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This thesis has been accepted for the award of the degree in the Faculty of Engineering
Piled Raft Foundation Systems in Expansive Soils

by

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A thesis submitted for the degree of Doctor of Philosophy

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SYNOPSIS

The behaviour of piled raft foundation systems in expansive soils has been analysed in order to indicate the influence of the ground movement on the overall performance of the foundation systems. The effects of swelling and shrinking of soils on the various foundation components, and the respective redistribution of stresses, have been analysed and presented in this work. The influence of the ground movement on the load distribution pattern between raft and piles has been presented and its impact on the total and differential settlements, stresses underneath the raft and along the pile shaft and moments has been analysed and discussed. The parameters which influence the foundation behaviour such as the number and location of piles, stiffness of raft, magnitude of the ground movement, applied loading have also been considered in the analysis. In order to carry out these works, a method of analysis has been developed and, to undertake the various stages of formulation, a computer program PRAES (Piled Raft Analysis in Expansive Soils) has been written using FORTRAN language. The analyses have been carried out with three piled raft configurations i.e. $2^2$, $4^2$ and $6^2$ groups, subjected to swelling as well as shrinking soils. The analyses have also been extended to free standing piled groups, i.e. piled raft systems with a gap between the raft and the soil surface. The advantages and disadvantages of free standing pile groups have been compared with those of piled raft foundation systems when subjected to ground movement. In order to establish the validity of the method of analysis, the results obtained from the computer program have been compared with actual field and laboratory measurements.

A thorough review of the literature related to this work has been carried out to investigate the development of knowledge at various stages of research. The shortcomings of the present state of knowledge have been recognised.

To understand the complexity of piled raft behaviour in an expansive soil environment, it is necessary to understand the individual behaviour of the rafts and the piles in a swelling or shrinking soil condition. Therefore, raft and pile foundation systems have also been analysed including the effect of the various ground movement conditions. The computer programs RAE$_s$S (Raft Analysis in Expansive Soils) and PAE$_s$S (Pile Analysis in Expansive Soils) have been developed to carry out these analyses. The analyses have been carried out satisfying the conditions of the "soil-structure interaction" at the "soil-raft", "soil-pile" and "pile-raft" interfaces. The effect of "non-linear conditions" has also been incorporated by considering yielding of soil
underneath the raft and at the pile base and slip at the soil-pile interface and "lifting-off" between the soil surface and the raft.

Finally, a summary and conclusions have been presented and, attention has also been drawn towards the practical implications in the foundation performances due to these ground movements. The suggestions for further work have also been presented and discussed.
The candidate has carried out the work described in this thesis in the Department of Civil Engineering at the University of Sydney. The work has been carried out under the supervision of Professor H.G.Poulos of the Civil Engineering Department. The candidature of the author for the degree of Doctor of Philosophy commenced in July 1991, on a part-time basis.

The By-Laws of the University of Sydney require that a candidate for the degree of Doctor of Philosophy indicate which sections of the thesis are original. Any information or ideas derived from the references used during this research have been acknowledged in the text. In accordance with these By-Laws, the author claims originality for the following work:

1. The method of analysis of raft foundation in Chapter 3 is not wholly original, however the modelling of the ground movement due to the swelling and shrinking soils and incorporating these movements underneath the raft are claimed as original. The formulation of the soil interaction factors is also claimed as original.

2. The method of analysis of pile foundation systems in Chapter 4 is not wholly original. The interaction factors are used as recommended in Poulos and Davis (1980). The application of ground movement along the pile shaft is however claimed as original.

3. The method of analysis of the piled raft foundation systems including the effect of ground movements caused by swelling and shrinking soils in Chapter 5 is claimed as original. The method of incorporating the ground movement underneath the raft and along the pile shaft and the strategies adopted to incorporate the compatibilities at the "pile-soil-raft" interfaces are also considered original. The method limiting the pile capacity and its respective redistribution to various foundation components are also claimed as original. However, the methods for calculating the influence factors are obtained from Poulos and Davis (1980).

4. The results for the piled raft foundation systems in shrinking and swelling soils in Chapters 6 and 7 respectively are claimed as original.

The method of analysis developed in Chapter 3 has been utilised in publishing the following paper:

The following two papers have been presented by using the method of analysis developed in Chapter 5:


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Notations

All notation and symbols are defined where they first appear in the text or figure. For convenience, the more frequently used symbols and their meanings are listed below:

\( (A_p) \) : Surface area of \( i \) th pile shaft element

\( (A_c) \) : Area of \( i \) th raft element

\( a \) : Vector of nodal variables

\( A \) : Area of element

\( A_b \) : Pile base area

\( A_{si} \) : Surface area of pile element \( i \)

\( b \) : Width of the raft element

\( B_R \) : Width of Raft

\( C_a \) : Maximum adhesion strength between the pile and the soil

\( D \) : Pile shaft diameter in metre

\( D_b \) : Pile base diameter

\( E_p \) : Young's modulus of pile material

\( E_r \) : Young's modulus of raft

\( E_s \) : Young's modulus of soil

\( e \) : Edge distance

\( F \) : Force Vector

\( I_{bb} \) : Interaction between pile bases

\( I_{bi} \) : Interaction from the pile base to a point \( i \) adjacent to pile shaft

\( I_{nj} \) : Influence at \( n \) due to the stress at \( j \)

\( I_{pp} \) : Pile element to pile element interaction factors

\( I_{pr} \) : Pile to raft interaction factors

\( I_{rp} \) : Raft to pile interaction factors

\( I_{rr} \) : Displacement influence matrix of raft

\( I_{rs} \) : Matrix obtained coupling \( I_{rr} \) and \( I_{rs} \)

\( I_{ss} \) : Displacement influence matrix of soil

\( K \) : Stiffness Matrix for Raft

\( K_p \) : Relative stiffness of pile

\( K_r \) : Relative stiffness of raft (\( = K_r \))

\( l \) : Length of the raft element
\begin{itemize}
\item \(L_p\) : Pile Length
\item \(L_R\) : Length of Raft
\item \(m\) : Mound Index
\item \(M_{apx}\) : Moment about X-axis at pinned node
\item \(M_{apz}\) : Moment about Z-axis at pinned node
\item \(M_s\) : Moment in unstiffened raft
\item \(M_{st}\) : Moment in stiffened raft
\item \(M_{xx}\) : Moment in X - direction (= \(M_{yy}\), for symmetrical condition)
\item \(M_{yy}\) : Moment in Y - direction
\item \(P\) : Matrix containing contact stresses under raft elements
\item \(P\) : Applied load
\item \(p_j\) : Intensity of the applied load at elemental area \(j\) (Contact stress underneath the raft)
\item \(P_{tension}\) : Allowable tensile stress between the raft and the soil
\item \(P_{tot}\) : Total applied load
\item \(P_{ult}\) : Pile ultimate capacity
\item \(P_{yield}\) : Yielding of soil under the raft due to stress exceeding its ultimate capacity
\item \(q\) : Applied loading on raft
\item \(S\) : Spacing between piles
\item \(S_i\) : Vector of free field soil movement underneath each raft element
\item \(S_o\) : Maximum free field soil movement
\item \(S_p\) : Soil surface movement at the pile head
\item \(S_{pi}\) : Soil surface movement at the pile node \(i\)
\item \(S_r\) : Vector of soil surface movement underneath the raft
\item \(S_v\) : Vector of soil surface movement along the pile shaft
\item \(S_z\) : Extent of ground movement along pile shaft
\item \(t\) : Raft thickness (= \(t_r\))
\item \(T\) : Translation at the centre of the raft
\item \(T_b\) : Thickness of beam in stiffened raft
\item \(T_s\) : Thickness of slab in stiffened raft
\item \(\gamma_c\) : Ground movement due to shrinking of soils
\item \(\gamma_e\) : Ground movement due to swelling of soils
\item \(\tau\) : Matrix containing shear stresses along the pile shaft elements and bases
\end{itemize}
\( \delta \) : Displacements
\( \sigma_p \) : Stress at the pile base
\( \tau_j \) : Shear stress along pile shaft element \( j \)
\( \delta_m \) :
\( \delta_{\text{max}} \) : Maximum displacements
\( \delta_p \) : Pile head settlement
\( \rho_p \) : Pile head settlement (= \( \delta_p \))
\( \delta_q \) : Vector of deflection at the center of each of the raft element for the pinned raft under the imposed loads
\( \nu_r \) : Poisson’s ratio of raft material
\( \delta_r \) : Displacement of raft element
\( \nu_s \) : Poisson’s ratio of soil
\( \delta_s \) : Soil deformation under raft element
\( \Delta_s \) : Differential settlement in unstiffened raft
\( \tau_{\text{slip}} \) : Shear stress along pile shaft beyond which pile soil slip takes place
\( \Delta_{st} \) : Differential settlement in stiffened raft
\( \theta_x \) : Rotation about X-axis at the centre of raft
\( \sigma_{\text{yield}} \) : Stress at the pile base, beyond which soil yielding takes place
\( \rho_z \) : Vertical displacement at depth \( z \) beneath the corner of a rectangular loaded area
\( \theta_z \) : Rotation about Y-axis at the centre of raft
\( \alpha_s \) : Settlement factor which indicates the reduction in the differential settlement due to introducing stiffened beams
\( \alpha_m \) : Moment factor which indicates increase in moment due to stiffening effect
CHAPTER 1

INTRODUCTION

1.1 THE NATURE OF THE PROBLEM

In recent years, piled raft foundation systems have been increasingly used to support large structures. The concept of using piles as settlement reducers underneath the raft has been increasingly popular and effective (Burland et al, 1977). In the majority of cases, the piles are designed to withstand the whole superimposed load, i.e., the load carried by the raft is totally neglected. However, in the actual situation, the raft also shares part of the load. Therefore, the design of a piled raft foundation system may be optimised by incorporating the load bearing features of the raft also. In a reactive soil environment, the load sharing characteristics between the piles and the raft may be influenced significantly by the ground movement, caused by the processes of swelling or shrinking of soils. In these situations, the soil surface underneath the raft and along the pile shaft changes its level with the seasonal moisture variation. As a result of this, the foundations are subjected to a different state of stress distribution to that when no ground movements act, and this may adversely effect the foundation performance. There have been several cases of structural failures and damage due to these ground movements. Many research engineers and practising agencies have carried out a significant amount of work in order to understand the behaviour of piles and raft systems in a reactive soil environment, where the movement of the ground is a potential problem. The influence of the ground movement on the foundation behaviour was identified and the codes of practices were modified to account for these situations. Some works are still in progress to refine the method of analysis as well as to obtain a thorough understanding of various conditions. Despite the awareness of the possible adverse effects of the ground movements on raft and pile foundation systems, the behaviour of piled raft foundation systems, when subjected to combined structural loading and ground movements, are not fully addressed. The methods to analyse the piled raft behaviour in these situations are not well-established.

The objective of this thesis is to develop a method which can be used to carry out the analysis of piled raft foundation systems in a "normal ground" condition as well as in a reactive soil environment. The behaviour of piled raft foundation systems, when subjected to a relative ground movement caused by the swelling or shrinking soils, will be thoroughly investigated by using the developed method of analysis. In addition to this, the methods to analyse piles
and raft foundations in reactive soils will also be developed, by adopting a "soil-structure interactive" approach. The behaviour of the piles and the rafts as independent foundation systems will be analysed including the effect of ground movement. In order to formulate these methods of analysis, the following computer programs are developed, using the FORTRAN 77 programming language:

- **RAEₚₗ**: Raft Analysis in Expansive Soils
- **PAEₚₗ**: Pile Analysis in Expansive Soils
- **PRAEₚₗ**: Piled Raft Analysis in Expansive Soils

These computer programs will be used extensively to analyse the behaviour of the foundation systems in swelling and shrinking soils. The performance of the free standing pile groups will also be analysed and compared with the performance of piled raft systems. The advantages and disadvantages of these foundation systems in a reactive soil environment will be presented. The results obtained from these computer programs will be compared with the field data to demonstrate the validity of the method of analysis. The critical aspects of the foundation behaviour in various ground movement conditions will be analysed, presented and discussed. Their implications on the overall foundation behaviour will also be discussed in detail. Finally, the conclusions of the overall work and the scope of the future work will be presented and discussed.

### 1.2 REPRESENTATION OF FOUNDATION COMPONENTS

In order to develop the method of analysis, the various components of the foundation systems are modelled in the following manner:

#### 1.2.1 REPRESENTATION OF THE RAFT

The raft is analysed as a plate resting on an elastic soil medium and is discretised into various elements and nodes. The analysis is carried out by using the finite element technique. The raft elements correspond to uniform blocks of reaction, which are used to compute the vertical deflection and moments.
1.2.2 Representation of the Soil System

The soil is assumed to be a semi-infinite, isotropic and homogenous mass. The vertical deformation of soil at the centre of each element area is obtained from classical elasticity theory in order to form the soil influence matrix. The influence of loaded areas at the centre of other element areas is found by using the method of superposition. In the case where the modulus of elasticity of the soil varies with depth, an equivalent modulus is adopted to carry out the analysis.

1.2.3 Representation of the Pile System

The piles are analysed as a number of uniformly loaded cylindrical elements together with uniformly loaded circular bases. The interaction effects between the shaft elements and the bases are considered in order to formulate the equations. Finally, solutions are obtained for the distribution of the shear stresses along the pile shafts, base pressures and the displacement of piles. In the case of more than one pile, the interaction between the elements of one pile and the other piles is also included in the analysis.

1.3 Representation of Free Field Soil Surface Profile

In the event of seasonal moisture variation, the clay soils change in volume and are subject to vertical as well as horizontal movement. Horizontal movements cause opening of surface cracks in the drier period of time whereas the vertical movements lead to cyclic changes in the soil surface levels. The presence of an impermeable surface or slab on the expansive soil alters the pattern of seasonal soil moisture change underneath the surface and causes a relative movement of the ground. In the analysis, this ground movement is incorporated as a free-field soil surface profile underneath the raft and along the pile shaft, based on a series of wetting and drying processes. The method of analysis provides the facility of incorporating any type of soil profile underneath the raft and along the pile shaft. Based on this profile, the analysis of the foundation system is undertaken by the computer programs.
1.4 SOIL-STRUCTURE INTERFACE COMPATIBILITIES

To undertake the analysis of various foundation systems, a non-linear soil-structure interactive approach has been adopted in formulating the final equations. The criteria adopted to incorporate the interactive conditions at the soil-structure interfaces are briefly described in the following sections.

1.4.1 RAFT FOUNDATION

Displacement compatibility has been considered at the raft-soil interface. In order to do so, the raft settlements are equated to the soil displacements.

The analyses have been carried out by incorporating the effect of non-linear as well as elastic conditions at the raft-soil interfaces. In the case of an elastic condition, neither lift-off between the raft and the soil nor the yielding of soil underneath the raft have been considered in the analysis. However, under non-linear conditions, a detachment between the raft and the soil surface, as well as the yielding of soil underneath the raft, has been allowed for in the analysis.

1.4.2 PILE FOUNDATION

In order to satisfy the soil-pile interface condition, it has been assumed that the soil displacement around the pile shaft shall be the same as the pile. In other words, the displacement of the surrounding soil has been equated with the displacement of the pile to incorporate displacement compatibility.

The analysis has also been extended to incorporate the effect of non-linear as well as elastic conditions. Under elastic conditions, the pile surface has been assumed to be perfectly rough and the soil as an ideal elastic material, which is capable of resisting any shear stress which may develop between the pile and the soil, i.e. no slip at the soil-pile interface has been allowed in the analysis. It has also been assumed that the bearing capacity of soil at the pile base is infinite and allows no yielding. To incorporate non-linear conditions, slip at the soil-pile interface has been considered, when the computed shear stress exceeds the limiting value. Also, yielding of the soil has been allowed if the computed stress at the pile base exceeds the bearing value of the soil.
1.4.3 Piled Raft Foundation Systems

In piled raft foundation systems, three types of soil-structure compatibilities have been taken into account in the analysis such as "soil-raft", "soil-pile" and "raft-pile". The "soil-raft" and the "soil-pile" compatibilities have been incorporated as described in sections 1.3.1 and 1.3.2, respectively. In order to incorporate the effect of the "raft-pile" connection, it has been assumed that the raft element which is connected to the pile, acts as an integral part of the raft as well as of the pile. The structural continuity of the piled raft element with the other raft elements has always been maintained in the analysis, and the interaction factors have been modified to satisfy this compatibility.

1.5 Description of Contents

The objective and the overview of the method adopted in carrying out the foundation analysis have been outlined and briefly described in this chapter 1. A broad picture of the philosophies adopted to develop the computer programs have also been outlined in this chapter.

Some of the relevant publications on raft, pile and the piled raft foundation systems have been reviewed and presented in chapter 2. The technical papers on the ground movement in a reactive soil environment have also been reviewed and discussed.

In chapter 3, the method of analysis adopted to carry out the raft foundation analysis has been described in detail. The various steps to formulate the computer program RAE,S have been presented. The modelling of the ground movement due to seasonal moisture fluctuation has also been presented and discussed. In order to demonstrate the consistency of the program output with the available literature, a comparison of work has also been carried out and presented in this chapter. A detailed analysis of the raft behaviour in a "normal" as well as a reactive ground condition has been carried out and presented by using the computer program RAE,S. The benefits of stiffened beams in reducing the adverse effect in the raft have also been analysed and briefly presented.

In chapter 4, the behaviour of pile foundations has been analysed and described in detail, including the effect of ground movements due to swelling and shrinking of soils. The method of analysis and the procedure to develop the computer program PAE,S have been described
and presented. The results obtained from the computer program have been compared with some available solutions in order to verify the accuracy of the analysis developed.

The method adopted to develop the computer program to analyse piled raft foundation systems has been described and discussed in chapter 5. The various steps in formulating the computer program as well as the methods to incorporate the interface compatibilities, are presented in detail. Some preliminary aspects of piled-raft behaviour have been analysed and compared with the available solutions to demonstrate the consistency of the computer program.

In the following chapters 6 and 7, extensive analyses of piled raft systems in shrinking as well as in swelling soils have been carried out and presented in order to understand the total behaviour of the foundation system in expansive soils.

Chapter 8 presents a comparison of results with a limited number of field and laboratory measurements to validate the methods of analysis. Chapter 9 provides a summary of conclusions of the overall findings and also suggests the potential future scope of work in this field to extend the state of knowledge and understanding of the behaviour of piled raft foundation systems.
CHAPTER 2

REVIEW OF LITERATURE

2.1 INTRODUCTION

In this chapter, some relevant literature on pile, raft and the piled raft foundation systems is reviewed in order to provide an overview of the work carried out by various authors, and the shortcomings in the current state of knowledge. Because the literature on these foundation types is vast, only a limited number of publications relating to foundation analysis and behaviour are reviewed. The development of the methods of analysis and their respective refinements to achieve further accuracy of results are presented in this context. Information is provided on the methods of analysis developed to predict the foundation behaviour in a reactive soil environment. A brief discussion is then presented on the soil models adopted by various authors to simulate the effect of wetting and drying cycles caused by seasonal moisture variations. Some literature on field results is also reviewed in order to indicate some of the practical aspects of the foundation behaviour.

2.2 RAFT FOUNDATIONS

2.2.1 RAFT IN NORMAL SOIL CONDITION

The analysis of raft foundations has long been of interest to civil engineers, as rafts are commonly used to support buildings, storage tanks, heavy machinery and bulk stored materials. The behaviour of raft foundations is dependent on the raft itself as well as on the supporting soil system.

The analysis of the raft may be carried out by using the finite element method. The displacements and the moments at various points of the raft may be accurately determined by applying the contact stresses underneath the raft, which are in turn obtained by using a suitable soil model. In order to obtain an accurate interaction analysis, many different methods have been developed, which range from one dimensional elastic models to a full three dimensional analysis. However, the deflection behaviour of the foundation system can often be adequately described by linear elastic theory, provided the appropriate elastic parameters are selected. The simplest way to account for the deformation of the soil is to assume that the pressure at any point on the soil surface is proportional to the deformation of the soil at that point. This results in the Winkler model (1867), which in effect treats the soil
as a series of isolated springs. This model assumes that no interaction exists between adjacent points of the foundation, and therefore does not treat the supporting medium as a true continuum.

Cheung and Zienkiewicz (1965) carried out an early raft analysis by using the finite element technique. The analyses were undertaken by modelling the soil by Winkler's approach as well as by the elastic continuum theory. It was assumed that the contact stresses were equivalent to forces, which could be applied at nodal points of the finite element mesh and that no separation occurs when negative stresses underneath the raft are present. The analysis indicated that the Winkler's model can lead to erroneous results and there is a danger of seriously mis-predicting foundation behaviour by using Winkler' model indiscriminately in carrying out the analysis. It can be seen from Figure (2.1), which is reproduced from this paper, that the Winkler’s foundation obtains the contact stress underneath the raft as uniform under all cases of raft stiffnesses, whereas the stresses obtained from the elastic continuum theory vary dramatically with the raft stiffnesses. In case of a very flexible footing, the stresses obtained from the Winkler’s model may be acceptable, but for a rigid raft condition, the values are totally misleading and erroneous. Therefore it was recommended that the Winkler type spring approximation should no longer be used where continuous foundations are to be analysed.

The finite element method was further extended by Cheung and Nag (1968) by incorporating the horizontal contact stresses into the analysis as well as investigating the effects of separation (lift-off) between the plate and the supporting medium. Svec (1974) also carried out a similar analysis by using iterative solution techniques for the problem of separation of the plate from the soil. The separation was introduced by setting the negative stresses to zero.

The approach of Cheung and Zienkiewicz was extended by Fraser and Wardle (1974, 76) to a multi-layered soil system and applied to a raft on a layered cross-anisotropic soil in which the raft was represented by conventional finite elements while the surface element stiffness matrix of the soil was derived from the surface settlements due to uniformly loaded rectangular areas. They derived a series of useful design charts based on parametric studies.

The effect of edge bearing failure on strip, circular and rectangular rafts has been considered by Schultz (1961), Brown (1968) and Hain (1975) respectively. Their methods take account of local yielding of the soil, which may occur due to high stresses under the edge of the rafts, and the subsequent redistribution of load to unyielded areas.
2.2.2 RAFT ON EXPANSIVE SOILS

The behaviour of raft foundations on expansive clay is complicated by the fact that a soil surface which was flat at the time of construction may develop a vertical profile with time and consequent loss of soil support can induce undesirable settlements and moments in the raft. The raft slab may act as an impermeable barrier and may alter the pattern of seasonal soil moisture changes as it impedes the direct evaporation of the moisture from the soil system. The soil surface profiles underneath the raft slab are influenced by the various stages of seasonal wetting and drying cycles, as shown in Figure (3.3 B) of Chapter 3. It can be seen that the ground surface, which is flat at the time of laying the foundation slab, develops vertical surface movements due to the change in the moisture content. As a result of this, lightly loaded residential slabs experience a relative movement of the ground support underneath it. The slabs, without adequate stiffening beams, may be subject to excessive differential settlement and, at times may experience failure of the foundation.

