMBERS
NUMBER OF DEGREES OF FREEDOM = 90

MODULUS OF ELASTICITY OF THE PILE = .38000E+08 KPA

WIDTH OF THE PILE = 1.800 M
LENGTH OF THE PILE = 19.500 M

THE SOIL IS DIVIDED INTO 5 LAYERS
GROUND LINE DEFLECTION FACTOR, DFLAC = 0.000
THE WATER IS COINCIDENT WITH THE UPPER BOUNDARY OF LAYER

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LOAD STEP 4  CYCLE NUMBER 1

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LOAD STEP 8 CYCLE NUMBER 1

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