Lytton (1970) carried out an analysis of beams or slabs including the effect of relative ground movement underneath. In order to model the ground movement underneath the raft slab, the soil profile was assumed to be "mound shaped" and was represented by a polynomial equation as below:

\[ y = c x^m \]  

\[ (2.1) \]

where

- \( y \) = the distance below the highest point of the mound
- \( x \) = distance along the beam
- \( m \) = an integer exponent (also known as mound index)
- \( c \) = a constant.

Lytton indicated that the \( m \) value has significant impact on the foundation behaviour. A quadratic soil profile, which provides more support for the beam, results in lower moments compared to parabolic profile. The smaller the mound index \( (m) \), the larger the maximum bending moment and differential settlement.

The development of a soil profile underneath a raft slab with the seasonal environmental fluctuation is a quite complex phenomenon on the surface as well as with depth. The expansive nature of the clay soils, and the prevailing climatic conditions, lead to the development of significant seasonal soil surface movements in the top few metres. In the
event of seasonal moisture variation, clay soils change in volume and are subject to vertical as well as horizontal movements. The horizontal movement causes opening of surface cracks in drier periods, whereas the vertical movements lead to cyclic changes in the soil surface level. The magnitude of these movements decreases with depth down to a level below which no volume change occurs. The presence of an impermeable surface or a slab on the expansive soil will alter the pattern of seasonal moisture change. The slab acts as a barrier to the process of direct drying and wetting.

Holland (1981) carried out numerous field tests and observed that if the site is very dry when a completely flexible slab is placed on it, the edge wetting and heave of the underlying clay will lead initially to the development of an edge heave, or dishing slab distortion mode. Theoretically, with time the heave under the slab will slowly progress inwards, until ultimately a centre heave or mound distortion mode will form under the slab. The "idealised mound development" is shown in Figure (3.3B) of Chapter 3. There is much observational and research evidence, however, to suggest that many housing slabs may never fully develop a centre heave distortion mode. The soil surrounding and under the edges of the slab will move up and down with the season's wetting and drying processes respectively, so leading to flexing of the edges over a distance commonly refers to as the edge distance e. However, if the slab is sufficiently rigid it will not flex and distress of the superstructure will not occur. The economic design of an actual house slab consists of making it sufficiently stiff, so that any slab deflection during mound development or due to seasonal wetting and drying under its edges will not lead to distortion of the house or superstructure. If the slab is placed on a wet site, a mound will effectively already exist, so only seasonal wetting and drying of the clay under the slab edges will need to be accommodated by the slab. In order to understand the actual field behaviour of rafts on expansive soils, a series of experiential slabs were constructed in the city of Melbourne, Victoria by Holland (1981). These slabs were extensively instrumented and their deformations were monitored for a significant period of time. In order to define the potential mound formation underneath the raft, the edge distance e and the differential heave $\gamma_m$ were monitored. The edge distance and the differential heave are shown in Figure (3.3B). It was found that a rough relation exists between the differential heave and the edge distance as indicated in Table 2.1, which is obtained from Holland (1981).
TABLE 2.1

(Likely relationship between edge distance and differential mound heave)

<table>
<thead>
<tr>
<th>Edge Distance (e) * (metres)</th>
<th>Differential Heave ((y_m)) * (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 to 0.5</td>
<td>0 to 20</td>
</tr>
<tr>
<td>0.5 to 1.0</td>
<td>10 to 40</td>
</tr>
<tr>
<td>1.0 to 1.5</td>
<td>20 to 50</td>
</tr>
<tr>
<td>1.5 to 2.0</td>
<td>30 to 60</td>
</tr>
<tr>
<td>2.0 to 2.5</td>
<td>50 to 90</td>
</tr>
<tr>
<td>2.5 to 3.0</td>
<td>70 to 120</td>
</tr>
</tbody>
</table>

* : Edge distance (e) and Differential heave (\(y_m\)) indicated in Figure (3.3 B) of Chapter 3.

Richards and Chan (1971) suggested that if the soil is very wet when a cover is placed, moisture will move laterally from the centre and eventually form a long term stable edge heave condition. The formation of a long-term mound after the initial development of an edge heave condition has been recorded by a number of researchers (Ward (1953), De Bruijn (1965 and 1973) and Johnson & Desai (1975)). The development of a stable long-term edge heave condition at an initially very wet site has however not been reported.

Holland and Richards (1984) presented a brief outline of the most common approaches to the design of foundations for high structures on expansive clays. The authors attempted to outline a realistic design approach which considers the clay properties, the footing cost and environmental factors associated with footing performances. They also mentioned that there is an unfortunate tendency on the part of structural engineers to approach foundations on expansive clays in the same manner as they would a beam in a multi-storeyed building. This approach is encouraged by many soils consultants who provide in their reports many meaningless soil parameters which are of dubious value and doubtful accuracy. Economic and hence realistic design of foundations on expansive soils needs an engineer with a good understanding of both the structural engineering and geomechanics. It was also mentioned that "the behaviour of a footing on expansive clays is not as predictable as structural engineers might assume and hence the rigid application of design models and exacting computation is of little value". In the author's opinion, to achieve an effective foundation solution, it is necessary for the structural engineers to understand the geotechnical phenomenon quite well. In the same manner, the geotechnical engineers should also be conversant with the superstructural aspects to incorporate the "soil-structure interaction" in totality.

Mitchell (1984) presented a method to design shallow footings on expansive soils, based on a simplification of actual conditions encountered in practice. A parametric study was carried out in this paper, which revealed that each of the design parameters (i.e. structure geometry,
loads, soil movement, permissible deflection, shape of initial soil surface and swelling stiffness) have a variable effect on the bending moment in the footing. It was concluded that "the behaviour of a heavily loaded residential raft footing undergoing severe distortion is shown to be successfully modelled by using the analysis. The results indicated that, although the method involves a simplification of the actual field conditions, a significant loss of accuracy does not necessarily occur." The mode of distortion due to the soil-structure interaction can be seen in Figure (2.2), which is reproduced from this paper.

Mitchell (1984 a) examined the behaviour of twenty five raft designs involving forty four buildings that had suffered the damage due to the relative ground movement. The capacity of each raft was related to the type of the building and the existing soil condition. It was found that the central heave bending moment is strongly influenced by the magnitude of the building load, and centre heave appears to be more associated with the heavier loaded structures. For lightly loaded structures, critical design appears to be associated with the edge heave case, at least for South Australia. The design charts for central and edge heave conditions were also formulated on the basis of these observations (see Figure 2.3).

Cameron (1989) reported the findings of a comprehensive investigation in which various forms of reactivity tests were compared. Estimates of the ground movements were also made and reviewed in light of the seasonal soil movements observed at 12 different sites. A number of refinements in the application of the model and more general conclusions were suggested by this study of relatively shallow seasonal ground movements.

Lee and Wray (1992) carried out laboratory soil suction measurements on Lubbock sandy clay. The total suction of the soil sample was measured by thermocouple psychrometer and non-contact filter paper test, while the matrix suction was evaluated by Agwa-II sensor, fibreglass moisture cell, gypsum block, and pressure plate apparatus. In the studied range of soil suction, reasonable agreement among the soil suction measurement results made by the selected instruments were demonstrated. Wray (1992) used a method based on soil suction theory to predict shrink/swell occurring beneath slab-on-ground structures. These predictions were compared with field measurements of actual shrink/swell at a dry climate test site and a relatively wet climate test site. The method was shown to be able to accurately reflect actual movements. This method can effectively be used in determining the potential movement of the ground underneath the raft in order to analyse the behaviour a raft in expansive soils.

Barthur, Jaksa, and Mitchell (1996) derived a probabilistic design approach, based on data derived from many built and tested footings. This approach has a distinct advantage over the traditional deterministic methods in that it enables the level of risk associated with each individual design to be quantified. This approach also enables the client to be informed regarding the desired level of risk involved against the economic cost, which is likely to reduce the possibility of future litigation problems.
Li, Cameron and Mills (1996) presented a moisture flow-soil-footing interaction model for the analysis of a stiffened raft resting on expansive soils, which is subject to seasonal surface suction variations at the edge of the slab. The transient flow of moisture and the deformation of soil had been simulated using a thermo-mechanical analogy. The paper presented the usefulness of a thermal analogy of moisture diffusion and soil deformation process in understanding the patterns of movements of expansive clay soil. It could provide the depth of seasonal moisture variation, the edge distance parameter, the suction distribution within the foundation at any time for the given boundary conditions and the soil free mound shape as a function of time. In author’s opinion, this type of thermo-mechanical model shall be very useful in determining the free field soil profile underneath the raft slab. It is necessary to undertake further research to calibrate this numerical model with the field data.

2.3 PILE FOUNDATIONS

2.3.1 Piles in Normal Ground Condition

Elastic theory is widely used to analyse pile foundations. In a common application of this method, the piles are divided into a number of elements, and the displacement due to an internal point load in an elastic half-space and the stress relationships between the elements are determined by using Mindlin’s solutions. The compatibility of displacements at the soil-pile interface is enforced and the respective equations are solved to determine the unknown pile displacements and interface stresses.

Poulos and Davis (1968) described the analysis of the settlement of a single incompressible pile and presented a series of results for a wide range of situations. The shear stress on a pile element represented as a vertical stress uniformly distributed around the outer circumference of the element. Previous investigators represented the shear stress as a vertical point load acting on the axis of the element. The effects of changes in pile length, depth of soil, Poisson’s ratio and enlarged pile bases were discussed in Poulos and Davis (1968), whose analysis also included the effect of slip at the soil-pile interface as well as the settlement due to consolidation. This work has been extended to study many other problems involving piles. The settlement behaviour of a group of rigid piles was considered by Poulos (1968). Mattes and Poulos (1969) presented solutions for the settlement analysis of single compressible piles. The settlements and the load distributions in floating and end bearing groups of compressible piles were presented by Poulos and Mattes (1971). Theoretical solutions for piles and pile groups were presented and compared with results from published field tests in clay by Mattes (1971). Some of these results were also discussed by Mattes and Poulos (1971) and Poulos (1974). These results show that many aspects of the settlement behaviour of piles and pile groups in clay can be satisfactorily predicted by elastic theory.
Butterfield and Banerjee (1971) presented comprehensive solutions for the behaviour of single compressible piles and pile groups. They investigated the approximation made by Poulos and Davis (1968) and Mattes and Poulos (1968) concerning the pile base and the presence of piles in the elastic half-space. Their more rigorous results agreed with those of Poulos and Davis (1968) to within 5% when the pile length was greater than 5 pile diameters.

In order to determine the influence of the stress around the pile element to a point in the soil medium, Poulos (1980) divided the soil cylinder around the pile at different angle $\theta$ and at different depths $z$ (see Figure 4.1, Chapter 4). The unit loads were applied at these locations and the influence of these loads at point $i$ was determined. This was defined as the interaction factor for the point $i$ due to the stresses generated around the pile element. This method involved extensive analytical and numerical integration along and around the pile associated with the determination of discrete nodal flexibility coefficients from continuously distributed shears. Although, this is a reasonably and accurate method, it is very time consuming and makes a direct solution of the whole group difficult. Sharnouby and Novak (1984) simplified the above method by the application of discrete point loads applied and located in such a manner that the resultant flexibility coefficients are almost the same as those obtained from the continuously distributed shears. The investigation indicated that the average displacement can be obtained with adequate accuracy from two equal point loads, which act on the cylinder surface at a distance of $L_i/4$ from the soil element ends, where $L_i$ = length of element, and which are related to the reference node O by angles $\theta = 80^\circ$ or $\alpha = 40^\circ$ as indicated in Figure (4.2) of Chapter 4. Thus the soil flexibility coefficient $l_{ij}$ is equal to the average of the vertical displacement at point $i$ due to two equivalent unit loads acting on element $j$ as indicated in Figure (4.2). This definition holds good for the elements of one pile as well as for the elements of different piles. A similar simplification was also suggested by Sharnouby and Novak (1984) for the pile bases. The distinct advantage of this equivalent point load method is the considerable reduction in computing time. The above approach may be used to establish the interaction factors for the elements of one pile as well as for elements of different piles.

Finite element methods have also been used to analyse piled foundation, often very effectively. Ellison, D’Appolonia and Thiers (1971) developed an axisymmetric finite element computer programme and analysed the load-settlement behaviour of bored piers in stiff London clay. A trilinear approximation of the stress-strain curve for clay was used in the finite element model. The theoretical and field behaviour of 5 piles tested by Whitaker and Cooke (1966) was studied in detail. The analysis indicated that adhesion failure, i.e. slip, occurs at the pile-soil interface before soil failure. However, slip can lead to a weakening of the soil near the pile shaft. A tension crack may develop in the soil near the pile base at high loads and this was modelled in the finite element analysis by spring elements of finite strength in the soil. Axisymmetric finite elements have been used in numerous other studies including
piles in sand by Desai (1974), and the effects of installation on load-settlement behaviour of piles by Balaam, Poulos and Booker (1975).

2.3.2 PILES IN EXPANSIVE SOILS

Pile foundations in expansive clays are frequently subjected to severe movements arising from moisture changes within the clay with consequent cracking and damage due to distortion. Piles have been used extensively for foundations in swelling soils in order to anchor the structure down at a depth where changes in moisture content are negligible, so that the movements of the structure are minimised. However considerable uplift forces may then be induced in such piles due to the action of the swelling soils.

Existing methods of analysis of piles in swelling and shrinking soils are generally confined to the estimation of forces induced in a pile by soil movement. Typical of such approaches are those described by Collins (1953), Mariotti and Khalid (1969) and Bozozuk (1972), which in general assume that full slip occurs between pile and soil along the shaft. Sahzin (1968) obtained expressions for the movement of a pile in swelling soil by considering the work done by frictional forces in the upper portion of the pile tending to lift the pile and the applied load and the frictional forces in the lower half of the pile tending to resist uplift. A more satisfactory analysis can be carried out by employing elastic theory in a similar fashion to that described by Poulos and Davis (1968) for pile settlements and Poulos and Mattes (1969) for negative friction on end bearing piles. Poulos (1973) proposed a method of analysis to estimate the movement of the pile in swelling soils. The analysis is based on the simplifying assumption that the soil can be treated as an elastic material but modifications to the analysis are described which allow consideration of such factors as non-homogeneity, slip at the soil-pile interface and crushing or tensile failure of the pile. Poulos (1990) summarised and assessed some of the proposed methods for pile foundation design to withstand negative friction. A load transfer analysis was used to evaluate the performance of a pile at different working loads, when negative friction is developed along the pile. The behaviour of a pile in a settling soil situation is shown in Figure (2.4). It was indicated that the negative friction itself will not cause geotechnical failure, but may cause excessive movement of the pile if the working load is too large. Thus the design of a pile subjected to negative friction is generally governed by settlement, rather than ultimate load considerations.

In South Africa, a thermal power station was to be constructed near the town of Vereeniging on the border between the Transvaal and the Orange Free State. Because of the vulnerability of power station installations to differential movement, virtually all plant and building foundations for the power station were to be piled, using bored cast in situ piles (Blight, 1984). Design for heavy loads was not a problem as the piles could be socketed into the carbonaceous shale at the base of the profile. However problems did arise with the design of lightly loaded piles and piles that would be installed two to three years prior to receiving.

_Piled Raft Foundation Systems in Expansive Soils_
their full dead load. As the desiccation in the profile was relieved and heave occurred, piles would become subject to frictional uplift and therefore needed to be designed as tension anchor piles. In order to assess the situation, Blight (1984) carried out a field test with a group of seven piles subjecting them to accelerated heave of the surrounding soil. Three piles out of the seven pile group were instrumented and monitored to observe their behaviour. Each of the three instrumented piles contained eight strain cells. The test group also included a multi-point extensometer to enable the distribution of heave in the profile to be monitored. These three instrumented piles represented a corner pile, a side pile and an interior pile of a typical pile group. The tests indicated that where a pile is likely to be subjected to heave prior to loading, it should be designed to withstand the full effects of uplift, as the subsequent load application will have only a minor effect on the tension in the pile. This case has been analysed and presented in section 8.3.1 of Chapter 8 in greater detail.

Bhandari et al (1987) carried out field tests with piles of uniform diameter, and single-bulb and double-bulb underreamed piles in a highly expansive black soil deposit subject to swelling. The tests were carried out with two piles of uniform diameter of 300 mm and 3.5 m long. The details of the pile testing is outlined in section 8.3.1.4 of Chapter 8. The uplift forces induced along the pile shaft were monitored using vibrating wire load cell assemblies placed at appropriate elevations within the piles. The vertical displacements of the piles were monitored using water level gauges and were found to be in the order of 25 to 40 percent of the ground surface heave. The tests indicated that the trend of uplift behaviour of piles was in good agreement with the results of elastic analysis proposed by Poulos and Davis (1973). The maximum uplift forces and uplift displacement were calculated for $E_s = 5,000 \text{kPa}$ (constant with depth), pile shaft adhesion = 37.5 kPa and $S_p$ = 60 mm by using the procedure outlined by Poulos and Davis (1980). The predicted and the observed values for the maximum uplift load were 68 kN and 46 kN respectively. Similarly the predicted and the observed values for the uplift displacements were 24 mm and 16 & 18 mm respectively. The difference between the observed and predicted values was partly due to the values of various parameters selected for the calculations and partly due to the departure from the actual conditions. In geotechnical engineering, a model or procedure may not produce accurate results, if appropriate soil data are not adopted in the analysis.

2.4 PILED RAFT FOUNDATIONS

In recent years, piled raft foundation systems have been increasingly used to support large structures. In the majority of cases, the piles are designed to withstand the whole superimposed load, i.e. the load carried by the raft is totally neglected. However, in the
actual situation, the raft also shares part of the load. Therefore, the design of a piled raft foundation system may be optimised by incorporating the load bearing features of the raft also. The load sharing between the piles and the raft may be influenced significantly by ground movement, caused by the process of swelling or shrinking of soils. In a reactive soil environment, the soil surface under the raft and along the pile shaft changes its level with the seasonal moisture variation. As a result of this, the foundations are subjected to a different state of stress, which may adversely affect the foundation performance. The literature on piled raft foundation systems in normal ground conditions as well as in expansive soils has been reviewed and is discussed below in order to outline the work carried out by various authors and to set the base for the work described in this thesis.

2.4.1 Piled Raft Systems in Normal Ground Condition

In subgrade reaction theory the soil may be considered as a set of isolated springs, and the stiffness of the springs is the modulus of subgrade reaction of the soil. To analyse piled raft foundation systems with this model, different moduli were assigned to the pile and the surrounding soil. Hansbo, Hofmann and Mosesson (1973) and Torstensson (1973) designed piled raft systems for complicated foundations in Sweden by this method. In both these papers, data was presented on the performance of the actual foundation and the foundation model. The effects of different pile spacings, the load transfer between pile cap and piles, and the reduction in heave during excavation caused by the installation of piles, were discussed. Although subgrade reaction theory provides distributions of settlement and bending moment in the raft, these quantities are sensitive to the values of subgrade modulus assigned to the soil and the piles. In this approach, no account was taken for the interaction between various foundation components.

Linear elastic continuum solutions overcome many shortcomings of the Winkler soil model. In this approach, the effect of interaction is taken into account in the analysis and a load not only produces displacement at its point of application but also throughout the continuum.

Poulos (1968) carried out an elastic analysis of the influence of a rigid circular pile cap on the settlement behaviour of an axially loaded incompressible pile in a semi-infinite elastic mass. It was found that the influence of the cap on the settlement was worthy of practical consideration. Davis and Poulos (1972) analysed the interaction between two such units, each consisting of a pile and cap, and used superposition, as suggested by Poulos (1968) for pile groups, to determine the behaviour of general piled raft systems. The analysis was also extended to incorporate the effects of slip at the soil-pile interface as well as bearing capacity failure of the cap. This indicates that the addition of relatively few long piles could
be effective in reducing the raft settlement, even though the piles may have reached their ultimate load. A method for the selection of the number of piles, on a settlement basis, rather than ultimate load basis, was presented in an example.

Butterfield and Banerjee (1971) analysed rigid rectangular caps with groups of compressible piles, using elastic half-space theory to determine interaction relationships. The analysis indicated that the settlement of the rigid cap was reduced by 5 to 15 % depending on the group size and pile spacing. The comparison of the settlement ratios of the cap bearing pile groups with free standing pile groups can be seen in Figure (2.5), which is reproduced from this paper. The behaviour of free standing pile groups (i.e. piled raft systems without any contact between the raft and the soil surface) have also been analysed and presented in this thesis. It should be noted that in a reactive soil environment, the ground movement may have a significant impact on these settlement ratios.

Hongladaromp, Chen and Lee (1973) studied the behaviour of a rectangular footing resting on piles, using the finite difference method to analyse bending in the raft. The piles were treated as independent elastic springs while the subgrade was considered as an elastic half-space. This approach ignores interaction effects due to the presence of the piles under the raft and so gives smaller settlements and different behaviour than if interaction was fully considered. Hain (1975) presented a method for analysing rafts on piles and an elastic continuum. Interaction effects of the piles were considered by using the displacements of the soil surface due to the presence of a loaded pile, as determined by Poulos and Mattes (1971). A finite element plate bending formulation was used to determine the stiffness matrix for the raft, and superposition was employed to incorporate appropriate pile-soil and pile-pile settlement interaction effects into the matrix for the soil and pile system. The effect of pile failure was also considered by Hain (1975) and settlement and moment results presented for a uniformly loaded square raft on 36 piles.

Hain and Lee (1978) extended the work of Hain, and developed a method of analysis to predict the behaviour of piled raft foundation systems. The analysis considered the raft as a flexible elastic plate supported on a group of compressible friction piles, and the supporting soil was represented as an elastic homogenous or non-homogenous material. The ultimate load capacity of the piles was taken into account by a load cut-off procedure. The method of limiting the pile capacity influences the analysis to a certain extent, which is discussed in section 5.4.1.1.2 of Chapter 5. In this paper, the analysis was carried out to establish the influence of the piles in reducing settlement of raft. It can be seen from Figure (2.6) that the \( S/D \) (spacing - diameter ratio) and \( L_p/D \) (length-diameter ratio) have a significant impact on the percentage load taken by piles for rigid and flexible raft conditions. The results were also compared with the field measurements. The analysis incorporated the interaction between
the pile to pile, surface to pile, pile to surface and surface to surface. The raft was analysed by using the finite element method. In the analysis, the connection between the raft and each pile was assumed to be sliding ball joint, because neither lateral forces nor moments were transmitted and no moment transfer between raft and pile head was considered.

Hooper (1979) in CIRIA (Construction Industry Research and Information Association) Report 83, presented a review of pile groups subjected to vertical applied loading, with particular reference to piled raft foundations in cohesive soils. Various methods of pile group analysis and design were discussed, and reference was made to experimental results obtained from model studies and field investigations. It was pointed out that the sophisticated level of the more rigorous methods of pile group analysis is not matched by an appreciable quantity of good experimental data. This was partly remedied by the recent testing of model pile groups, but there is a conspicuous lack of reliable field data obtained from full-scale structures, especially in cases where piles were installed only to reduce settlements. It was recommended that, with reference to future field research, the difficulties in instrumentation should be resolved and, further attempts should be made to monitor the more complex piled raft foundations.

Cooke (1986) presented the results of model tests on unpiled rafts, free-standing piles and piled rafts of various sizes. The study suggested that at the same overall safety factor the settlement of structures on piled rafts are likely to be comparable with the settlements of structures on unpiled rafts of similar sizes. Since the piled raft foundations are frequently employed because the settlements of unpiled rafts are considered to be excessive, the small settlements that structures on these foundations normally experience are probably due to the fact that the safety factors in practice are higher than those assumed in the design. In this context, the author would like to point out that the differential settlement is greatly influenced by the location of the piles beneath the raft. Although a group of piles beneath the raft may have a certain bearing capacity, their location may affect the total and differential settlement quite significantly. Therefore, introducing piles underneath the raft may increase the overall capacity of the foundation system, but may not be effective in reducing the differential settlement, if not situated at the appropriate location.

Kuwabara (1989) analysed piled raft systems by using a boundary element analysis based on elastic theory. The characteristics of settlement and load transfer for piled raft foundations whose raft rests on a homogeneous, isotropic elastic half-space soil were contrasted with free-standing pile groups and single piles. Griffiths, Clancy and Randolph (1991) developed a program to carry out the piled raft analysis. The piles were modelled as rod elements and the raft as thin plate elements. The ground resistance at each node was represented by non-
linear t-z springs (see Figure 2.7). The three kinds of interactions, namely "pile-soil-pile", "pile-soil-raft" and "raft-soil-raft" were considered by using Mindlin's elastic theory.

Poulos (1991) discussed the following methods of analysis adopted by various authors:


2. Modified analyses of a plate on a soil mass, in which the piles are represented by springs (eg. Hongladaromp et al, 1973; Brown et al, 1975)

3. Boundary element analysis in which the raft and the piles are discretised and interaction between them is considered by use of appropriate solutions for the theory of elasticity (Authors: Brown and Wiesner, 1975; Wiesner and Brown, 1976)

4. Finite element analyses, in which the raft and pile group are represented by a stiffer block within the soil mass (Hooper, 1979);

5. Finite element analyses in which the raft modelled as a series of plate elements and the pile are modelled either as discrete piles (Hain and Lee, 1978) or as a series of equivalent concentric ring (eg. Hooper, 1973; Naylor and Hooper, 1974)

Poulos (1991) developed a method to analyse piled strip foundation systems which falls between the above categories (2) and (3). The method of analysis was implemented via a computer program GASP (Geotechnical Analysis of a Strip with Piles). In this analysis, the strip was modelled as a beam whose stiffness may vary along its length. The piles were modelled as springs whose stiffness was either input or computed from elastic theory, and whose ultimate capacity in compression and tension was specified. The limiting values of strip-soil contact pressure were specified for both compression and tension to allow for the bearing failure or lift off of the strip from the soil. At a later stage, GARP (Geotechnical Analysis of Raft with Piles) was also developed to analyse the piled raft foundation system. In this paper the influence of the number of piles on differential and total settlement was addressed (see Figure 2.8). It was indicated that beyond a limit, further increase in the number of piles may not have any effect on the settlement (see Figure 2.9).

Sommer et al (1991) described the foundation design, construction and the monitoring system of the 256 m high MesseTurm, Europe's tallest high-rise building. The effect of a 12 m groundwater lowering on the behaviour of the foundation was also presented. The influence of the construction sequences on the foundation behaviour was analysed and is discussed in detail in Chapter 8 of this thesis.
Randolph and Clancy (1994) presented a detailed numerical analysis of the foundation system and compared with the measured performance. In this paper, the principle of locating piles towards the centre of a pile group, in order to minimise the differential settlement was presented with the piles omitted from beneath the outer foundation units. This analysis indicated a marginal increase in the total settlement, but significantly lower differential settlement, than the fully piled design, for approximately half the number of piles. The strategy of introducing piles as settlement reducers was recommended to achieve an economic design for the piled raft systems in the future.

Various finite element analyses have been performed on piled raft problems where the soil was modelled by finite elements. The analyses were carried out by various authors such as Hooper (1973), Desai et al. (1974) and Zhuang et al. (1991) at various stages of development. Hooper (1973) compared the observed behaviour of a piled raft foundation in London clay with an axisymmetric finite element model. A concentric ring of piles in the foundation was represented by a continuous annulus in the finite element study with an overall stiffness equal to the sum of the stiffnesses of the individual piles. Good agreement was obtained between the computed and measured values of load and displacement. These were recorded over a six year period. The analysis demonstrated the influence of various factors which contributed to the observed behaviour of the raft such as:

- the depth and method of construction of the basement
- the variation of soil deformation modulus with depth and
- the contribution of the superstructure to the overall bending stiffness of the foundation.

Desai et al. (1974) performed a two-dimensional finite element analysis to predict the behaviour of a pile-supported gravity lock. Rows of piles were represented by a continuous strip in a plane strain analysis. The various sequences of dewatering, excavation, pile installation, backfilling and other aspects of construction were simulated to predict settlements and distribution of loads among the rows of battered piles. Certain analytical refinements used in this paper, and the use of a hyperbolic stress-strain relationship for the soil, demonstrate the flexibility of the finite element method in piled raft problems. Pichumani, Crawford and Triandafilidis (1974) used solid three-dimensional finite elements to analyse the behaviour of rigid and flexible pavements supported on piles. The influence of pile stiffness on the vertical settlements and stresses under a single wheel load was investigated. Ottaviani (1975) also used solid three-dimensional elements to study the behaviour of two pile groups with a contacting pile cap. The efficiency and accuracy of this approach of piled footing analysis was evaluated by Poulos, Brown and Wiesner (1975) and Poulos (1976), in discussions of Pichumani et al. (1974) and Ottaviani (1975) respectively. These discussions supported the conclusion of Wardle and Fraser (1974) that three dimensional finite element analyses were very expensive in terms of computing time and of
dubious accuracy. They should only be adopted when solutions based on elastic half-space theory are clearly inappropriate.

Zhuang, Lee and Zhao (1991) presented a simple practical method for estimating the apportionment of the total load taken by the piles for pile-raft and pile-box foundations, based on the analytical results obtained by 3-D FEM. The validity of the method was also compared with field measurements. Their study also indicated that the settlement of a raft-pile system can be reduced within limitations by increasing the pile stiffness and length. The differential settlement can be reduced significantly by employing piles in a raft foundation. For a rigid raft (Relative stiffness of raft $K_R > 1$), the effect of piles on the differential settlement is slight. The bending moment is significantly affected by the variation of $K_R$, ranging from 0.01 to 1.0. The apportionment of total load taken by piles increases with the increasing of $K_p$ and $L_p/D$ and decreases with the increasing of $K_R$ and $S_p/D$. For a common range of $K_R$, $K_p$, $L_p/D$ and $S_p/D$, it was concluded that the piles can carry about 30% - 90% of the total load.

2.4.2 PILED RAFT SYSTEMS IN EXPANSIVE SOILS

It is obvious that a piled raft system constructed in a reactive soil environment will be subjected to a relative movement of the ground with the variation in the seasonal moisture content. The piles are influenced due to the relative ground movement along the shaft, whereas the raft is influenced due to the relative ground support condition underneath it. In piled raft systems, this behaviour is rather more complex, because of the complicated interaction between various foundation components. The ground movement induces an upward or downward force in the pile as well as changing the state of stress under the raft and, due to their structural connection, these forces are redistributed between the various parts of the foundation, depending on their relative stiffnesses. These features influence the foundation behaviour quite significantly and may cause an adverse effect on the foundation performance. Therefore, it is imperative to undertake a thorough investigation to achieve an effective foundation solution in these situations under various loadings. Until now, very little work has been carried out to provide a better understanding of this behaviour. The majority of the developments on piled raft foundation systems have been undertaken in a "normal ground" condition. However, it is also very necessary to consider the impact of the reactive nature of the soil system on the foundation behaviour, as neglect of the reactive characteristics of the soil systems may lead to a dangerous consequence on the foundation behaviour.
Poulos (1991), via the GASP program mentioned previously presented a simplified method for analysis for the behaviour of a piled strip foundation subjected to concentrated vertical load and moment loading, distributed loading and externally-imposed soil movements. A boundary element formulation was used in which the strip was modelled as a simple beam and the piles were modelled as spring elements. The interaction through the soil among the piles and between the strip and the piles is considered by treating the soil as an elastic continuum. In the analysis, the displacement was expressed as below:

\[ (p) = (B/E_{pr})[I](p) + (S_o) \]  \hspace{1cm} (2.2)

where

\( (p) \) = Soil movement vector  \
\( B \) = Width of strip  \
\( (p) \) = Vector of contact pressures  \
\( E_{pr} \) = A reference value of soil modulus  \
\( (S_o) \) = Vector of free-field soil movements at the nodal points (Downward movement is adopted as positive)  \
\( [I] \) = \((n+1)\) square matrix of soil displacement influence factors

Poulos (1993) analysed the piled raft foundation systems in consolidating soils and provided some useful insights into the piled raft behaviour. It was mentioned that "there has been very little or no consideration has been given to the possible effects of externally imposed soil movements on the performance of the piled raft foundation systems". The paper considered the downdrag force caused by the consolidating soils as well as upward induced force along the pile shaft due to the swelling soils. It was mentioned that "in circumstances where external vertical soil movements are likely to develop, the use of piled raft foundation is best avoided".

In the author’s opinion, the performance of piles, rafts and the piled raft foundation systems in a reactive soil environment is greatly dependent on the intensity of imposed load. The imposed load generally causes a counteracting effect against the swelling soils. The behaviour of the piled raft systems in these situations is very complex and is dependent on the interaction between various parameters. Therefore, a thorough analysis should be undertaken with the various conditions to which the foundation will be subjected. The integrity of the foundation system should be checked and verified under these critical conditions. These aspects have been analysed and presented in this thesis and the critical conditions, which are to be avoided to achieve an effective foundation system, have also been discussed.

Piled Raft Foundation Systems in Expansive Soils

2-23
2.5 SHORTCOMINGS OF AVAILABLE METHODS

The above review of the literature describes the development of the methods of analysis of raft, pile and piled raft foundation systems at various stages. The advancement in computer technology has enabled research engineers to undertake more sophisticated and rigorous computer analysis of the foundation system than was possible one or two decades ago.

The need for further understanding of the foundation behaviour under different conditions was envisaged by various research and practising engineers and the work was undertaken to determine establish those features of the foundation systems. The review of literature indicates that some aspects of the piled raft foundation systems require further attention in order to understand their behaviour under various conditions. These shortcomings of the present state of knowledge are discussed, outlining that how these aspects are included in the present analysis.

PILE RAFT SYSTEMS IN EXPANSIVE SOILS

This review indicates that in various stages of advancements, the issue of the ground movement was addressed quite seriously in the case of raft and pile foundation systems. Methods of analysis were developed and their accuracy were established with the field measurements. It was understood by both research and practising engineers that in order to achieve an effective foundation solution in a reactive soil environment, the forces induced by the ground movement require careful attention in analysing the foundation system and the negligence of these aspects may cause a detrimental damage to the overall integrity of the structure.

Despite the awareness of the possible adverse effects of the ground movements on the raft and the pile foundations, the behaviour of piled raft foundation systems, when subjected to the swelling and shrinking soils, is not well established. The relative movement of the ground underneath the raft and along the pile shaft may have a significant impact on the load distribution pattern between the piles and the raft. This may influence the piled raft behaviour very significantly in comparison to the flat ground condition. The piles used as settlement reducers may not be stiff enough to reduce the settlement due to the load imposed by the ground movements. The forces exerted by the relative movement of the ground may develop undesirable stresses at the "pile-raft" interface, causing an adverse affect to the structural integrity of the foundation system. Therefore, it is necessary to address these aspects, when the piled raft systems are subjected to a reactive soil environment, where the movement of the ground due to the atmospheric moisture fluctuation is a potential problem.
In this thesis, the work has been focussed to provide an understanding of piled raft foundation behaviour in expansive soil situations.

**DISCRETISING FOUNDATION COMPONENTS**

It is understood that in geotechnical numerical analysis of a foundation system, accuracy is dependent on the number of elements and their sizes. The accuracy of the analysis may be improved by discretising the whole foundation system into various elements. For example, the method of analysis which incorporates the effect of interaction between all pile elements in the system will present a better result compared to methods which consider the interaction between pile to pile as a whole. In the analysis to be presented in this thesis, the piles have been divided in various cylindrical elements and the interactions between various elements of the foundation have been considered in the analysis.

**NON-LINEAR EFFECTS**

The method of incorporating the non-linear effects in the analysis may also influence the final result very significantly. In the majority of cases reviewed, the restriction on the pile capacity is incorporated when the load on the piles reaches its ultimate capacity. In this situation, the pile behaviour is assumed to be linear until the load on it reaches the ultimate capacity. However, the stress distribution along the pile shaft is not uniform and some portions of the shaft experience higher stress than others. Therefore, slip occurs at those more highly-stressed portions of the shaft well before the ultimate capacity is reached. This results in non-linear behaviour. In order to incorporate this effect in the present analysis, slip between the soil and the pile at every element has been allowed, whenever the shear stress exceeds the limiting value.

**SOIL SURFACE MOVEMENT**

The magnitude of the soil surface movement generally reduces with depth. Therefore, it is necessary to include the magnitude of the ground movement at the respective depth along the pile shaft i.e. at various nodes of the piles. This process influences the accuracy of the foundation analysis very significantly, especially in a non-linear analysis. This procedure obtains the location of the "neutral plane" along the pile shaft as well as defining the remaining components of the induced force along the shaft accurately. This is a very important feature of the analysis of piles subjected to in ground movements.
Other aspects have also been considered in the analysis to obtain the foundation behaviour more accurately. These analytical strategies are described in their respective chapters in greater detail. The present method of analysis to be described in subsequent chapters may be regarded as a more rigorous approach than the majority of methods described in the existing literature.
Figure 2.1: Contact pressure distribution along the centreline of the plate

Figure 2.2: Soil-structure interaction and its influence on soil profile
Figure 2.3 (a): Central heave bending moment

Figure 2.3 (b): Edge heave bending moment
Figure 2.4: Influence of soil surface movement on load-settlement behaviour of a pile

Figure 2.5: Comparisons of settlement ratios of cap bearing pile groups with free standing groups

Note: Settlement ratio is the ratio of group displacement under a load of \([N \times P]\) to the single uncapped pile displacement under load \(P\)
Figure 2.6: Percentage of load taken by \( 8^2 \) pile group for a raft-pile foundation

Figure 2.7: Representation of piles and soil (Griffith, Clancy and Randolph, 1991)
Figure 2.8 Solutions for uniformly loaded square piled raft on elastic soil

Figure 2.9: Influence of number of piles on settlement and differential settlement
CHAPTER - 3

THE ANALYSIS OF RAFT FOUNDATIONS

3.1 INTRODUCTION

The behaviour of a raft foundation on soils subjected to relative ground movement due to the process of swelling or shrinking is a critical phenomenon because of its potentially adverse effect on the overall foundation performance. The soil surface, which is flat at the time of construction, develops a vertical soil surface profile with time due to the changes in the moisture content and soil suction. This alters the ground support condition underneath the slab and may induce excessive differential settlement and bending moments, which are beyond the acceptable ranges of the foundation behaviour. There are numerous cases of structural damage due to these effects.

The objective of this chapter is to present an analysis for the behaviour of a raft subjected to "central" and "edge heave" conditions caused by the process of shrinking and swelling of the supporting soil. The effects of these heave formations on the bending moment and differential settlement have been analysed and discussed. The influence of the cyclic wetting and drying processes on the raft behaviour have also been briefly analysed and discussed. The influence of stiffened beams in restricting the differential settlement due to relative ground movement have been also studied.

In order to carry out the analysis, the raft is assumed to be an elastic plate resting on a semi-infinite and homogeneous mass. The contact pressures between the raft and the soil are assumed to be uniform blocks of pressure acting in a vertical direction only. These contact pressures are reapplied to the raft to obtain the bending moment and displacements. A computer program RAE₆₆ (Raft Analysis in Expansive Soils) has been developed to carry out the analysis.

Some preliminary aspects of raft behaviour are also studied and compared with the available work in order to demonstrate the level of consistency with previous investigations carried out by various authors.

Analyses have been undertaken for "elastic" as well as "non-linear" conditions. The influence of "lift-off" and "local soil yield" is analysed and compared with the "elastic
condition”. Finally, a non-linear approach has been adopted to undertake the analysis of stresses at the “soil-raft” interface.

A comparison between the field measured data and the results obtained from the computer analyses is discussed and presented in Chapter 8.

3.2 ANALYSIS

3.2.1 RAFT ANALYSIS

The raft is analysed as a plate resting on an elastic soil medium and is discretised into the required number of elements and nodes. A four noded rectangular element is adopted to carry out the analysis. Each node consists of four degrees of freedom such as vertical displacement, rotations in $X$ and $Z$ directions and twists (Figure 3.1b ). The element was developed by Bogner et al (1965).

The vertical displacement $\omega$ in $Y$ direction can be evaluated within the element from the nodal variable $\mathbf{a}$ as

$$\omega = \begin{bmatrix} N_1(e) & N_2(e) & N_3(e) & N_4(e) \end{bmatrix} \begin{bmatrix} a_1(e) \\ a_2(e) \\ a_3(e) \\ a_4(e) \end{bmatrix}$$

$$\omega = N \mathbf{a} \quad \ldots \ldots 3.1$$

where

$$a_i^{(e)} = \begin{bmatrix} \omega_i, (\delta \omega / \delta x)_i, (\delta \omega / \delta z)_i, (\delta^2 \omega / \delta x \delta z)_i \end{bmatrix}^T$$

and

$$N_1(e) = \begin{bmatrix} n_1m_1, n_2m_1, n_1m_2, n_2m_2 \end{bmatrix}$$

$$N_2(e) = \begin{bmatrix} n_1m_3, n_2m_3, n_1m_4, n_2m_4 \end{bmatrix}$$

$$N_3(e) = \begin{bmatrix} n_3m_3, n_4m_3, n_3m_4, n_4m_4 \end{bmatrix}$$

$$N_4(e) = \begin{bmatrix} n_3m_1, n_4m_1, n_3m_2, n_4m_2 \end{bmatrix}$$
where

\[ n_1 = \frac{1}{l^3} \left( 2x^3 - 3lx^2 + l^3 \right) \]
\[ n_2 = \frac{1}{l^3} \left( 2x^3 - 3lx^2 \right) \]
\[ n_3 = \frac{1}{l^2} \left( x^3 - 2lx^2 + l^2x \right) \]
\[ n_4 = \frac{1}{l^2} \left( x^3 - lx^2 \right) \]

and similarly

\[ m_1 = \frac{1}{b^3} \left( 2z^3 - 3bz^2 + b^3 \right) \]
\[ m_2 = \frac{1}{b^3} \left( 2z^3 - 3bz^2 \right) \]
\[ m_3 = \frac{1}{b^2} \left( z^3 - 2bz^2 + b^2z \right) \]
\[ m_4 = \frac{1}{b^2} \left( z^3 - bz^2 \right) \]

The displacement \( \omega \) is differentiated to determine the strain within the element. The strain is defined as

\[
\varepsilon = \begin{bmatrix}
-\frac{\partial^2 \omega}{\partial x^2} \\
-\frac{\partial^2 \omega}{\partial x \partial z} \\
\frac{\partial^2 \omega}{\partial z^2}
\end{bmatrix}
\]

\[ \cdots \cdots 3.2 \]

The corresponding stresses, derived from the above equations are the bending and twisting moments per unit length in the \( X \) and \( Z \) directions and are expressed as

\[
\sigma = D \varepsilon = \begin{bmatrix}
M_x \\
M_z \\
M_{xz}
\end{bmatrix}
\]

\[ \cdots \cdots 3.3 \]
The plate bending matrix \( D \) may be expressed for an isotropic plate as:

\[
D = \frac{E_s t^3}{12(1 - \nu_r^2)} \begin{bmatrix}
1 & \nu_r & 0 \\
\nu_r & 1 & 0 \\
0 & 0 & \frac{1 - \nu_r}{2}
\end{bmatrix}
\] ..........3.4

where

\[
E_s = \text{Young's modulus of plate} \\
\nu_r = \text{Poisson's ratio of plate} \\
t = \text{Plate thickness}
\]

The matrix \([B]\) may be expressed as

\[
B = \begin{bmatrix}
-\frac{\partial^2 N}{\partial x^2} \\
-\frac{\partial^2 N}{\partial z^2} \\
2\frac{\partial^2 N}{\partial x \partial z}
\end{bmatrix}
\] ..........3.5

The finite element equations may be written as

\[
K \ a = F \] ..........3.6

where

\[
K = \text{Stiffness Matrix} = \int_B D B dA \\
F = \text{Force Vector} \\
a = \text{Vector of nodal displacements, rotations and twist as defined in eq. (3.1)} \\
A = \text{Area of element}
\]

The stiffness matrix for an element can be obtained by using the method of numerical integration known as Gaussian quadrature over the rectangular area of the element. The global stiffness matrix for the whole plate is formed by assembling element stiffness matrices for each element.
The force vector is formed for the imposed load (or applied load) on the raft for various load conditions to determine their respective displacements. These conditions are described in section 3.2.3.2.

The nodal variables $a$ as defined in equation (3.1) are obtained by solving equation (3.6). The moments per unit length are then calculated at the Gaussian points or at corners of the elements by using the following equation

$$\sigma = D \varepsilon = D B a$$  

......3.7

3.2.2 Soil Analysis

The soil is assumed to be a semi-infinite, isotropic and homogenous mass. The vertical deformation of soil at the centre of each element area is obtained in order to form the soil influence matrix. The influence of loaded areas at the centre of various element area is found out by using the method of superposition as described in section 3.2.3.1.

The soil is analysed for elastic as well as non-linear conditions. Under elastic condition, neither the detachment between the soil & the raft nor the yielding of soil under raft elements have been considered in the analysis, whereas in case of non-linear analysis, these effects have been incorporated. The analysis also included the effect of adhesion between the raft and the soil in case of lift-off.

3.2.3 Interaction between Soil and Raft

The base of the raft is assumed to be perfectly smooth. The interaction analysis combines the influence matrices for the soil and the raft to solve for the contact pressures as described in the following sections.

The relative stiffness of the raft with respect to soil is expressed as (Hain and Lee, 1978)

$$K_r = \frac{4 E_r t_r^3 B_r (1 - \nu_r^2)}{3 \pi E_r L_r^4}$$  

......3.8

where
\[ E_s = \text{Soil Modulus} \]
\[ L_R = \text{Length of Raft} \]
\[ B_R = \text{Width of Raft} \]
\[ \nu_s = \text{Soil Poisson’s Ratio} \]

The values of \( K_R \) less than 0.001 and greater than 1 correspond to a flexible and a rigid raft conditions, respectively. Intermediate values signify semi-rigid conditions.

### 3.2.3.1 Influence Matrix of Soil

The vertical deflections \( I_{l_1}, I_{l_2}, I_{l_3}, I_{l_4}, \ldots, I_{l_i} \) of various areas are determined due to the unit load pressure at any area \( j \). These factors are defined as the influence factors for the soil system. Therefore, for a stress \( p_j \) at \( j \), the deflection at various areas will be \( p_j \) times of the above influence factors, because of the assumption of a “linear elastic soil system”.

The deflection at the corner of a rectangular loaded area may be expressed as

\[ \rho = \frac{p b}{E_s} (1 - \nu_s^2) \left( A_s - \frac{1 - 2\nu_s}{1 - \nu_s} B_s \right) \]

\[ \rho_z = \text{Vertical displacement at depth } y \text{ beneath the corner of a rectangular loaded area} \]
\[ p = \text{Intensity of the applied load} \]
\[ l & b = \text{Length & width of the rectangular area respectively} \]

The factors \( A_s & B_s \) are defined as

\[ A_s = \frac{1}{2\pi} \left( \ln \frac{1 + m_s^2}{\sqrt{1 + m_s^2}} + \frac{m_r^2 - m_s}{\sqrt{1 + m_s^2}} \right) \]

where

\[ m_r = \frac{l}{b} = \frac{\text{Length of rectangle}}{\text{Width of rectangle}} \]

For deflection at \( y = 0 \) i.e. for the deflection at the surface
\[ B_r = 0 \]

Therefore, equation (3.9) may be written as

\[ \rho_z = \frac{p_b}{E_s} \left(1 - v_s^2\right) A_r \quad \ldots\ldots 3.10 \]

The principle of superposition is used to determine the vertical displacement at a point other than the corner of the rectangle. For example, in Figure (3.2) the displacement at \( c \) because of the stress load over an area of \( aeih \) may be obtained as = corner displacement of [area \( (abcd) \) - area \( (ebcg) \) - area \( (hfcd) \) + area \( (ifcg) \)]

Therefore, the soil displacement at a point \( i \) due to the load \( p_j \) on area \( j \) may be written as

\[ \delta_{si} = \sum_{j=1}^{m} I_{ij} p_j \quad \ldots\ldots 3.11 \]

where

- \( \delta_{si} \) = Soil displacement at a point \( i \) due to load on any area \( j \)
- \( p_j \) = Intensity of applied load on area \( j \)
- \( I_{ij} \) = Influence factor, i.e. influence at a point \( i \) due to an unit stress at area \( j \)
- \( m \) = Number of areas

For a single loaded area \( j \), the vertical deflections of soil at various areas \([\omega_1, \omega_2, \omega_3, \ldots, \omega_n]\) may be expressed as

\[
\begin{bmatrix}
\omega_1 \\
\omega_2 \\
\omega_3 \\
\vdots \\
\omega_i \\
\vdots \\
\omega_n
\end{bmatrix}
= 
\begin{bmatrix}
I_{1j} \\
I_{2j} \\
I_{3j} \\
\vdots \\
I_{ij} \\
\vdots \\
I_{nj}
\end{bmatrix}
\begin{bmatrix}
p_j
\end{bmatrix}
\]
where
\[ \omega_i = \text{Displacement at the centre of area } i \text{ due to the loaded area } p_j. \]

Considering all contact stress blocks \( p_1, p_2, p_3, p_4, \ldots, p_n \) and superimposing the effect on each other, the set of equations may be expressed in matrix form as:

\[
\begin{bmatrix}
\omega_1 \\
\omega_2 \\
\omega_3 \\
\vdots \\
\omega_n \\
\end{bmatrix} = 
\begin{bmatrix}
I_{11} & I_{12} & I_{13} & \cdots & \cdots & \cdots & I_{1n} \\
I_{21} & I_{22} & I_{23} & \cdots & \cdots & \cdots & I_{2n} \\
I_{31} & I_{32} & I_{33} & \cdots & \cdots & \cdots & I_{3n} \\
\vdots & \vdots & \vdots & \ddots & \ddots & \ddots & \vdots \\
I_{n1} & I_{n2} & I_{n3} & \cdots & \cdots & \cdots & I_{nn} \\
\end{bmatrix} 
\begin{bmatrix}
p_1 \\
p_2 \\
p_3 \\
\vdots \\
p_n \\
\end{bmatrix}
\]

\[ \ldots \ldots \ldots 3.12 \]

The above equation may also be expressed as:

\[ \delta_i = I_{ii} P \]

\[ \ldots \ldots \ldots 3.13 \]

where

\[ I_{ss} =
\begin{bmatrix}
I_{11} & I_{12} & I_{13} & \cdots & \cdots & \cdots & I_{1n} \\
I_{21} & I_{22} & I_{23} & \cdots & \cdots & \cdots & I_{2n} \\
I_{31} & I_{32} & I_{33} & \cdots & \cdots & \cdots & I_{3n} \\
\vdots & \vdots & \vdots & \ddots & \ddots & \ddots & \vdots \\
I_{n1} & I_{n2} & I_{n3} & \cdots & \cdots & \cdots & I_{nn} \\
\end{bmatrix} \]

\[ \delta_i = (\omega_1, \omega_2, \omega_3, \ldots, \omega_n)^T \]

\[ P = (p_1, p_2, p_3, \ldots, p_n)^T \]

\( I_{ss} \) is a square matrix and is defined as the "displacement Influence matrix" of soil.

### 3.2.3.2 Influence Matrix of Raft

The raft is discretised into the required number of nodes and elements. In order to introduce the boundary condition, a pin is introduced at the centre of the raft \( (x_p, z_p) \) and an unknown translation \( T \) is incorporated in the equation to account for the displacement at the pin.

The force vector is formed by applying unit load on the raft elements. The stiffness matrix \( [K] \) is formulated as described in equation (3.6) and section 3.2.1.
Equation (3.6) is solved for the nodal variables \( a \). The displacement at the centre of each element is obtained by using equation (3.1). These displacements are included in the matrix \([I_\beta]\), called as "influence matrix" for raft.

The force vector is formed once again by applying imposed load on the raft with the pin at the centre. Equation (3.6) is solved to obtain the displacements at the centre of each raft elements. These displacements are indicated as \( \delta_\beta \) in equation (3.14).

If the raft is loaded non-uniformly, it will rotate about its axis. Therefore two rotational unknowns \( \theta_x \) and \( \theta_z \) about the \( x \) and \( z \) axes are introduced to fix the raft against rotation. Hence the following set of equations are obtained for the raft displacement:

\[
\delta_\beta = I_\beta \cdot (-P) + a \cdot T + b \cdot \theta_x + c \cdot \theta_z + \delta_q \quad \ldots \ldots 3.14
\]

where,

\[
\begin{align*}
\mathbf{a} &= (1, 1, 1, \ldots, 1)^T \\
\mathbf{b} &= (z_1 - z_p, z_2 - z_p, z_1 - z_p, \ldots, z_n - z_p)^T \\
\mathbf{c} &= (x_1 - x_p, x_2 - x_p, \ldots, x_n - x_p)^T \\
I_\beta &= \text{Influence matrix containing influence factors for raft (i.e. } I_{12} \text{ indicates the displacement at the centre of element 1 due to the unit loading at element 2)} \\
\delta_q &= \text{Vector of deflection at the center of each raft elements for the pinned condition under the imposed loads} \\
x_j & \text{ and } z_j \text{ = } x \text{ and } z \text{ coordinates at the center of a raft element } j \\
x_p & \text{ and } z_p \text{ = } x \text{ and } z \text{ coordinates of the pinned node}
\end{align*}
\]

In the case of a symmetric condition, the rotation factors \( \theta_x \) and \( \theta_z \) may be disregarded as the slope at \( X \) and \( Y \) axes will zero. Substituting this, the equation (3.14) may be reduced to the following form:

\[
\delta_\beta = I_\beta \cdot (-P) + a \cdot T + \delta_q \quad \ldots \ldots 3.15
\]

3.2.3.3 *SOIL-RAFT INTERACTION*

In order to satisfy the soil-structure displacement compatibility, the displacement of the raft and the soil is equated as below:

\[
\delta_\beta = \delta_s \quad \ldots \ldots 3.16
\]
Substituting $\delta_z$ and $\delta_r$ in the above equation from equations (3.13) and (3.14), the following equation may be obtained as

$$I_{ss} \quad P = \quad I_{rr} \quad (-P) \quad + \quad a \quad T \quad + \quad b \quad \theta_x \quad + \quad c \quad \theta_z \quad + \quad \delta_q$$

$$(I_{ss} + I_{rr}) \quad P \quad - \quad a \quad T \quad - \quad b \quad \theta_x \quad - \quad c \quad \theta_z = \quad \delta_q \quad ....3.17$$

The total vertical applied load $P_{tot}$ is equated with the total reaction due the contact pressure underneath the raft to form another equation as :

$$\begin{bmatrix} - \tau \\ a \end{bmatrix} \quad P = \quad P_{tot} \quad ......3.18$$

where

$$\begin{align*}
\tau &= \quad (A_1, A_2, A_3, ..........., A_p, ..........., A_n), \\
A_j &= \quad \text{Areas of raft element } j.
\end{align*}$$

The moments $M_{apx}$ and $M_{apz}$ about the pinned node are equated with the moments due to the contact stresses under the raft elements i.e.

$$\begin{align*}
\begin{bmatrix} - \tau \\ b \end{bmatrix} \quad P &= \quad M_{apx} \quad ......3.19 \\
\begin{bmatrix} - \tau \\ c \end{bmatrix} \quad P &= \quad M_{apz} \quad ......3.20
\end{align*}$$

$$\begin{align*}
b &= \quad [A_j(z_i - z_p), A_2(z_2 - z_p), A_3(z_3 - z_p), ..........., A_n(z_n - z_p)] \\
c &= \quad [A_j(x_i - x_p), A_2(x_2 - x_p), A_3(x_3 - x_p), ..........., A_n(x_n - x_p)]
\end{align*}$$

Combining equations (3.17), (3.18), (3.19) and (3.20), the following matrix may be obtained :

$$\begin{bmatrix} I_{ss} + I_{rr} & -a & -b & -c \\ -a^T & 0 & 0 & 0 \\ -b^T & 0 & 0 & 0 \\ -c^T & 0 & 0 & 0 \end{bmatrix} \begin{bmatrix} P \\ T \\ \theta_x \\ \theta_z \end{bmatrix} = \begin{bmatrix} \delta_q \\ -P_{tot} \\ -M_{apx} \\ -M_{apz} \end{bmatrix}$$
The above equations can be solved to obtain the following unknowns:

- Contact pressures \([P]\) underneath raft elements
- Translation \(T\)
- Rotation \(\theta_x\) about \(X\)-axis
- Rotation \(\theta_z\) about \(Z\)-axis

For a symmetrical condition, the rotations \(\theta_x\) and \(\theta_z\) will be zero and the above set of equations may be reduced to

\[
\begin{bmatrix}
I_{xx} + I_{rr} & -a \\
-a^T & 0
\end{bmatrix}
\begin{bmatrix}
P \\
T
\end{bmatrix}
= 
\begin{bmatrix}
\delta q \\
-P_{\nu t}
\end{bmatrix}
\quad \ldots \ldots 3.22
\]

The above equations can be solved for the contact stresses \([P]\) and the translation \(T\), where \(T\) is the displacement of the raft at its pinned location i.e. the centre of the raft. These contact stresses are used in combination with the imposed load to form the force vector \([F]\). The nodal variable \(a\) is obtained by solving the equation (3.6) by using the force vector. The moments per unit length are then calculated as described in equation (3.7).

These nodal displacements are for the pinned raft and the final displacements may be obtained by adding the translation.

### 3.2.4 Analysis with Free Field Soil Movement

In a reactive soil environment, clay soils change in volume to varying degrees in response to changes in their moisture content (Holland, 1981). The volume of the soil increases with the wetting of the soil system and decreases with the subsequent seasonal drying effect. Due to this change in the volume, the clay soils subject to vertical as well as horizontal movements. The horizontal movement causes opening of surface cracks in the drier months of the year, which close during the wetter months. Whereas, the vertical movements lead to cyclic changes in the soil surface level. The magnitude of these cyclic movements decreases with depth down to a level below which no volume change occurs. This depth is known as the "depth of seasonal movement". The following major factors influence the magnitude of seasonal heave of the clay soils:

- Type and amount of clay
- Soil profile
• Site drainage
• Climate
• Foundation loading
• Localised moisture excess
• Location and type of vegetation

The presence of an impermeable surface or slab on expansive soil influences the pattern of seasonal moisture change. The slab acts as a barrier to the process of direct drying or wetting of the soil system. Therefore, the movement of the ground underneath the raft is expected to be different compared to the free ground condition.

A raft slab, constructed during the driest period, experiences an upward ground movement due to the swelling of soils in the following wet season. As a result of this, the edge of the raft experiences higher upward movement compared to the central portion of the raft due the swelling of soil along the raft edge (Figure 3.3 (B)(ii)). The swelling of the soils along the edge of the raft is high because of its direct exposure to the seasonal temperature fluctuations. This type of soil movement is designated as "edge heave formation" underneath the raft. The following cycles of the wetting and drying processes cause a slow penetration of moisture towards the central portion of the raft and results in swelling over the course of time. The clay soil towards the central portion of the raft is not sensitive to the seasonal moisture fluctuation as it is not directly exposed to the atmosphere. Whereas, the soil along the edge of the raft experiences swelling and shrinking effects with the seasonal moisture fluctuation. The cycles of moisture variation cause further increase in the moisture intrusion towards the centre of the raft. As a result of this, the soil under the central raft swells and remains in that state, because of the entrapment of the moisture. This process causes a "central mound formation" underneath the raft (Figure 3.3 (B) (v)). Therefore, a raft constructed in the driest period of time initially experiences an "edge heave", but in a long term it subjects to a "central heave" formation.

Similarly, a raft slab constructed in a wet ground condition, already rest on a swollen soil profile, which is flat initially, but with the following dry season, the soil along the edges of the raft experiences a shrinking effect and results in a downward movement. The moisture inside the central portion of the raft remains entrapped, because the raft acts as an impermeable barrier. This process causes a "central mound formation" underneath the raft.

Therefore, the initial state of the ground moisture condition dictates the formation soil profiles in the following cycles of the seasonal wetting and drying processes. These relative ground movements have a great impact on the total and the differential settlements of the raft.
Lytton (1970) assumed the soil profile to be a polynomial shape and adopted the equation for the "mound" as

\[ y = c \cdot x^m \] ......3.23

where

- \( y \) = Distance below highest point of mound
- \( x \) = Distance from the highest point along the raft
- \( m \) = Integer exponent (mound shape index)
- \( c \) = A constant

Poulos (1984) also recommended the above soil profile as suitable for the practical purposes. It is also understood that the soil profile is a function of the "integer exponent \( m \)" which will depend on the soil type and its inherent property. The influence of \( m \) on the foundation performance has been studied and discussed in section 3.4.2. Mitchell (1984) investigated twenty-five raft slabs in expansive soil environment and also presented their respective mode of distortions. Out of them some square raft slabs experienced a "central doming profile" underneath them. Other profiles were also observed for various sizes and shapes of the raft slabs.

In the present analysis, a "dome shaped soil profile" has been adopted to carry out the analysis of the square rafts. In other words, in the case of a "central heave" formation the soil surface moves down in a dome shaped parabolic profile in all direction from the centre of the raft, where \( x \) in equation (3.23) represents the distance at any point underneath the raft from the centre of the raft.

The raft analysis is carried out incorporating the effect of rotation and twisting in between various raft elements and also including the movement of the ground underneath it. A non-linear approach is adopted to solve the stresses at the soil-raft interface. In order to incorporate the movement of the ground, central and edge heave formations as indicated in Figure (3.3 A) are considered in the analysis.

### 3.2.4.1 Free Field Soil Movement Profile

The analysis is undertaken with the soil profiles representing central and edge heave conditions in order to incorporate the various stages of the swelling and shrinking of soils.
underneath the foundation. The mathematical formulation to define the heave profile under the foundation is described below.

In the case of "central heave formation", the maximum free-field soil movement \( S_o \) occurs at the corner of the raft (Figure 3.3 (A)(ii)). Substituting this value in equation (3.23), it may be obtained as:

\[
S_o = c \left( \frac{L_R}{\sqrt{2}} \right)^m \quad \text{......3.24a}
\]

where

\[
L_R = \text{Length of a square raft}
\]

\[
c = S_o \left( \frac{\sqrt{2}}{L_R} \right)^m
\]

Substituting \( c \) in equation (3.23), it may be written as

\[
y_c = S_o \left( \frac{\sqrt{2}}{L_R} \right)^m \cdot x^m
\]

where

\[
y_c = y \text{ values for central heave profile}
\]

\[
y_c = S_o \left( \frac{\sqrt{2}}{L_R} x \right)^m \quad \text{......3.25}
\]

In the case of a strip footing subjected to a "central heave formation", \( S_o \) occurs at the edge of the foundation and the equation (3.25) may be expressed as

\[
y_c = S_o \left( \frac{2x}{L_R} \right)^m
\]

In the case of "edge heave formation", the soil profile may be defined as

\[
y_e = -y_c
\]
where
\[ y_e = \text{y values for the edge heave} \]

These \( y \) values are included in a matrix \( S_i \), indicated as the vector of soil movement underneath the foundation.

For the analysis purposes, a parabolic shaped mound \((m = 2)\) has been adopted to incorporate the ground movement underneath the raft, which represents a highly reactive soil condition. It should be noted that the mound index is dependent on the soil type and the moisture intrusion towards the central portion of the raft. Therefore, prior to analysing a foundation system in these situations, a proper mound index should be adopted based on local experience and the site condition.

### 3.2.4.2 Foundation System With Free-Field Soil Movement

In order to incorporate the effect of the free-field soil movements, the equation for soil displacement (3.13) is modified as follows:

\[ \delta_s = I_{ss} \ P + S_i \quad \ldots \ldots 3.26 \]

where,
\[ S_i = \text{Vector of free field soil movement underneath each raft element} \]

To satisfy the "soil-raft" interface compatibility, the displacements of raft and soil elements are equated as below:

\[ \delta_r = \delta_s \]

Substituting, the displacements from equations (3.15) and (3.26), it may be expressed as

\[
\begin{align*}
[-P] \ I_{rr} + a \ T + \delta_q &= [P] \ I_{ss} + S_i \\
[I_{rr} + I_{ss}] \ [P] - a \ T &= \delta_q - S_i \\
\end{align*}
\]

\[ \ldots \ldots 3.27 \]

The above set of equations are solved for the contact stresses and the translation as described in previous sections. These contact stresses are used to determine the moment and the nodal displacements as described in section 3.2.3.3 and equation (3.7).
3.2.5 Other Conditions

The analyses are extended to incorporate the effect of "non-linear" as well as "elastic" conditions. The characteristics of those conditions are described in sections 3.2.5.1 and 3.2.5.2.

3.2.5.1 Elastic Condition

Under elastic conditions, neither lift-off nor yielding of soil under the foundation have been considered in the analysis. In case of any separation between the soil and the raft, the elastic analysis considers a tensile force at the soil-raft interface in the form of negative contact stress. It also considers the bearing capacity of the founding soil to be infinite and allows for no yielding of soil under the foundation.

3.2.5.2 Non-Linear Conditions

In contrast to the elastic condition, the non-linear analysis employed here incorporates the effect of lift-off as well as yielding of soil underneath the slab. These conditions are incorporated in the analysis as described in the following sections.

3.2.5.2.1 Lift-off Condition

In the event of detachment between the raft and the soil, the non-linear analysis allows lifting-off and restricts the contact stresses in the lifted areas by zero or a specified limiting tensile stress. The limiting tensile stress will depend on the adhesion property between the soil and the raft material.

3.2.5.2.2 Local Soil Yield

Under high imposed loading on the foundation, the induced contact stresses in the soil may reach the bearing capacity of the soil at various points within the mass. Local yielding then occurs and if the foundation is further loaded, a redistribution of stresses occurs within the soil. In other words, if the contact stress under any part of the raft reaches the bearing
capacity, the contact stress for that part is equated to the bearing capacity and any further increase in stress is redistributed to other portions of the raft.

In order to incorporate this effect, the contact stress under elements which have reached the limiting value, is set equal to the limiting value. The other equations are modified to incorporate the redistribution of stresses on the other parts of the slab.

3.3 DEVELOPMENT OF COMPUTER MODEL

In order to carry out the analysis of raft foundations, a computer program has been developed by adopting the above method of analysis. The program is written in Fortran 77 and is named RAES (Raft Analysis in Expansive Soils). The various stages of the computer analysis are outlined below:

1. INPUT DATA:

   In this stage, the required data outlining the various foundation conditions are read by the program from the input file. The input block of the program is structured to read the following data to undertake the analysis:

   - Number of nodes and elements
   - Geometrical properties of the raft
   - Material properties of the raft and the soil
   - Boundary conditions
   - Imposed loads on the raft
   - Maximum vertical soil movement due to swelling or shrinking of the soil
   - Required analysis type i.e. Elastic or Non-linear

   If elastic analysis is required
   - No Lift off and no local soil yield conditions are allowed

   If non-linear analysis is required
   - Limiting lift-off and local soil yield pressures are specified

2. GENERATING SOIL PROFILE:

   In this segment of the program, the movement of the ground due to swelling or shrinking soils is incorporated in the analysis. The vertical soil profile under each raft element is generated by using the equation (3.25) and stored in an array for later application.
3. **Formation Of Stiffness Matrix:**

The stiffness matrix is formed for all individual elements and coupled to obtain the global stiffness matrix. This is obtained by using numerical integration (Gaussian Quadrature) over the rectangular area of the element as discussed earlier. The stiffness matrix is formed in a banded form in order to reduce the computer memory requirements. It is denoted as \([K]\) in equation (3.6).

4. **Formation of Load Vector Due to Imposed Loading:**

The load vector is formed from the specified imposed loads on the raft. The present analysis has the ability to incorporate the uniform and the point loads at various locations of the raft. The capability of the method may be enhanced to incorporate the applied moments with some modifications in the equations. The load vector is a column matrix containing \(4 \times \text{no. of nodes}\) number of rows.

5. **Boundary Conditions:**

The derived equations from step (3) and (4) are modified for the following boundary conditions:

i) **Pinned Condition:** the center of the raft is taken to be pinned.

ii) **Rotations & Twists:** The rotations and twists along the axes of symmetry are set to zero.

6. **Solving for Pinned Raft Displacement For Imposed Load:**

The matrices are solved for the nodal variables \(a\). The displacements at the center of the raft elements are determined by using the equation (3.1). These displacements are denoted as \(\delta_i\) in equation (3.17).

7. **Formation Of Load Vector By Applying Unit Loading:**

The force vector is formed by applying an unit load on the raft. The process of obtaining the matrix is similar to that described in step (4).
8. **Boundary Conditions**:

The above load vector is modified to incorporate the boundary conditions, such as the conditions of symmetry.

9. **Solving For Raft Interaction Factors**:

The equation (3.6) is solved for the unknown vector \( a \) by using the stiffness matrix determined in step (3) and the force vector obtained in step (7). The displacements at the center of the raft elements are determined by using the equation (3.1). These displacements are used as the interaction factors \( (I_r) \), as defined in equation (3.14).

10. **Soil Influence Matrix**:

The soil influence matrix \([I_{SS}]\) is determined by applying an unit uniform load over an area \( j \) and determining the displacements at the centre of all element. The mathematical formulation is described in section 3.2.3.1. These factors are coupled with the raft influence factors to obtain \((I_{SS} + I_{rr})\), as defined in equation (3.17).

11. **Formation Of Total Load Equation**:

The vertical applied load \( P_{vor} \) is equated to the total contact pressures multiplied by the relevant element areas as indicated in equation (3.18). The main matrix is formulated by arranging the above determined factors as shown in equation (3.21).

12. **Introducing Vertical Soil Profile Under The Raft**:

\( \delta_q \) is modified to incorporate the soil movement vector \((S_i)\), as determined in step-2 to incorporate the ground movement due to swelling or shrinking of soil as shown in equation (3.27).

13. **Modifying The Matrix For Non-Linear Conditions**:

This section of the program is activated after the first run and undertakes following two processes, if directed:
**Lift-off Condition:**
If the contact stress under any raft element is subjected to tensile contact stress
(which is found out after the first run), then this section of the program will process
those contact stresses as described in section 3.2.5.2.1.

**Local Soil Yield:**
If the contact stress under any element exceeds the specified value, which is determined after
the first run, this section of the program will process those contact stresses as described in
section 3.2.5.2.2.

The lift-off condition under various elements is incorporated before the yielding of soil under
raft elements. This process releases the tensile stress at the soil-raft interface and allows the
detachment. This sequence is carried out to eliminate the tensile stresses, which are beyond
the limiting value. Otherwise, these stresses will impose higher load on the raft, which is not
the actual loading. Therefore, processing of the soil yielding is undertaken after the complete
processing of lift off.

14. **Solving The Main Matrix:**
The set of equations shown in equation (3.22) are solved for the contact stresses \([P]\) and
translation \(T\), by using the Gaussian Elimination Method.

15. **Screening Stage:**
In this stage of the program, the contact stresses are screened for the following, if directed:

- Tensile Stresses underneath the raft elements exceeding the specified limit &
- Contact Stresses exceeding the specified value.

If the contact stress under any raft element is detected in the above category, the
program is diverted to step (13) to follow the further steps. This process is
repeated until all contact stresses satisfy the following criteria:

- All contact stresses are equal to or more than the specified tensile stress value.
- All contact stresses are equal to or less than the specified limiting value of
  the contact stress.

16. **Formation Of The Force Vector:**
The force vector \([F]\) is formed with the contact stresses determined in step (14) together
with the imposed loading on the raft.
17. **SOLVING FOR UNKNOWNS:**

Equations (3.6) are solved with the force vector determined in step (16). The nodal variables such as displacements, rotations & twists are determined. These nodal variables are used to determine the moment and twist per unit length using equation (3.3).

### 3.4 SOME PRELIMINARY ASPECTS OF RAFT BEHAVIOUR

In this section, the basic behaviour of a raft foundation subjected to uniform and non-uniform ground conditions has been analysed and presented in order to form a basis for the further investigation. The results have also been compared with the available solutions developed previously by other authors.

#### 3.4.1 RAFT ON UNIFORM GROUND CONDITION

Generally, the behaviour of a uniformly loaded raft foundation depends on the relative raft stiffness and the applied load on it. The distributions of contact pressure, settlement and bending moment of a raft foundation depend greatly on the raft rigidity.

In order to carry out the analysis, a quarter portion of $3m \times 3m$ raft is analysed by dividing it into $100$ equal size elements. The results are presented in Figures (3.5), (3.6) and (3.7) in a dimensionless form, therefore these values are applicable to any raft sizes and loading.

In case of a uniformly loaded relatively rigid raft, the differential settlement is very small (nearly zero) and the maximum bending moment occurs at the centre of the raft. The raft undergoes uniform settlement and is subject to very high contact pressures at the edge. Under elastic conditions, the contact pressures tend to build up at the corner, but in the case of a non-linear condition, the soil at these highly stressed areas yields and the stresses are redistributed to other parts of the raft. This process continues until a balance is achieved between the imposed load & the contact pressures following the process of redistribution of stresses. The raft may be subject to a complete bearing failure if the yield of soil occurs under all raft elements. The yielding of the soil causes reduction in the edge pressure as well as the bending moment, but increases the total settlement.

On the contrary, a fully flexible raft is unable to redistribute the effect and, as a result of that the contact pressures at all points of the raft become equal to the corresponding applied loads. For such cases, the bending and twisting moments are very small, but the differential
settlements are relatively high. The settlement at the centre of the raft is significantly higher than at the corner or the edge.

It can be seen in Figure (3.6) that the differential settlement is high in case of a flexible raft and reduces with the increase in the stiffness. The moments are higher in the case of a rigid raft and reduce with a decrease in raft stiffness (Fig. 3.5). The total settlement is higher for a flexible raft than for a rigid raft (Fig. 3.7).

Because of the concave shape of the displacement profile, the maximum bending moment is always positive, i.e. tension occurs at the bottom of the raft, and the maximum value occurs at the centre of the raft. As the raft stiffness approaches the fully flexible condition, the maximum bending moment moves towards the edge.

The above results have been compared with the solutions of Wardle and Fraser (1976) and presented in Figures (3.5) to (3.7). It can be seen from Figures (3.6) and (3.7) that the settlement values obtained from the present analysis agree well with Wardle and Fraser (1976). The agreement for the moment values is also good with the maximum difference in result of the order of 10%. It should be noted that the number of elements adopted in both analyses would cause some difference in result.

3.4.2 RAFT ON NON-UNIFORM GROUND CONDITION

In order to demonstrate the behaviour of foundation on soils subjected to the relative ground movement a strip footing with a length to width ratio of 20 (20 m x 1m) has been analysed for both central and edge heave conditions. The footing, which is 10m by 0.5m, has been divided into 20 elements to undertake the analysis.

The type of soil profile influences the foundation performances quite significantly. It is obvious that higher “mound index” will cause a flatter soil profile and will create a less adverse effect to the foundation. It can be seen from figures (3.8) and (3.9) that the settlement and the moment values are inversely related to the mound index m.

The soil profile due to the swelling or shrinking of soils induces additional differential settlement and bending moment into the foundation. The settlement profiles of the strip footing for central heave formation are shown in Figure (3.12) for various strip stiffnesses.
The reduction in settlement is approximately 62% for the mound index value varying from 2 to 4 for central heave condition. The moment is reduced by approximately 25% for the same mound index variation.

The relative strip stiffness influences the settlement performance significantly. Higher stiffness causes reduction in total as well as in differential settlement (Figure 3.12). The displacement and the moment profiles along the raft centreline are also analysed and are shown in Figures (3.10) and (3.11).

The accuracy of result is dependent on the number and the size of the elements. In case of separation in between the raft and the soil surface, the elastic analysis considers a tensile stress at the soil-raft interface. For a larger raft element, if the detachment occurs at the middle of an element, the whole element recognises a tensile stress and causes a higher pull down effect on the footing. In this situation, finer discretisation will improve the accuracy of result. The tensile stress is higher in case of rigid raft due to its rigidity and therefore, the number of elements and the soil profile play a very important role in the stress analysis. In contrast, a flexible raft follows the profile of the soil surface and experiences less or no tensile stress underneath it.

These obtained results have been compared with Poulos (1984) and presented in Figures (3.8) to (3.11). It can be seen from Figures (3.10) and (3.11) that the agreement of results are generally good, with the maximum difference within an order of 8%.

3.5 **UNIFORM RAFTS**

In this section, the behaviour of uniformly thick raft foundations have been analysed for central-heave and edge heave conditions. The analyses have been undertaken for elastic and non-linear conditions. The influence of the non-linear conditions such as lift-off and local soil yield under various raft elements have been studied and compared with the elastic conditions. The implication of these parameters on the raft performance has also been discussed.

3.5.1 **INFLUENCE OF GROUND MOVEMENT**

In this section, the attention has mainly been focused on the raft behaviour under elastic and non-linear conditions. The analysis has been carried out by incorporating the soil profile as indicated in equation (3.26). As discussed earlier, in case of any separation between the slab and the ground, the elastic analysis considers tensile stress at
the slab-ground interface. These tensile forces cause a "pull-down" effect on the raft. However in most real situations, the soil is incapable of withstanding tension on the raft beyond a limit. Therefore a detachment occurs at the "raft-soil interface" and the stresses are redistributed to other parts of the raft. This process causes a non-linear effect on the foundation behaviour and releases the raft from the tensile stresses. Due to this, a significant reduction in moment and differential settlement takes place. It can be seen from Figures (3.13) to (3.16) that, in the elastic condition, very high moment and differential settlement occur in the foundation compared to the non-linear condition. The contact stresses under some raft elements attain a very high value in the elastic case, but for the non-linear condition, a process of detachment and redistribution of the imposed load take place. Whenever these contact stresses exceed the sustaining capacity of soil, local yielding occurs and redistributes the stress to other raft elements. This process continues until a balance between the imposed load and the contact pressure is reached. This process also causes non-linearity in raft behaviour and increases the total settlement as indicated in Figure (3.17).

Although the elastic analysis has the advantage of generalising the raft behaviour dimensionally, it may mislead in the case of any detachment between the raft and the soil. The elastic analysis also gives high moments and differential settlements in the case of a rigid and semi-rigid raft, compared to non-linear analysis (Figures 3.13 to 3.16). As the raft tends to become more flexible the difference reduces because of its lesser ability of redistributing the load and, as result of this the raft deflects to follow the soil profile, i.e. there is no occurrence of lifting-off. Therefore, no tension develops under the raft element and the results from the elastic and non-linear analyses are similar.

The total settlement of a rigid raft is high under the elastic condition, as the raft experiences additional force due to the tensile stress between the raft and the soil. The total settlement also increases due to yielding of soil under the raft. The influence of various specified soil yielding values ($p_{yield}$) on the total settlement of a rigid raft is shown in Figure (3.17). It can be seen that in case of central heave condition, the increase in settlement is approximately 39% for a variation of "$p_{yield}$" value from 300 kPa to 100 kPa. For the edge heave condition, a 47% reduction in the upward displacement takes place for the same variation in "$p_{yield}$" values.

### 3.5.2 Settlement Characteristics

The ground movement due to swelling or shrinking soils, changes the stress distribution pattern underneath the raft because of the non-uniform ground support condition. This
phenomenon may induce additional differential settlement and moment into the raft, which also depend on the raft stiffness. For a uniformly loaded raft, "central heave" causes the maximum displacement at the corner of the raft and the minimum at the centre, whereas "edge heave" results in maximum displacement at the centre and minimum at the corner.

The contact stress distribution underneath the raft is dependent on the type of heave. It can be seen in Figures (3.13) and (3.15) that for the "Lift Off Only Case" the differential settlement is higher in case of the edge heave compared to the central heave condition. The differential settlement values for central heave for the "Lift-Off Only" and "Lift Off + Soil Yield @ 300 kPa" cases indicate the same behaviour, because after lift-off, generally no contact stress exceeds 300 kPa, whereas in the case of the edge heave condition, there is a difference in the magnitude of the differential settlement in these two cases, because after lift-off, contact stresses under some elements exceed 300 kPa, which causes further reduction in the differential settlement.

The differential settlement reduces as the soil yield pressure ($P_{yield}$) decreases. Soil yielding under the raft causes an increase in the total settlement, which results in increased areas of contact between the raft and the soil. This process reduces the unsupported portion of the raft and as a result causes a reduction in the differential settlement.

It is evident that due to the ground movement, the raft foundation may experience additional differential settlement, which may be beyond the range of acceptable foundation behaviour.

In order to cater for this problem, the most reliable and widely acceptable solution is to adopt a rigid raft or to introduce stiffening beams at strategic locations to restrict the differential settlement in the raft. Generally, a stiffened raft may be the preferred option as it provides an effective solution to this problem. Although the behaviour of stiffened rafts have been analysed by various authors in the past, in section (3.6), a brief analysis of these type of foundation systems has been carried out with the developed method of analysis and the domed soil profile underneath the raft. The present analysis incorporates the effect of torsion and twisting in the slab and the beam, which may contribute further refinement in the end result.

### 3.5.3 Moment Analysis

The magnitude of moments are high in the case the edge heave condition compared to the central heave case (Fig.3.14) and (Fig.3.16). The moment reduces significantly with a
decrease in the value of $p_{\text{yield}}$ for rigid and semi-rigid rafts. For the flexible rafts ($K_R \leq 0.001$), the moments are not influenced by the soil yield stress values ($p_{\text{yield}}$).

For a uniformly loaded raft, the maximum bending moment is always positive in case of edge heave i.e. tension occurs at the base of the raft, but is negative in case of the central heave condition.

3.6 STIFFENED RAFTS IN SWELLING AND SHRINKING SOILS

In order to analyse stiffened rafts, a non-linear approach has been adopted at the soil-raft interface. The analysis has been carried for central as well as edge heave conditions. The parameters adopted for the analysis are summarised Table 3.1.

<table>
<thead>
<tr>
<th>Table 3.1</th>
<th>(soil and raft parameters)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameters</td>
<td>Values</td>
</tr>
<tr>
<td><strong>Raft:</strong></td>
<td></td>
</tr>
<tr>
<td>Size ($L_R \times B_R$)</td>
<td>$10m \times 10m$</td>
</tr>
<tr>
<td>Poisson's ratio of raft material ($v_r$)</td>
<td>0.2</td>
</tr>
<tr>
<td>Modulus of raft material ($E_r$)</td>
<td>$2 \times 10^8$ kPa</td>
</tr>
<tr>
<td>Applied loading ($q$)</td>
<td>10 kPa</td>
</tr>
<tr>
<td>Slab thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Beam width</td>
<td>300 mm</td>
</tr>
<tr>
<td>Beam depth</td>
<td>300 mm</td>
</tr>
<tr>
<td><strong>Soil:</strong></td>
<td></td>
</tr>
<tr>
<td>Poisson's ratio of soil ($v_s$)</td>
<td>0.3</td>
</tr>
<tr>
<td>Modulus of soil ($E_s$)</td>
<td>$1 \times 10^4$ kPa</td>
</tr>
<tr>
<td>Soil Yield ($p_{\text{yield}}$)</td>
<td>300 kPa</td>
</tr>
<tr>
<td>Maximum soil free field movement ($S_s$)</td>
<td>100 mm</td>
</tr>
<tr>
<td>Allowable tensile stress ($p_{\text{tension}}$)</td>
<td>10 kPa</td>
</tr>
</tbody>
</table>

The central line of beams are placed at equal distances. The objective of this section is to analyse the influence and benefits of stiffened beams in restricting the differential settlement and their effect on the maximum moment. The effects of slab-beam configuration have also been analysed and discussed for various slab stiffnesses. The analysis has been carried out for two configurations (Config.I & Config.II) as indicated in Figure (3.4) in order to demonstrate their influence on the overall foundation performance. The results for the central and the edge heave formations are presented in.
Figures (3.18 to 3.21) and Figures (3.22 to 3.25) respectively. Figure (3.26) presents the raft behaviour in the central as well as the edge heave condition.

3.6.1 Settlement Analysis

It can be seen from Figures (3.18) and (3.22) that the differential settlement of a rigid unstiffened raft is negligible and increases as the raft tends to become more flexible. The differential settlement factor ($\Delta/S_n$) approaches unity for a very flexible raft, in other words, the deflected shape follows the profile of the soil underneath the foundation, causing maximum differential settlement.

The differential settlement of the raft may be reduced by incorporating stiffened beams at strategic locations, thus creating a bridging action across the heave. In Figure (3.20) and (3.24), the factor $\alpha_r$ is indicated as the reduction in the differential settlement because of the stiffened beams. These reductions are compared with the values of the unstiffened raft as shown in Figures (3.18) and (3.22). It can be seen from Figures (3.20) and (3.24), that configuration II provides a better stiffening effect than configuration-I, because of the greater number of beams and their arrangement.

The reduction of the differential settlement is also influenced by the depth of the stiffening beam. It can also be seen from Figures (3.20) & (3.24) that an increase in the beam depth reduces the differential settlement. However, it may be observed that beyond a certain value of $K_R$, no further reduction in differential settlement takes place with further increase in the slab stiffness. This may be defined as a limit beyond which the rigidity of the slab itself is stiff enough to restrict the differential settlement and any further addition of beams has no influence in the raft performance. It is also understood that the differential settlement is not a critical issue for higher $K_R$ values.

3.6.2 Moment Analysis

The moment in an unstiffened rigid raft is high, but reduces as the raft tends to become flexible (Fig.3.19 and 3.23). In Figures (3.21) and (3.25), the factor $\alpha_m$ expresses the increase in the maximum moment due to the stiffened beams. It can be seen from these figures that the maximum moment increases with an increase in the beam depth. The increase in moment is less for Config.II compared to Config.I. This is because of greater number of beams are sharing the effect of relative ground movement underneath the raft.
Generally, the moment is negative for a shrinking profile of the soil, and positive in the case of a swelling soil profile.

The analysis has also been carried out for various $S_0$ values due to swelling and shrinking soils. The results are presented in Figure (3.26) and may be adopted as design charts to assess the stiffening requirements in order to limit the differential settlement to a specified value for the raft size indicated in Table 3.1.

3.7 SUMMARY

Raft behaviour under normal ground conditions is generally well understood. For elastic conditions, the bending moment and settlement response characteristics are generally assumed to be linear and may be represented by means of non-dimensional factors as indicated in Figures (3.5) and (3.6). These non-dimensional factors represent the raft behaviour for any raft stiffness and uniform applied loading.

The overall behaviour of the raft foundations in a reactive soil environment is generally understood, however the issues of various shapes of the ground movement underneath the raft and the accurate assessment of its impact on the soil-structure interaction need to be resolved. In various stages of development, both analytical and field testing were undertaken to understand the mode of deformation of the raft when subjected to a series of wetting and drying cycles. The methods of analysis were developed and their results were compared with the field data. The methods were acceptable, however there is a scope of further refinement in the analysis to achieve higher degree of accuracy. The computer model RAE5 provides some additional features, which should improve the accuracy of the end result. The program is capable of incorporating the effect of rotations and twisting in the raft, which are important aspects in the raft behaviour when subjected to a deformation due to the change in the ground support condition. The program also provides the facility of incorporating different types of free field soil movements under various raft elements, including non-linear effects in the foundation behaviour.

The actual assessment of the foundation behaviour in reactive soil is quite complex because of the movement of the ground underneath the raft. It is necessary to carry out a non-linear analysis to determine the accurate deformation of the raft. The behaviour of the raft under these circumstances may not be represented by means of elastic solutions, because the elastic analysis considers tensile force at the soil-raft interface in case of detachment. These tensile forces add on to the imposed loading and indicates entirely a different picture of the foundation response, which is misleading. It is understood that soil can not generate tensile
force on the raft surface beyond a limit and this limiting value depends on adhesion property of soil with the raft material. Therefore it is desirable to incorporate these effects into the analysis in order to ascertain the real raft behaviour subjected to these conditions. These effects are included in the non-linear conditions described in this chapter. The settlements and the moments obtained from a non-linear analysis may be significantly different compared to the elastic condition. For rigid and semi-rigid rafts, elastic analysis results in high moment and differential settlement, whereas non-linear conditions lead to smaller bending moments and differential settlements. For a very flexible raft, elastic and non-linear analyses produce similar result as the raft follows the soil profile and no tensile stress occurs between the soil and the raft.

Local soil yield i.e. local bearing capacity feature, reduces the differential settlement for both central as well as edge heave conditions. Yielding of soil causes a reduction of differential settlement and an increase in the total settlement. The increase in the total settlement causes further contact between the raft and the soil. This process reduces the unsupported area of the raft and, as a result of this, a reduction in the differential settlement takes place.

The mound index \( m \) has a great deal of influence on the raft performance. It is dependent on the soil type and the local conditions of the site. Higher values of \( m \) causes a flatter soil profile and leads to lesser bending moment and settlement, whereas a lower value of mound index forms a steeper soil profile and leads to higher bending moment and settlement.

The problem of excessive settlements caused by the swelling or shrinking soils may be countered by strengthening the foundation structurally to cater for the uneven ground support condition.

The concept of strengthening of raft by stiffening it structurally at required locations is an effective and reliable solution to this problem. The raft may be stiffened by incorporating beams at strategic locations with various types of beam-slab configuration. Different types of beam-slab configuration influence the load distribution pattern differently and leads to different performance of the raft. As discussed earlier Configuration-II (Figure 3.4) is more effective in reducing the differential settlement than Configuration-I. The moments are higher in Config.-I than Config.-II, because a greater number of beams in Configuration-II are sharing the effect of load redistribution, resulting from the relative ground movement.

The behaviour of a raft slab on non-uniform ground support condition is difficult to generalise because of its non-linear behaviour. Therefore, it is generally necessary to carry out a numerical analysis to estimate the moment and settlement behaviour. The
The mathematical model adopted to develop the computer program may be considered as a useful method of obtaining a solution to this problem.

The settlement behaviour of a raft may also be influenced significantly by introducing piles at various locations underneath the raft. The concept of introducing piles in order to reduce the differential settlement requires more careful analytical attention, as piles are also influenced by the ground movement. Piled raft systems subjected to ground movement are analysed and discussed in detail in later chapters of this thesis.
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The Analysis of Raft Foundations

Finite Element Mesh for Raft

Z - Axis

X - Axis

Y - Axis

Semi - Infinite Soil mass

Figure 3.1(a) : Discretisation of Raft

Z - Axis

(1,2)

(2,2)

b

X - Axis

(1,1)

(2,1)

l

Figure 3.1(b) : Rectangular Plate Element

Figure 3.2 : Method of super-imposition
(i) Raft on Soil

(ii) Central Heave Modelling

(iii) Edge Heave Modelling

*Figure 3.3 (A): Soil profile modelling*
(i) Slab placed on dry site

(ii) Middle of First Wet Season After Placement (Edge heave formation)

(iii) Middle of First Dry Season After Placement

(iv) Middle of Second Wet Season After Placement

(v) Long Term Mound Condition (Central heave formation)

Figure 3.3 (B) : Idealised mound development
Plan showing slab-beam arrangement

Figure 3.4(a): Configuration - I

Plan showing slab-beam arrangement

Figure 3.4(b): Configuration - II

Figure 3.4: Stiffened raft analysis: - slab-beam configuration
Figure 3.5: Bending moment analysis

Figure 3.6: Differential settlement analysis
Figure 3.7: Maximum settlement analysis

Figure 3.8: Effect soil profiles on strip settlement
Strip Footing Analysis with free field soil movement

$L_x / B_x = 20$
$K_x = 0.005$

Central Heave Analysis

$m = 2$
$m = 4$

Figure 3.9: Effect of soil profile on strip moment

Strip Footing Analysis with free field soil movement

$L_R / B_R = 20$
$K_R = 0.001$

Edge Heave Analysis
Parabolic profile

$\delta = \text{Upward displacement}$

Figure 3.10: Settlement along strip
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Figure 3.11 : Moment along strip

Figure 3.12 : Settlement along strip for various stiffnesses ($K_R$)
Figure 3.13: Non-linear effects on differential settlement

Figure 3.14: Non-linear effects on moment
Figure 3.15: Non-linear effects on differential settlement

Figure 3.16: Non-linear effects on moment
Figure 3.17: Influence of specified yield stress ($p_{yield}$) on raft displacement

Figure 3.18: Raft stiffness vs Differential settlement
**Figure 3.19 : Raft stiffness vs Moment (Central heave)**

**Figure 3.20 : Differential settlement reduction factors (Central heave)**
Figure 3.21: Moment increment factor (Central heave)

Figure 3.22: Raft stiffness vs Differential settlement (Edge heave condition)
Figure 3.23: Raft stiffness vs Moment (Edge heave condition)

Figure 3.24: Differential settlement reduction factors (Edge heave condition)
Figure 3.25: Moment factors (Edge heave condition)

Figure 3.26 (a) Differential settlement
Figure 3.26 (b) : Moments
CHAPTER - 4

THE ANALYSIS OF PILE FOUNDATIONS

4.1. INTRODUCTION

The behaviour of pile foundations in swelling or shrinking soils is greatly influenced by the additional forces induced by the movement of the ground along the pile shaft. If piles are installed in a "wet ground" condition, they may be subject to a downdrag force when the soil shrinks and moves downwards due to the reduction in the moisture content. These downdrag forces cause additional compressive force on the pile and may increase the settlement quite significantly. In the same manner, when piles are installed in a "dry ground" condition, they may experience an uplift force when the soil swells and moves upward due to the increase in the water content. These uplift forces are generally critical in case of the piles used for the anchoring purposes, where the applied force is upward. These features influence the overall foundation performance quite significantly. Therefore, the pile design under these circumstances requires careful attention to ensure that the induced displacements and the forces remain within the acceptable range.

In this chapter, piles subjected to the "free-field soil movement" caused due to the process of swelling or shrinking of soils have been analysed and the results have been compared with the piles in the normal ground condition (i.e. no ground movement). The settlement characteristics has been analysed and presented for both piles in compression as well as in tension. The stress distribution along the pile shaft due to the ground movement has also been discussed in detail. The understanding of this behaviour has been used in later chapters in analysing the behaviour of pile groups and the piled raft foundation systems in swelling or shrinking soils.

The piles are analysed as a number of uniformly loaded cylindrical elements together with uniformly loaded circular bases. The interaction effects between the shaft elements and the bases are considered in order to formulate the equations. Finally, solutions are obtained for the distribution of the shear stresses along the pile shafts, base pressures and the displacement of piles.

A computer program PAES (Pile Analysis in Expansive Soils) is developed by using FORTRAN 77 to carry out the necessary mathematical steps. The program is formulated to undertake "elastic" as well as "non-linear" conditions in the analysis. The influence
of the "soil-pile" slip along the pile shaft and "yielding" of soil under the base are considered in order to incorporate the non-linear effects.

Some preliminary aspects of the pile behaviour have also been studied and compared with the available work in order to demonstrate the level of consistency with previous investigations, carried out by various authors.

Some examples of actual measured data have been compared with the computer results and presented in Chapter 8.

4.2. METHOD OF ANALYSIS

The pile is assumed to consist of $n$ cylindrical elements, each acted upon by a uniform shear loading $\tau$ and a circular base having a uniform vertical stress $\sigma_b$. It is assumed in the initial analysis that the pile shaft is perfectly rough but the base is perfectly smooth, i.e. no shear stress is developed at the base.

The vertical displacement of the soil adjacent to the pile element $i$ resulting from the stress $\tau_j$ on an element $j$ as shown in Figure (4.1) may be expressed as

$$\delta_{ij} = \frac{D}{E_s} I_{ij} \tau_j \quad \ldots\ldots 4.1$$

where

$$I_{ij} = \text{Displacement factor for element } i \text{ due to shear stress at element } j$$

$$D = \text{Pile shaft diameter}$$

Similarly, the vertical displacement of the soil adjacent to the pile element $i$ due to the base pressure may be expressed as

$$\delta_{ib} = \frac{D_b}{E_s} I_{ib} \sigma_b \quad \ldots\ldots 4.2$$

where

$$I_{ib} = \text{Displacement factor for element } i \text{ due to the uniform stress at the pile base } b$$

$$D_b = \text{Pile base diameter}$$
For a single pile, the soil displacement at element $i$ due to $n$ cylindrical pile elements and one base may be expressed as

$$
\delta_i = \frac{D}{E_s} \left( \sum_{j=1}^{n} I_{ij} \tau_j \right) + \left[ \begin{array}{c}
(D_b / E_s) \\
I_{ib}
\end{array} \right] \sigma_b \ldots \ldots 4.3
$$

Similarly, the expression may be written for the base displacement as

$$
\delta_b = \frac{D}{E_s} \sum_{j=1}^{n} \left( I_{bj} \tau_j \right) + \left[ \begin{array}{c}
(D_b / E_s) \\
I_{bb}
\end{array} \right] \sigma_b \ldots \ldots 4.4
$$

where

$I_{bj}$ = Displacement factor for the pile base due to the shear stress at element $j$

$I_{bb}$ = Displacement factor for the base due to the uniform stress $\sigma_b$ at the base

Combining equations (4.3) and (4.4), the soil displacements at pile nodes for a single pile (if $D = D_b$) may be expressed as

$$
\{ \delta \} = \frac{D}{E_s} \left[ I_s \right] \{ \tau \} \\
= \left[ I_{pp} \right] \{ \tau \} \ldots \ldots 4.5
$$

where,

$$
\{ \delta \} = [\delta_1, \delta_2, \ldots, \delta_n, \delta_b]^T
$$

- known as soil-displacement vector,

where, $[\delta_1, \delta_2, \ldots, \delta_b]$ are the displacements at nodes 1, 2, n, b.

$$
\{ \tau \} = [\tau_1, \tau_2, \ldots, \tau_n, \sigma_b]
$$

- known as pile stress vector,

where, $[\tau_1, \tau_2, \ldots, \tau_n, \sigma_b]$ are the stresses at nodes along the pile shaft and $\sigma_b$ is the stress at the base.

$$
[I_{pp}] = n + 1 \text{ square matrix containing soil displacement factors}
$$
Therefore, the set of equations for one pile consist of \( n \) shaft elements and one base element may be written as

\[
\delta_1 = \left[ (l_{pp})_{11} \tau_1 + (l_{pp})_{12} \tau_2 + \ldots + (l_{pp})_{1n} \tau_n + (l_{pp})_{1b} \sigma_b \right]
\]

\[
\delta_2 = \left[ (l_{pp})_{21} \tau_1 + (l_{pp})_{22} \tau_2 + \ldots + (l_{pp})_{2n} \tau_n + (l_{pp})_{2b} \sigma_b \right]
\]

\[
\delta_3 = \left[ (l_{pp})_{31} \tau_1 + (l_{pp})_{32} \tau_2 + \ldots + (l_{pp})_{3n} \tau_n + (l_{pp})_{3b} \sigma_b \right]
\]

\[
\delta_n = \left[ (l_{pp})_{n1} \tau_1 + (l_{pp})_{n2} \tau_2 + \ldots + (l_{pp})_{nn} \tau_n + (l_{pp})_{nb} \sigma_b \right]
\]

\[
\delta_b = \left[ (l_{pp})_{b1} \tau_1 + (l_{pp})_{b2} \tau_2 + \ldots + (l_{pp})_{bn} \tau_n + (l_{pp})_{bb} \sigma_b \right]
\]

Above equations may be expressed in the matrix form as

\[
\{\delta\} = \begin{bmatrix}
(l_{pp})_{11} & (l_{pp})_{12} & (l_{pp})_{13} & \cdots & (l_{pp})_{1n} & (l_{pp})_{1b} \\
(l_{pp})_{21} & (l_{pp})_{22} & (l_{pp})_{23} & \cdots & (l_{pp})_{2n} & (l_{pp})_{2b} \\
\vdots & \vdots & \vdots & \ddots & \vdots & \vdots \\
(l_{pp})_{n1} & (l_{pp})_{n2} & (l_{pp})_{n3} & \cdots & (l_{pp})_{nn} & (l_{pp})_{nb} \\
(l_{pp})_{b1} & (l_{pp})_{b2} & (l_{pp})_{b3} & \cdots & (l_{pp})_{bn} & (l_{pp})_{bb}
\end{bmatrix}
\begin{bmatrix}
\tau_1 \\
\tau_2 \\
\vdots \\
\tau_n \\
\sigma_b
\end{bmatrix}
\]

\[\ldots 4.6\]

In order to satisfy the equilibrium of forces, the total vertical applied load \( P \) is equated with stresses along the pile shaft and the base, multiplied with their respective areas, as below:

\[
\sum_{i=1}^{n} \tau_i \pi \frac{L_i}{n} + \sigma_b \pi \frac{D_b^2}{4} = P
\]

where

\[
L_i = \text{Length of pile element}
\]

\[
n = \text{Number of elements}
\]
This may also be expressed as

\[ A_{s1} \tau_1 + A_{s2} \tau_2 + \cdots + A_{s1} \tau_i + \cdots + A_{sn} \tau_n + A_b \sigma_b = P \ldots 4.7 \]

where,

\[ A_{si} = \text{Surface area of the pile element } i \]
\[ A_b = \text{Pile base area} \]
\[ L_P = \text{Length of Pile} \]
\[ P = \text{Applied load on pile} \]

Combining the above set of equations, the following equation are derived:

\[
\begin{bmatrix} (I_{pp})_{11} \tau_1 + (I_{pp})_{12} \tau_2 + (I_{pp})_{13} \tau_3 + \cdots + (I_{pp})_{nn} \tau_n + (I_{pp})_{1b} \sigma_b \\ (I_{pp})_{21} \tau_1 + (I_{pp})_{22} \tau_2 + (I_{pp})_{23} \tau_3 + \cdots + (I_{pp})_{2n} \tau_n + (I_{pp})_{2b} \sigma_b \end{bmatrix} = \delta_1
\]
\[
\begin{bmatrix} (I_{pp})_{31} \tau_1 + (I_{pp})_{32} \tau_2 + (I_{pp})_{33} \tau_3 + \cdots + (I_{pp})_{3n} \tau_n + (I_{pp})_{3b} \sigma_b \end{bmatrix} = \delta_2
\]

\[
\begin{bmatrix} (I_{pp})_{n_1} \tau_1 + (I_{pp})_{n_2} \tau_2 + (I_{pp})_{n_3} \tau_3 + \cdots + (I_{pp})_{nn} \tau_n + (I_{pp})_{n_b} \sigma_b \\ (I_{pp})_{b_1} \tau_1 + (I_{pp})_{b_2} \tau_2 + (I_{pp})_{b_3} \tau_3 + \cdots + (I_{pp})_{bn} \tau_n + (I_{pp})_{bb} \sigma_b \end{bmatrix} = \delta_n
\]

\[ A_{s1} \tau_1 + A_{s2} \tau_2 + A_{s3} \tau_3 + \cdots + A_{sn} \tau_n + A_b \sigma_b = P \ldots 4.8 \]

Above set of equations are solved to obtain the shear stresses along the pile shaft, the stress at the base and the settlement, as described in sections 4.2.1 and 4.2.2 for incompressible and compressible piles respectively.

The influence factors are determined by using the method, introduced by Poulos & Davis (1980) for the displacement caused by a point load within a "semi-infinite soil mass". In order to determine the influence of the stress around the pile element at a point in the soil medium, the method suggested is to divide the soil cylinder around the pile at different angles \( \theta \) and at different depths \( z \). The unit loads are applied at these locations and the influence of these loads at point \( i \) is determined. This is defined as the "influence factor" for the point \( i \) due to the stresses generated around the pile element. This method involves extensive analytical and numerical integration along and around the pile associated with the determination of discrete nodal flexibility coefficients from.
continuously distributed shears. Although, this is an accurate method, it is also very time consuming and makes a direct solution for the whole group difficult.

Sharnouby and Novak (1984) simplified the above method by the application of discrete point loads applied and located in such a manner that the resultant "flexibility coefficients" are almost the same as those obtained from the continuously distributed shears. The investigation indicated that the average displacement can be very nearly obtained from two equal point loads, which act on the cylinder surface at a distance of $L/4$ from the soil element ends, where $L_i = \text{length of element, and which are related to the reference node } O \text{ by angles } \theta = 80^\circ \text{ or } \alpha = 40^\circ$ (see Figure 4.2). Thus the soil "flexibility coefficient $I_{ij}$" is equal to the average of the vertical displacement at point $i$ due to two equivalent unit loads acting on element $j$ as indicated in Figure 4.2. This definition holds good for the elements of one pile as well as for the elements of different piles.

A similar simplification is also suggested by Sharnouby and Novak (1984) for the pile base. A vertical unit load uniformly distributed over the base produces non uniform displacements, with the value at the base center being $\delta_{b_i}$. Almost the same displacement $\delta_b$ can be generated at the center of the base by the unit load concentrated as a point load at a distance from the base center equal to $r_b = D_b / \pi$, where $D_b = \text{base diameter}$. To account for the rigidity of the base, the displacement $\delta_{b_i}$ is corrected by a factor of $\pi/4$ to yield the expected base flexibility, $I_{bb} = \delta_b$. The distinct advantage of this equivalent point load method is the considerable reduction in computing time.

The above simplified method has been adopted to carry out the analysis and to reduce the computer run time. The soil cylinder attached to the pile is divided into soil tributary elements and pile elements as indicated in Figure (4.2). Therefore, the displacement of any node $i$ due to the element $j$ is found by following steps:

- applying two equivalent point loads on element $j$ at a distance of $L/4$ from the soil element ends

- obtaining the displacement at the pile node $i$ because of these two applied loads

- taking the average of these two displacement values as the "soil flexibility coefficient" (influence factor) related to pile node $i$ because of element $j$. This is denoted as $I_{ij}$. Thus the soil flexibility $I_{ij}$ is equal to the average of the vertical displacements at point $O$ on the element $i$ due to two equivalent point load acting on element $j$ at a distance of $L/4$ from the soil element ends.
The above approach is used to establish the influence factors for the elements of one pile as well as for elements of different piles.

An equivalent point load is applied at the center of the base to determine the influence of the base on pile elements as well as on the other bases to obtain their respective influence factors $I_{ib} \& I_{bb}$.

### 4.2.1. INCOMPRESSIBLE PILES

In the case of an incompressible pile, the displacements at all nodes are the same for a single pile. Therefore, to incorporate this effect, the displacements are expressed as:

$$\delta = \delta_1 = \delta_2 = \ldots \ldots = \delta_n = \delta_b \quad \ldots \ldots 4.9$$

where

$$\delta = \text{displacement of the single incompressible pile}$$

Therefore, the set of equations (4.8) may be expressed as

$$
\begin{bmatrix}
(I_{pp})_{11} \tau_1 + (I_{pp})_{12} \tau_2 + (I_{pp})_{13} \tau_3 + \ldots + (I_{pp})_{1n} \tau_n + (I_{pp})_{1b} \sigma_b \\
(I_{pp})_{21} \tau_1 + (I_{pp})_{22} \tau_2 + (I_{pp})_{23} \tau_3 + \ldots + (I_{pp})_{2n} \tau_n + (I_{pp})_{2b} \sigma_b \\
(I_{pp})_{n1} \tau_1 + (I_{pp})_{n2} \tau_2 + (I_{pp})_{n3} \tau_3 + \ldots + (I_{pp})_{nn} \tau_n + (I_{pp})_{nb} \sigma_b \\
(I_{pp})_{b1} \tau_1 + (I_{pp})_{b2} \tau_2 + (I_{pp})_{b3} \tau_3 + \ldots + (I_{pp})_{bn} \tau_n + (I_{pp})_{bb} \sigma_b
\end{bmatrix} = \delta
$$

$$
\begin{bmatrix}
A_{s1} \tau_1 + A_{s2} \tau_2 + A_{s3} \tau_3 + \ldots + A_{sn} \tau_n + A_b \sigma_b
\end{bmatrix} = P
$$
Above equations may further be modified as:

\[
\begin{bmatrix}
(I_{pp})_{11} \tau_1 + (I_{pp})_{12} \tau_2 + (I_{pp})_{13} \tau_3 + \ldots + (I_{pp})_{1n} \tau_n + (I_{pp})_{1b} \sigma_b \\
(I_{pp})_{21} \tau_1 + (I_{pp})_{22} \tau_2 + (I_{pp})_{23} \tau_3 + \ldots + (I_{pp})_{2n} \tau_n + (I_{pp})_{2b} \sigma_b \\
\ldots \\
(I_{pp})_{n1} \tau_1 + (I_{pp})_{n2} \tau_2 + (I_{pp})_{n3} \tau_3 + \ldots + (I_{pp})_{nn} \tau_n + (I_{pp})_{nb} \sigma_b \\
(I_{pp})_{b1} \tau_1 + (I_{pp})_{b2} \tau_2 + (I_{pp})_{b3} \tau_3 + \ldots + (I_{pp})_{bn} \tau_n + (I_{pp})_{bb} \sigma_b \\
\end{bmatrix} - \delta = 0
\]

\[
\begin{bmatrix}
A_{s1} \tau_1 + A_{s2} \tau_2 + A_{s3} \tau_3 + \ldots + A_{sn} \tau_n + A_b \sigma_b \\
\end{bmatrix} = P
\]

Above set of equations may be expressed in the matrix form as below:

\[
\begin{bmatrix}
(I_{pp})_{11} & (I_{pp})_{12} & (I_{pp})_{13} & \ldots & (I_{pp})_{1n} & (I_{pp})_{1b} \\
(I_{pp})_{21} & (I_{pp})_{22} & (I_{pp})_{23} & \ldots & (I_{pp})_{2n} & (I_{pp})_{2b} \\
\vdots & \vdots & \vdots & \ddots & \vdots & \vdots \\
(I_{pp})_{n1} & (I_{pp})_{n2} & (I_{pp})_{n3} & \ldots & (I_{pp})_{nn} & (I_{pp})_{nb} \\
(I_{pp})_{b1} & (I_{pp})_{b2} & (I_{pp})_{b3} & \ldots & (I_{pp})_{bn} & (I_{pp})_{bb} \\
\end{bmatrix}
\begin{bmatrix}
\tau_1 \\
\tau_2 \\
\vdots \\
\tau_n \\
\sigma_b \\
\delta \\
\end{bmatrix} =
\begin{bmatrix}
0 \\
0 \\
\vdots \\
0 \\
P \\
\end{bmatrix}
\]

These equations are solved for the shear stresses along the pile shaft, base pressure and the pile displacement for an incompressible pile.

4.2.2. **Compressible Piles**

In order to incorporate the pile compressibility, a relative displacement approach is adopted to derive the displacement equations. Only pure axial compression of piles is considered in the analysis. The compression of each pile element with respect to the node above, resulting from the stresses acting on the pile zone and the base, is evaluated and
the equations are modified to incorporate this effect. The element deformations are added together to obtain the pile node displacement with the top of the pile as datum.

For a pile divided into \( n \) number of nodes, the displacements of various nodes may be expressed as

\[
\delta_1 = \left( I_{pp} \right)_{11} \tau_1 + \left( I_{pp} \right)_{12} \tau_2 + \ldots + \left( I_{pp} \right)_{1n} \sigma_n
\]

\[
\delta_2 = \left( I_{pp} \right)_{21} \tau_1 + \left( I_{pp} \right)_{22} \tau_2 + \ldots + \left( I_{pp} \right)_{2n} \sigma_n + \frac{P_1 L_1}{E_p A_p}
\]

\[
\delta_i = \left( I_{pp} \right)_{i1} \tau_1 + \left( I_{pp} \right)_{i2} \tau_2 + \ldots + \left( I_{pp} \right)_{in} \sigma_n + \frac{P_i L_i}{E_p A_p} + \ldots + \frac{P_{i-1} L_{i-1}}{E_p A_p}
\]

\[
\delta_n = \left( I_{pp} \right)_{n1} \tau_1 + \left( I_{pp} \right)_{n2} \tau_2 + \ldots + \left( I_{pp} \right)_{nn} \sigma_n + \frac{P_n L_n}{E_p A_p} + \ldots + \frac{P_{n-1} L_{n-1}}{E_p A_p} + \frac{P_b L_{n-1}}{E_p A_p}
\]

where,

\( P_1, P_2, \ldots, P_{n-1} \) = Axial forces developed on pile elements due to shaft shear stress, causing structural compression

\( P_b \) = Axial force at the pile base due to the stress at the base

\( L_1, L_2, \ldots, L_{n-1} \) = Length of pile elements

The influence matrix as indicated in equation (4.5) is modified by including the "pile compression factors" as below:

\[
\{\delta\} = \left[ I_{pp} \right] \{\tau_j\}
\]

.....4.11

where,

\[
\left[ I_{pp} \right] = \text{Matrix with the modified influence factors for pile compressibility}
\]

These equations are solved for the shear stresses along the pile shaft, base pressure and the pile displacement for a compressible pile.
4.2.3. **Free field conditions**

In a reactive soil environment, the clay soils subject to a change in volume to varying degrees in response to changes in their moisture contents as discussed in section 3.2.4 of Chapter 3. This behaviour leads to a cyclic surface movements as indicated in Figure (4.3a). The magnitude of these cyclic movements decreases with depth down to a level below which no seasonal moisture change occurs and hence no volume change takes place. This depth is called as depth of seasonal movement. The factors affecting the magnitude of the seasonal heave of clay soils in the open (i.e. outside the influence of ground covers and buildings) are already discussed in section 3.2.4 of Chapter 3. A pile constructed in this ground conditions subject to a force along its shaft due to the vertical movement of the soils around it. The upward movement of soil due to the swelling effect exerts an upward force along the pile shaft, whereas the shrinking soil induces a downward force to the pile. The influence of this phenomenon on the pile behaviour is dependent on the initial moisture condition of the ground.

If a pile is installed in a drier period, it may be subject to an uplift force due to the swelling of soil around the pile shaft. The upward movement of soil in the process of swelling exerts an upward force on the pile due to the adhesion between the pile and the surrounding soil. In the same manner, a pile, installed in a wet ground condition, may experience a downdrag force due to the shrinking of soil. The magnitude of ground movement generally reduces with depth and beyond a certain point, it becomes zero, as discussed above. The movement of the ground along the pile shaft may be defined as a profile with the maximum value at the top and reducing with depth. In the analysis, a linear profile is adopted to incorporate the effect of ground movement, as indicated in Figure (4.3b).

The magnitude of the ground movement \((S_{p1}, S_{p2}, \ldots S_{pn})\) along the pile shaft at various nodes is determined and indicated as \(S_r\) in equation (4.12). The values in this vector \(S_r\) are positive for shrinking soils and negative for swelling soils.

The equation (4.5) may be modified, in order to incorporate the effect of ground movement along the pile shaft as below:

\[
\{ \delta \} = + \frac{D}{E_s} \left[ I_s \right] \{ \tau \} + \{ S_v \} \quad \ldots \ldots 4.12
\]
where,

\[
\{S_V\} = \begin{bmatrix} S_{p1} \\ S_{p2} \\ \vdots \\ S_{pn} \end{bmatrix}^T
\]

= Vector of the ground movement along the pile shaft
  (positive for shrinking soils and negative for swelling soils)

For a single pile, divided into \( n \) cylindrical elements and one base, the equation (4.12) may be expressed as

\[
\begin{bmatrix}
(I_{pp})_{11} \tau_1 + (I_{pp})_{12} \tau_2 + (I_{pp})_{13} \tau_3 + \ldots + (I_{pp})_{1n} \tau_n + (I_{pp})_{1b} \sigma_b \\
(I_{pp})_{21} \tau_1 + (I_{pp})_{22} \tau_2 + (I_{pp})_{23} \tau_3 + \ldots + (I_{pp})_{2n} \tau_n + (I_{pp})_{2b} \sigma_b \\
\vdots
\end{bmatrix} - \delta_1 = -S_{p1}
\]

\[
\begin{bmatrix}
(I_{pp})_{31} \tau_1 + (I_{pp})_{32} \tau_2 + (I_{pp})_{33} \tau_3 + \ldots + (I_{pp})_{3n} \tau_n + (I_{pp})_{3b} \sigma_b \\
\vdots
\end{bmatrix} - \delta_2 = -S_{p2}
\]

\[
\begin{bmatrix}
(I_{pp})_{n1} \tau_1 + (I_{pp})_{n2} \tau_2 + (I_{pp})_{n3} \tau_3 + \ldots + (I_{pp})_{nn} \tau_n + (I_{pp})_{nb} \sigma_b \\
(I_{pp})_{b1} \tau_1 + (I_{pp})_{b2} \tau_2 + (I_{pp})_{b3} \tau_3 + \ldots + (I_{pp})_{bn} \tau_n + (I_{pp})_{bb} \sigma_b \\
A_{11} \tau_1 + A_{12} \tau_2 + A_{13} \tau_3 + \ldots + A_{1n} \tau_n + A_b \sigma_b
\end{bmatrix} = P
\]

\[
\delta_n = -S_{pn}
\]

\[
\delta_b = -S_{p(n+1)}
\]

The above equations are solved for the compressible and incompressible pile conditions as described in previous sections.

4.2.4. Other Conditions

The analyses are carried out by incorporating the following conditions:

4.2.4.1. Elastic Condition

Under elastic conditions, the pile surface is assumed to be perfectly rough and the soil is an ideal elastic material, which is capable of resisting any shear stress which may develop between the pile and the soil. In other words, no slip at the soil-pile interface is allowed under any stress condition. It is also assumed that the bearing capacity of soil at the pile base is infinite and allows no yielding of the soil.
4.2.4.2. **Non-linear Conditions**

As discussed in section 4.2.4.1, the elastic condition considers the pile-soil capacities as infinite, but in a real situation, the soil has a finite strength value. Whenever the shear stress along the pile shaft exceeds the adhesion capacity, slip occurs between the soil and the pile. Similarly, the soil at the pile base yields locally if the base stress exceeds its limiting capacity. These effects are incorporated into the analysis, as described in the following sections.

4.2.4.2.1. **Soil-Pile Slip**

Whenever the shear stress along any pile element exceeds the soil-pile adhesion capacity, a slip occurs at the soil-pile interface. The elastic conditions are no longer valid beyond this point. In order to incorporate this effect, the equation for the pile element, which has slipped, is modified by substituting the shear stress with the maximum adhesive strength value as below:

\[ \tau_{\text{max}} = C_a \] ...

where,

- \( C_a \) - maximum adhesion strength between the pile and the soil
- \( \tau_{\text{max}} \) - maximum shear stress along the pile shaft

The stresses are redistributed between the remaining elastic elements and the bases, which are still in the elastic condition. This process is repeated until equilibrium is achieved or all the elements in the pile, including the base, have failed.

4.2.4.2.2. **Local Soil Yield Under Pile Base**

The soil under the pile base yields as the stress exceeds its limiting capacity. For piles in cohesive soils, under undrained conditions, equations for those pile bases are modified by substituting base stress with the limiting value as below:

\[ \sigma_{b(\text{max})} = cN_c \] ...

where,

- \( \sigma_{b(\text{max})} \) - Maximum stress at pile base
- \( c \) - Undrained cohesion
- \( N_c \) - Bearing Capacity Factor
The stresses are redistributed between the remaining elastic elements, which are still in the elastic condition.

The procedures described in sections 4.2.4.2.1 and 4.2.4.2.2 are processed simultaneously and are carried out until equilibrium is achieved or all the elements in the pile, including the base, are failed.

In case of total pile failure, the ultimate load $P_{ult}$ of the pile is reached, where

$$P_{ult} = \pi D L_P C_a + \left(\frac{\pi D^2}{4}\right) c N_c$$

.....4.16

where,

$L_P$ - Length of pile

$D_b$ - Pile base diameter

## 4.3. DEVELOPMENT OF COMPUTER MODEL

A computer model *PAES* (*Pile Analysis in Expansive Soils*) has been developed in order to carry out the necessary analysis. The mathematical equations are formulated by using FORTRAN 77. The various stages of the computer programming are outlined below:

### 1. INPUT DATA:

The pile geometry and the soil conditions are read by the program. This block is created to input the following data:

- Number of Piles
- Geometrical properties of Piles
- Number of piles and its respective arrangement
- Material properties of piles and soil such as Poisson's ratio & modulus
- Element numbers & its sizes
- Applied Load on Piles
- Vertical soil movement due to swelling or shrinking of soil
- Required analysis type i.e. Elastic or Non-linear
If elastic analysis is warranted
- No soil-pile slip & yielding of soil under the pile base are considered

If non-linear analysis is required
- Soil-pile slip & yielding of soil under the pile base are allowed at specific values.

2. **Generating Vertical Soil Movement:**

The soil movement along the pile shaft is generated at various nodes of the pile and is stored in an array for later application. These values are indicated in equation (4.12) as \( \{S_v\} \).

3. **Pile Influence Factors:**

These factors are determined to form the matrix as shown in equation (4.6).

4. **Formulating Applied Load Equation:**

The shaft surface and base areas are determined to formulate the equation (4.7).

5. **Introducing Soil Movement Along The Pile Shaft:**

The soil movements along the pile shaft as determined in step 2 are introduced to each pile node as described in equations (4.12) and (4.13).

6. **Modifying The Matrix For Non-Linear Conditions:**

This section of the program is activated after the first run and undertakes the following two processes:

**Soil-Pile Slip:**

In the event of any shear stress along the pile shaft exceeding the specified adhesion limit, these shear stresses are processed as described in section 4.2.4.2.1.
SOIL YIELD UNDER PILE BASE:

If the stress at any pile base exceeds the specified limiting value, this section of the program will process those stresses as described in section 4.2.4.2.2.

In case of shrinking soils, the slip for the negative shear stresses are processed after the full processing of the slip due to positive shear stresses and the soil yielding at the base. The processing of positive shear stress will cause a slip and will result in an additional settlement. This may change some negative shear stress (downdrag) into positive. Then the processing of negative shear stress is undertaken by the analysis.

In swelling soils, the negative shear stresses are processed totally before the positive shear stress (upward force generated by the swelling soils). The processing of negative shear stress will reduce the positive shear stress along the shaft in the swelling zone due to the slip in the stable zone, in case of pile in tension. Then the processing of positive shear stress is undertaken by the analysis.

7. SOLVING THE MAIN MATRIX:

The set of equations shown in (4.13) is solved for the shear stresses $\{\tau\}$, base stress $(\sigma_b)$ and the displacements $\{\delta\}$ by using the Gaussian Elimination Method.

8. SCREENING STAGE:

This part of the program is activated if a non-linear analysis is required. The stresses are screened in two steps to pick up the following conditions:

- Positive shear stresses exceeding the specified stress limit
- Stress at the pile base exceeding the specified limit

The program run is diverted to step-6, if any stress is screened in the above category.

After processing the above conditions for all elements, the following condition is screened:

- Negative shear stress exceeding the specified limit

The program run is diverted to step-6 to solve for the above condition, if any pile element screened is in this category. The whole process is repeated until no stress falls into this category.
9. **Producing the Results:**

The pile displacements and the shear stresses are written to an output file in the required format.

4.4. **Some Preliminary Aspects of Pile Behaviour**

In this section, the output from the computer model is compared with the available works, carried out by various authors, in order to demonstrate the level of consistency.

The settlement of pile for various loads and $L_p/D$ is compared with the settlement calculated from Poulos and Davis (1980). The comparison is presented in Table 4.1. The comparison indicates an excellent agreement.

<table>
<thead>
<tr>
<th>$L_p/D$</th>
<th>Applied Load $P$ (KN)</th>
<th>$E_s$ (kPa)</th>
<th>$\nu_s$</th>
<th>Settlement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Poulos &amp; Davis (1980)</td>
</tr>
<tr>
<td>20</td>
<td>3000</td>
<td>50000</td>
<td>0.5</td>
<td>0.0054</td>
</tr>
<tr>
<td>40</td>
<td>5000</td>
<td>40000</td>
<td>0.5</td>
<td>0.0065</td>
</tr>
<tr>
<td>60</td>
<td>9000</td>
<td>40000</td>
<td>0.3</td>
<td>0.0083</td>
</tr>
<tr>
<td>80</td>
<td>12000</td>
<td>30000</td>
<td>0.3</td>
<td>0.011</td>
</tr>
<tr>
<td>100</td>
<td>15000</td>
<td>20000</td>
<td>0.3</td>
<td>0.018</td>
</tr>
</tbody>
</table>

A pile in swelling soils is analysed for the conditions, indicated in Figures (4.4), (4.5) & (4.6) and the results are compared with Poulos (1973). The comparison generally indicates similar trend and the settlement values agree reasonably well. The difference (see Figures 4.4 & 4.6) may be due to the limited number of elements used in the earlier solutions, as well as the procedure adopted to incorporate the non-linear effects such as limitation on the shaft and the base stresses. These analytical features are discussed in greater detail in the following chapters.
4.5. **PILE BEHAVIOUR IN SHRINKING SOILS**

In this section, the analysis is carried out for a pile, experiencing a downdrag force due to the shrinking soils. The pile characteristics are analysed for elastic as well as for non-linear conditions. The influence of soil shrinkage on the settlement and on the load distribution pattern along the pile shaft has been presented and discussed. Effects of parameters such as \( L_D / D \) ratios, soil modulus \( (E_s) \), Poisson's ratio \( (\nu_s) \) and the pile compressibility \( (E_p) \) have also been analysed in detail. In the analysis, the pile diameter is adopted as one metre unless otherwise mentioned.

4.5.1. **CHARACTERISTICS OF BEHAVIOUR**

4.5.1.1. **ELASTIC AND NON-LINEAR CHARACTERISTICS**

As discussed earlier, the elastic analysis considers the shaft to be capable of resisting any shear stress which may develop between the pile and the soil and the soil under the base can sustain any stress without failure. In contrast, the non-linear condition incorporates slip between the pile & the soil beyond a limiting value and yielding of soil under the pile base exceeding the bearing stress, as described in sections 4.2.4.1 & 4.2.4.2, respectively.

In order to indicate the influence of non-linear effects on the pile behaviour, two piles of the \( L_D / D \) ratios as 15 and 25 are analysed and the results are presented in Figures (4.7) and (4.8) respectively. It can be seen from these figures that under the application of lower loads, the elastic condition indicates a higher settlement compared to the non-linear condition, whereas, at higher loads the elastic settlement is lower than the non-linear settlement. Under elastic conditions, as no slip takes place between the pile & the soil, the movement of the ground forces the pile shaft to move with the same magnitude of its movement. As a result of this, a very high magnitude of force is imposed along the pile shaft, causing high settlement compared to the non-linear condition, as the non-linear condition allows a slip and causes lesser transfer of force to the pile.

Under higher applied loads, the elastic settlement is less compared to the non-linear settlement, because no slip occurs in the lower portion of the pile, which is in the stable zone and has to resist higher loads. In case of the non-linear condition, due to slip in the lower portion of the pile shaft, which is in the stable zone, the pile experiences higher settlement compared to elastic condition.

In real situations, the ultimate capacity of a pile is finite and the non-linear conditions incorporate this behaviour by accounting for the effect of slip along the pile shaft and the
soil yielding under the pile base. Elastic analysis may be suitable for a pile where no ground movement occurs \((S_p = 0)\), but may provide highly misleading results in the case of a shrinking soil condition, as discussed above. Therefore, a non-linear approach should be adopted to analyse a pile in a shrinking soil condition. In later sections, most of the analyses are undertaken incorporating the effect of non-linear conditions, unless otherwise stated.

4.5.1.2. **Load Distribution Along Pile**

The downward ground movement along the pile shaft induces additional force to the pile due to the property of adhesion between the soil and the pile. The force is induced to the pile by means of the shear stress along the pile shaft in the soil shrinking zone as shown in Figures (4.10) and (4.11). The magnitude of this force is dependent on the “soil-pile adhesion” properties as well as on the pile geometry. It can be seen from Figures (4.10) & (4.11) that the shear stress along the pile shaft is significantly influenced by the ground movement \((S_p)\). This phenomenon alters the stress profile along the pile shaft and induces downward force up to a depth, known as “neutral plane”, as indicated in these figures. The neutral plane may be defined as the plane at which the shear stress along the pile shaft changes its sign i.e. the plane up to which the downdrag force acts and beyond which the force along the pile shaft changes to a resistance.

The neutral plane moves upward as the load on the pile increases (Figures 4.10 & 4.11) and reaches at the top, when the pile is loaded to its ultimate capacity. These Figures (4.10 & 4.11) indicate the movement of the neutral plane under the loads, varying from 0 to 0.75 \(P_{ult}\). In the soil shrinking zone, the shear stress at any point along the pile shaft changes its sign from negative (downward) to positive (upwards) as the settlement of that specific point becomes greater than the magnitude of the ground movement. Therefore, at higher imposed loads, the pile experiences higher settlement due to the ground movement and as a result of that the neutral plane moves upwards until the pile reaches its ultimate capacity. Under the ultimate load, all shear stresses along the pile shaft become positive i.e. no downdrag force will be present along the shaft, although the settlement will be large.

The position of the neutral plane indicates the portion of pile shaft in the stable (resistive) zone, although it moves with the magnitude and the direction of the applied force. For example, a pile of \(L_p/D = 25\) (Figure 4.11), which is subjected to a downward ground extending to a depth \((S_v)\) of 5m, the neutral plane moves from 5m to 3m, under the application of compressive force from 0 to 0.75 \(P_{ult}\). The settlements of the pile under these applied loads are 0.4 mm and 6mm respectively. The settlement at \(P = 0.75 P_{ult}\)
causes some downdrag force (negative shear stress) to change to positive, and as a result of this, the neutral plane moves upwards. Similarly, for a relatively shorter pile \((L_p/D=15)\), which is subjected to the same magnitude of ground movement (Figure 4.10), the neutral plane moves from 5m to 2m, under the application of compressive force from 0 to 0.75\(P_{ult}\) and the corresponding settlements are 1mm and 28 mm (Figure 4.7), respectively. The shorter pile experiences very high settlement (Figure 4.7) under higher applied load and is unable to stabilise if further increases occur in the \(S_p\) value.

### 4.5.1.3. Settlement Analysis

The settlement of a pile increases, when subjected to downward ground movement due to shrinking soils (see Figures 4.17 to 4.20). Under the elastic condition, the settlement increases with the increase in the magnitude of the ground movement, but in the case of non-linear conditions the settlement of the pile increases only up to a certain extent. Beyond that, any further increase in the soil movement causes no additional increase in the pile settlement as a slip occurs between the pile and the soil, resulting in no transfer of any further load to the pile. In other words, any further increment in the ground movement will result in a slip at the soil-pile interface, and therefore will cause no further increase in the pile settlement.

It can be observed from Figure (4.18) that the increase in settlement of a shorter pile \((L_p/D=15; \; K_p=10^3)\), subjected to a 50\(mm\) ground movement (i.e. \(S_p=0.05m\)), is approximately 2.2 times of the settlement in a flat ground condition \((S_p=0)\). Under the same ground movement condition, a longer pile \((L_p/D=50, \; K_p=10^3)\), experiences approximately 1.2 times the settlement in a flat ground condition. It can also be observed that the settlement of the larger pile \((L_p/D=50)\) stabilises at \(S_p=0.05m\), whereas the shorter pile \((L_p/D=15)\) still experiences settlement with the increase in the ground movement. The influence of \(L_p/D\) ratio on the pile settlement is illustrated in Figures (4.12) and (4.13), under various load conditions. The increase in settlement is also dependent on the extent of the ground movement. The influence of the extent of ground movement \((S_e)\) along the pile shaft is indicated in Figure (4.14). It should be noted that a pile may also experience a downdrag force due to consolidation of soil around the shaft, and may influence the pile behaviour in a similar manner.

In other words, under the same ground movement, a longer pile (i.e. higher \(L_p/D\)) is less influenced because of its longer shaft in the stable portion of the soil compared to a shorter pile. Although a pile experiences high settlement due to the ground movement, but
fails at its original ultimate capacity. Therefore, the ground movement may be regarded as a settlement problem rather than a capacity problem.

4.5.2. **Extension of Results**

In this section, the influence of various parameters on the pile performance is analysed and discussed.

4.5.2.1. **Influence of Soil Modulus**

The settlement behaviour of piles of \( L_p/D = 15 \) and 50 undergoing the same magnitude of ground movement is analysed and presented in Figures (4.21) and (4.22). Although the settlement reduces with the increase in soil modulus, it should be noted that at any particular modulus value, the increase in settlement of a shorter pile due to the increase in the ground movement is higher in comparison to a longer pile. For \( E_s = 20,000 \text{ kPa} \), the increase in settlement of a shorter pile (\( L_p/D=15 \)) is 54% due to the increase in \( S_p \) from 0 to 0.05m, whereas for a similar magnitude of ground movement, a longer pile (\( L_p/D=50 \)) experiences 10% increase in settlement.

4.5.2.2. **Influence of Poisson's Ratio**

It can be seen from Figures (4.23) and (4.24) that the settlement increases with an increase in the Poisson's ratio. The increase in settlement for a pile of \( L_p/D = 15 \) is approximately 17% for the Poisson's ratio (\( v_p \)) varying from 0 to 0.5 for the \( S_p = 0.1m \) (Figure 4.23), whereas a longer pile (\( L_p/D=50 \)) experiences a 38% increase under the same variation (Figure 4.24) and under the application of half of their ultimate capacities.

4.5.2.3. **Influence of Pile Compressibility**

In addition to externally applied load, the downdrag force caused by the downward moving soil also causes structural compression of the pile. The settlements of piles for \( L_p/D = 15 \) & 25 are analysed for \( K_p = 10^2, 10^3 \) & \( 10^5 \) and are presented in Figures (4.15) & (4.16). The settlement is higher in case of a compressible pile (\( K_p=10^2 \)) compared to a rigid pile (\( K_p=10^5 \)), under the same load conditions. The structural compression of a longer pile (\( L_p/D = 25 \)) is higher than the shorter pile (\( L_p/D = 15 \)).

Under elastic conditions, the structural compression is much higher as the ground movement induces a higher load in the pile (Figure 4.17).
4.6. **PILE BEHAVIOUR IN SWELLING SOILS**

In this section, the pile behaviour is analysed and presented including the effect of the upward ground movement caused by the swelling of soils. The pile characteristics are analysed for elastic as well as for non-linear conditions. The influence of swelling soils on the settlement and the load distribution pattern along the pile shaft is presented and discussed. The effects of parameters such as \( \frac{L_p}{D} \) ratio, soil modulus \( (E_s) \) and the pile compressibility \( (E_p) \) are also analysed and discussed in detail.

4.6.1. **Characteristics Of Behaviour**

4.6.1.1. **Elastic And Non Linear Characteristics**

It can be seen from Figures (4.25) and (4.26) that under the application of lower loads, the pile experiences an upward displacement due to the upward ground movement around its shaft and the upward displacement reduces with the increases in the applied load (downward). Under elastic conditions, the ground movement induces a very high magnitude of force, as no slip is allowed between the soil and the pile, whereas the non-linear condition incorporates a slip and limits the transfer of force from the soil to the pile. Therefore, the elastic condition indicates higher upward displacement in comparison with the non-linear condition. As the load on the pile is increased, the uplift force is counteracted by the applied load and the upward displacement is reduced.

In case of elastic conditions, the upward displacement increases with an increase in the ground movement, whereas in non-linear condition, the upward displacement stabilises beyond a limit due to the slip at the pile-soil interface. In real situations, the swelling soil will only be capable of transferring force to the pile shaft while the limiting stress is not exceeded. Beyond this, any further ground movement will cause a slip and no further load will be transferred to the pile. This phenomenon is incorporated in non-linear condition. Therefore, to analyse the actual foundation performance in swelling soils, a non-linear approach should be adopted. In later sections, the analyses will incorporate the effects of non-linear conditions, unless otherwise stated.

4.6.1.2. **Load Distribution Along Pile**

The upward ground movement along the pile shaft alters the stress distribution pattern along the pile shaft. The extent of influence is dependent on the "pile-soil adhesion" properties and the geometry. Higher adhesion will generate higher upward force to the pile.
Under zero applied load, the swelling soil induces upward shear stress along the pile shaft up to the neutral plane as indicated in Figure (4.27). Below this plane, the stress acts downward along the shaft and counteracts the upward force exerted by the swelling soils. As a result of this, the pile experiences upward displacement.

Under the application of compressive load, the upward swelling force is counteracted and the settlement occurs in the downward direction. Due to this, the neutral plane moves downward and reaches the pile base, prior to the failure. Conversely, the application of tensile load on the pile adds to the swelling force and as a result of that (see Figure 4.28) the pile experiences higher upward movement compared to the normal ground condition ($S_p = 0$). In this process, the neutral plane moves upward and reaches the top at failure. The understanding of the neutral plane at any stage of the pile behaviour is important in order to determine the portion of the pile in the stable zone.

The load distribution along the pile shaft for $L_p/D = 15$ & 25 is indicated in Figures (4.27) and (4.28). The maximum load induced along the pile shaft, can be seen in these figures. Under zero applied load (ie $P=0$), the pile is totally in tension and experiences an upward displacement due to the ground movement. In order to counteract the upward pile displacement, the required axial force is approximately 45% of the maximum load generated along the pile shaft. Figures (4.25) and (4.27) may be used to estimate the axial force required to counter-balance the upward pile movement and the maximum force generated along the pile shaft, respectively.

4.6.1.3. Settlement Analysis

The settlement of piles in swelling soils is dependent on the magnitude of the applied load and its counteracting effect to the upward force caused by the ground movement. It can be seen in Figures (4.25) and (4.26) that under no load condition, the pile is totally under tension and experiences upward displacement. The upward displacement is suppressed and becomes downward by the application of downward applied force. Beyond a certain point, further increases in the ground movement causes no influence on the pile displacement due to the occurrence of slip between the pile and the soil (Figures 4.29 & 4.30).

The upward movement of the pile under the application of a tensile force is critical as the swelling force and applied load act in the same direction. This phenomenon causes a very high upward displacement (Figures 4.31 & 4.32) compared to the pile under normal ground condition ($S_p = 0$).
Longer piles (higher $L_p/D$ ratios) are less influenced by swelling soils under a similar load condition because of their longer portion of the shaft being in the stable soil. It can be seen from Figures (4.33) and (4.35) that the upward displacement of a pile of $L_p/D = 15$ is 1.6 times higher than for a pile of $L_p/D = 25$, under a zero load condition (see Figures 4.31 and 4.32).

Although the settlement of piles is influenced by the ground movement, the failure of a pile takes place at its original ultimate capacity (Figures 4.31 & 4.32), assuming that the limiting shaft base resistances are not influenced by the action of the ground movements.

Therefore, a pile in swelling soil will experience a reduction in settlement under the application of compressive force and will undergo an increase in upward displacement under the application of tensile force.

### 4.6.2. Extension Of Results

#### 4.6.2.1. Influence Of Soil Modulus

The settlement behaviour of piles subjected to the same magnitude of ground movement is analysed and presented in Figures (4.37) & (4.38). It can be seen that the shorter pile is more sensitive to the soil modulus compared to the longer piles. As expected, the upward displacement of piles reduces with an increase in the soil modulus.

#### 4.6.2.2. Influence Of Pile Compressibility

In swelling soils, the dimensionless pile compressibility ($K_p$) is critical in the case of the tensile force application. It can be seen from Figures (4.33) to (4.36) that the upward displacement increases with an increase in the pile compressibility. This occurs because of the elongation of the pile material due to the upward swelling force, in addition to the applied tensile force. The increase in the upward displacement for a pile of $L_p/D = 50$ is approximately 35% due to the pile compressibility ($K_p$) changing from $10^5$ to $10^2$ (Figure 4.36). The elastic solution indicate higher upward displacement, because it imposes higher upward force along the pile shaft.

### 4.7. Summary

The vertical movement of the ground around the pile shaft induces additional forces to the pile. The direction of the induced force along the pile shaft depends on the direction of
the ground movement. A pile installed in a wet ground condition, experiences a downward ground movement around its periphery due to the shrinking of soils, caused by the reduction of moisture content. This induces a downdrag force on the pile. Conversely, a pile constructed in soil in a dry state, experiences an upward ground movement due to the swelling of soil, caused by the increase in the moisture content. This phenomenon induces an upward force to the pile. These induced forces influence the pile performance quite significantly. The assessment of the pile behaviour in these situations, requires attention to achieve an effective and safe foundation performance.

The method of analysis influences the accuracy of result to a great extent. Therefore, it is necessary to understand the implications of using various conditions of analysis and their adequacy for the purpose. Elastic analysis assumes no slip between the soil and the pile and also considers no bearing failure of the soil at the pile base. As a result, the movement of the ground may transfer a very high magnitude of force along the pile shaft, in other words the adhesion capacity between the soil and the pile is considered as infinite. This may also give very high pile displacement. However in real cases, the adhesion at the soil-pile interface is finite and the shear stress along the pile shaft exceeding a specific value will cause an interface slip and no further load is transferred with a further increase in the ground movement. Similarly, beyond a specified limit, the stress developed at the pile base will cause yielding of soil locally. Non-linear analysis includes the effect of these limiting stress conditions. Such limiting stresses may be derived from the pile load test results or from conventional methods of pile capacity assessment. Although the elastic analysis may be adequate in the case of a pile in normal ground conditions, it may provide highly misleading results in the case of swelling or shrinking soils. Therefore, it is necessary to adopt a non-linear approach to undertake the analysis of piles subjected to ground movement.

In shrinking soils, the behaviour of pile, under the application of compressive force, is critical, as the applied load and the force induced by the ground movement act in the same direction. As a result, the pile may experience a large settlement, compared to a pile in the normal ground condition. Therefore, careful attention is required in assessing the settlement of a pile in these situations. Piles under the application of tensile force are not critical, as the applied load and the shrinking force counteract each other and reduce the possibility of adverse effects.

In swelling soils, the pile is critical under the application of tensile force, as the applied load and the swelling force act in the same direction. Due to this, the resultant upward displacement is high in comparison to a pile in normal ground conditions (i.e. no swelling). In the case of applied compressive force, the pile settlement is not critical,
because of the counteracting effect of the applied load and the swelling force. It is worthwhile noting that an unloaded (or lightly loaded) pile may experience an upward displacement due the swelling effect and may cause an adverse effect to the superstructure. Therefore, in a reactive soil environment, piles should be loaded enough to counteract these swelling forces to restrict the upward displacement. The analysis indicates that a compressive force equivalent to 45% of the maximum force generated by the swelling soil on an unloaded pile is sufficient to restrict the upward movement of the pile. This criterion should be addressed at the time of construction, when the piles are not fully loaded and are subjected to a full cyclic process of wetting and drying.

The location of the neutral plane along the pile shaft is important as it defines the portion of shaft in the stable zone, which offers the resistance against the pile displacement. It moves up or down depending on the ground movement, its direction and the magnitude of the applied force. Therefore, at a specific value of pile displacement, the location of the plane should be determined for an accurate analysis of the load distribution pattern along the pile shaft.

It is very important to note that although the ground movement influences the settlement of a pile, the ultimate failure takes place at its original capacity. Therefore, the behaviour of a pile subjected to ground movement may be regarded as a settlement problem, not a capacity problem.
Figure 4.1: Influence of elements on a distant point $i$
Figure 4.2: Discretisation of pile into various elements and nodes
Figure 4.3 (a) : Seasonal vertical soil movements

Figure 4.3 (b) : Ground movement distribution along pile shaft

Figure 4.3 : Modelling of ground movement along pile shaft
Figure 4.4: Effect of $L_p/D$ ratio on pile displacement in swelling soils

\[ P_{\text{max}} = \text{Maximum force exerted by swelling soils} \]

Figure 4.5: Effect of $L_p/D$ ratio on maximum load on pile in swelling soils
Figure 4.6: Influence of axial load on pile movement

Figure 4.7: Non-linear effects on pile settlements (Lp/D = 15)
Figure 4.10: Distribution of side shear under various load conditions \( (L_p/D = 15) \)
Figure 4.10: Distribution of side shear under various load conditions (L╲D = 15)
Figure 4.11: Distribution of side shear under various load conditions ($L_p/D = 25$)
Figure 4.12: Influence of $L_P/D$ on pile settlements for $P = 0.5P_{ult}$

Figure 4.13: Influence of $L_P/D$ on settlement under constant load conditions
Figure 4.14: Influence of depth of seasonal movement

Figure 4.15: Influence of pile compressibility \((L_p/D=15)\)
Figure 4.16: Influence of pile compressibility ($L_p/D=25$)

Figure 4.17: Pile compressibility ($L_p/D=15$) under no load condition
Figure 4.18: Pile compressibility \((L_p/D=15)\) under 0.5 \(P_{uk}\)

Figure 4.19: Pile compressibility \((L_p/D=50)\) under no load condition
Figure 4.20: Pile compressibility ($L_p/D=50$) under 0.5 $P_{\text{ult}}$

Figure 4.21: Influence of soil modulus on pile ($L_p/D=15$)
Figure 4.22: Influence of soil modulus (L_p/D=50)

Figure 4.23: Influence of Poisson's ratio on pile settlement (L_p/D=15)
Figure 4.24 : Influence of Poisson's ratio on pile settlement (L_p/D=50)

Figure 4.25 : Non-linear effects on pile displacements (L_p/D = 15)
Figure 4.26: Non-linear effects on pile displacements ($L_p/D = 25$)
Figure 4.27: Distribution of side shear along pile shaft ($L_p/D = 15$)
Figure 4.28: Distribution of side shear along pile shaft ($L_n/D = 25$)
Figure 4.29: Influence of ground movement on displacement ($L_P/D = 15$)

Figure 4.30: Influence of ground movement on pile displacement ($L_P/D = 25$)
Figure 4.31: Pile under tensile loading ($L_p/D = 15$)

Figure 4.32: Pile under tensile loading ($L_p/D = 25$)
Figure 4.33: Influence of pile compressibility, under no loading \((L_p/D=15)\)

Figure 4.34: Influence of pile compressibility under \(0.5P_{ult}\) loading \((L_p/D=15)\)
Figure 4.35: Influence of pile compressibility under no loading ($L_p/D=50$)

Figure 4.36: Influence of pile compressibility under $0.5P_{ult}$ loading ($L_p/D=50$)
Figure 4.37: Influence of soil modulus on pile (L_p/D=15)

Figure 4.38: Influence of soil modulus on pile (L_p/D=50)
CHAPTER - 5

THE ANALYSIS OF PILED RAFT FOUNDATION SYSTEMS

5.1 INTRODUCTION

In this chapter, the method adopted to carry out the analysis of piled raft foundation systems has been described. The effects of interaction between the various foundation components have been considered in order to formulate the final mathematical model. The compatibility between the movements of the raft, the piles and the soil has been incorporated to satisfy the "soil-structure interface" conditions. The effect of relative ground movement along the pile shaft and under the raft has also been incorporated in order to analyse the foundation behaviour in swelling and shrinking soils.

The raft is assumed to be a plate resting on a soil mass and also on the piles. The equations for raft elements are developed in a similar fashion to that described in Chapter 3, including the interaction effect from the pile elements. Similarly, the piles are analysed as a number of uniformly loaded cylindrical elements, together with uniformly loaded circular bases as described in Chapter 4, incorporating the interaction effect from the raft elements. The displacement compatibilities are incorporated to satisfy the "soil-structure interaction" conditions. Finally, the resulting equations are solved to determine the stress distribution as well as the displacement of the raft and the piles. The contact stresses underneath the raft are used to estimate the moments and the rotations at various locations of the raft. The equations are developed for "incompressible" as well as for "compressible" pile conditions. The influence of the ground movement underneath the raft and along the pile shaft is also included in the analysis to incorporate the effect of the swelling and the shrinking soils on the foundation system.

The behaviour of the foundation system has been analysed for "elastic" as well as for "non-linear conditions". The effects of separation between the raft and the soil, yielding of soil underneath the raft, slip at the "soil-pile" interface and local soil yielding under the pile base have been considered to incorporate the non-linear effects in the analysis. In order to carry out the analysis, a computer program PRAES (Piled Raft Analysis in Expansive Soils) has been developed by using FORTRAN 77 to undertake the necessary mathematical procedure. The results obtained from the computer model have been compared with the available solutions in order to demonstrate the applicability of the approach adopted.
The objective of this chapter is to describe the procedure adopted to develop the piled raft analysis and to compare the program output with the available work. The details of piled raft behaviour in swelling and shrinking soils are presented in later chapters.

5.2 METHOD OF ANALYSIS

In order to develop the analytical formulations for the piled raft foundation systems, the effect of mutual interaction between the foundation components (i.e. piles, raft and soil) is considered. The interactions are considered in the following manner:

- Raft Elements to Raft Elements
- Soil to Soil underneath the raft
- Raft Elements to Pile Elements
- Pile Elements to Raft Elements
- Pile Elements to Pile Elements

The above interactions are illustrated in Figure (5.1). These factors are determined and used to formulate the main set of equations. The equations are modified to incorporate the compatibility between the raft, the soil and the pile and are solved for the required unknowns. The analysis considers only the vertical response of the foundation system when subjected to vertical loading. The procedure adopted to develop these equations system is described in the following sections.

5.2.1 RAFT ANALYSIS

The raft is analysed as a plate resting on a semi-infinite soil mass as described in section 3.2.1, including the effect of piles underneath it. The raft and the piles are divided into various number of elements as illustrated in Figure (5.2). The influence of pile elements on each raft element is taken into account in formulating the equations. The displacement of raft elements as described in equation (3.15) may be written as:

\[ \delta_r = I_{rr} (-P) + a T + \delta q \]  \hspace{1cm} 5.1

Now, including the interaction effect of pile elements to each of the raft elements, the equation may be modified as

\[ \delta_r = I_{rr} (-P) + a T + \delta q + I_{pr} (-\tau) \]  \hspace{1cm} 5.2
where

\[ I_{rr} = \text{Raft to raft influence factors} \]
\[ I_{pr} = \text{Influence factor from pile elements to raft elements} \]
\[ (P) = \text{Contact stresses underneath the raft} \]
\[ (\tau) = \text{Stress along the pile shaft and the base} \]
\[ \delta q = \text{Vector of deflection at the center of each raft elements for the pinned condition under the imposed loads} \]
\[ T = \text{An unknown translation to incorporate the effect of pinned condition.} \]
\[ a = (I, I, I, \ldots, I)^T \]

The influence factors for pile to raft elements \( I_{pr} \) are obtained by using the methodology introduced by Poulos and Davis (1980), as described in section 4.2. The piles are divided into various cylindrical elements. The displacements under raft element \( i \) due to the stress around pile element \( j \) is determined by applying two equivalent point loads at a distance of \( L/4 \) from each pile element ends. The average of these displacements is adopted as the influence factor \( (I_{pr})_q \), i.e., the influence of stress at pile element \( j \) to the displacement of the centre of the raft element \( i \).

The deformation of soil, as described in equation (3.13) may be written as

\[ \delta S = I_{ss} \begin{bmatrix} P \end{bmatrix} \]

where

\[ I_{ss} = \text{Influence factor for soil to soil} \]

Equating raft and soil displacements from equations (5.2) and (5.3), it may be written as

\[ \delta_S = \delta_r \]

\[ \begin{bmatrix} I_{rr} + I_{ss} \end{bmatrix} \begin{bmatrix} P \end{bmatrix} - a \begin{bmatrix} T \end{bmatrix} + \begin{bmatrix} I_{pr} \end{bmatrix} \begin{bmatrix} \tau \end{bmatrix} = \delta q \]

\[ \begin{bmatrix} I_{rs} \end{bmatrix} \begin{bmatrix} P \end{bmatrix} - a \begin{bmatrix} T \end{bmatrix} + \begin{bmatrix} I_{pr} \end{bmatrix} \begin{bmatrix} \tau \end{bmatrix} = \delta q \]

where

\[ I_{rs} = (I_{rr} + I_{ss}) \]

The total vertical applied load on the raft \( P_{tot} \) is equated with the total reaction due the contact stresses underneath the raft to form another equation as:

\[ a^T \begin{bmatrix} P \end{bmatrix} = P_{tot} \]
The equations (5.5) and (5.6) may be expressed, for a raft discretised into $n$ elements and $m$ pile nodes, as below:

\[
\begin{align*}
(l_{rs})_{11} p_1 + (l_{rs})_{12} p_2 + \ldots + (l_{rs})_{1n} p_n + a T + (l_{pr})_{11} \tau_1 + (l_{pr})_{12} \tau_2 + \ldots + (l_{pr})_{1m} \sigma_m &= \delta_{q1} \\
(l_{rs})_{21} p_1 + (l_{rs})_{22} p_2 + \ldots + (l_{rs})_{2n} p_n + a T + (l_{pr})_{21} \tau_1 + (l_{pr})_{22} \tau_2 + \ldots + (l_{pr})_{2m} \sigma_m &= \delta_{q2} \\
(l_{rs})_{31} p_1 + (l_{rs})_{32} p_2 + \ldots + (l_{rs})_{3n} p_n + a T + (l_{pr})_{31} \tau_1 + (l_{pr})_{32} \tau_2 + \ldots + (l_{pr})_{3m} \sigma_m &= \delta_{q3}
\end{align*}
\]

\[
(l_{rs})_{n1} p_1 + (l_{rs})_{n2} p_2 + \ldots + (l_{rs})_{nn} p_n + a T + (l_{pr})_{n1} \tau_1 + (l_{pr})_{n2} \tau_2 + \ldots + (l_{pr})_{nm} \sigma_m &= \delta_{qn} \\
(-A_r)_{1} p_1 + (-A_r)_{2} p_2 + \ldots + (-A_r)_{n} p_n + 0 + 0 + 0 + \ldots + 0 &= -P_{tot}
\]

where

\[
\begin{align*}
(A_{ri}) &= \text{Area of } i \text{ th raft element} \\
\{\tau\} &= \{\tau_1, \tau_2, \tau_3, \ldots, \sigma_m\} \\
\{P\} &= \{p_1, p_2, p_3, \ldots, p_n\}
\end{align*}
\]
5.2.2 PILE ANALYSIS

The piles are analysed by using a similar method to that described in section 4.2, including the interaction effect from the raft elements. The piles are divided into a number of cylindrical shaped elements. As described in section 4.5, the nodal displacement of piles including the influence from the raft elements, may be written as:

$$\delta_p = I_{rp} \{P\} + I_{pp} \{\tau\} \quad \text{......5.8}$$

where,

$$I_{rp} = \text{Influence factors from raft elements to pile elements}$$
$$I_{pp} = \text{Influence factors from pile elements to pile elements}$$

For a layered soil mass, the influence factors may be obtained from a numerical analysis, such as that described by Wardle and Fraser (1976). For the case of a homogeneous semi-infinite soil mass, the influence factors $I_{rp}$ can be obtained by using the method described below.

The deflection at the corner of a rectangular loaded area may be expressed as

$$\rho_z = \frac{pb}{E_s} (1 - v_s^2) \left( A_r - \frac{1 - 2v_s}{1 - v_s} B_r \right) \quad \text{......5.9}$$

where,

$$\rho_z = \text{Vertical displacement of pile node at depth } z \text{ beneath the corner of a rectangular loaded area}$$
$$p = \text{Intensity of applied load underneath raft elements}$$
$$b = \text{Width of rectangular area}$$
$$v_s = \text{Poisson's ratio of soil}$$

The factors $A$ and $B$ are defined as:

$$A_r = \frac{1}{2\pi} \left( \ln \frac{\sqrt{1 + m^2 + n_i^2}}{\sqrt{1 + m_i^2 + n_i^2}} + m_i + m_i \ln \frac{\sqrt{1 + m^2 + n_i^2}}{\sqrt{1 + m_i^2 + n_i^2}} - 1 \right)$$
\[ B = \frac{n_1 \tan^{-1} \frac{m_1}{n_1 \sqrt{1 + m_1^2 + n_1^2}}}{2\pi} \]

where,

\[ m_1 = \frac{1}{b} = \frac{\text{Length of rectangle}}{\text{Width of rectangle}} \]

\[ n_1 = \frac{z}{b} \]

\[ z = \text{Depth at which the displacement } p_z \text{ caused due to the stress at the rectangular area} \]

The principle of superimposition is used to determine the vertical displacement of a point at a depth \( z \) other than the corner of the rectangle as described in section 3.2.3.1.

Therefore, the influence factors \( I_{rj} \) for a pile node \( i \) at depth \( z \) due to the stress underneath the raft \( p_j \) on an area \( j \) may be expressed as \( (I_{rj}) \).

For example, \( (I_{rj})_{24} \) indicates the displacement of pile node 2 due to the unit stress under the raft element 4. These factors are combined with pile to pile influence factors as described in equation (5.8) in order to obtain the set of equations (5.11).

To satisfy the equilibrium of forces along the pile, the total load on each pile is equated with the shaft stress along \( k \) pile elements and one base reaction, as follows:

\[ \sum_{i=1}^{k} \tau_i \pi D L_i + \sigma_b \frac{\pi}{4} D_b^2 = P_{pile} = A_{rp} p_p \]

where,

\[ A_{rp} = \text{Area of the raft element connected to the pile, which transfers force from raft to pile} \]

\[ p_p = \text{Contact stress under the raft element connected to the pile} \]

\[ P_{pile} = \text{Total load imposed on the pile from the raft element} \]

\[ L_i = \text{Length of pile element} \]

Equations (5.7), (5.8) and (5.10) may be expressed for a system discretised into \( m \) number of pile nodes and \( n \) number of raft elements as:
\[(I_{rp})_{ii}p_i + (I_{rp})_{i2}p_2 + \ldots + (I_{rp})_{in}p_n + (I_{pp})_{ii}\tau_1 + (I_{pp})_{i2}\tau_2 + \ldots + (I_{pp})_{in}\sigma_m = \delta_{pi}\]

\[(I_{rp})_{i1}p_1 + (I_{rp})_{i2}p_2 + \ldots + (I_{rp})_{in}p_n + (I_{pp})_{i1}\tau_1 + (I_{pp})_{i2}\tau_2 + \ldots + (I_{pp})_{in}\sigma_m = \delta_{pi}\]

\[(I_{rp})_{m1}p_1 + (I_{rp})_{m2}p_2 + \ldots + (I_{rp})_{mn}p_n + (I_{pp})_{m1}\tau_1 + (I_{pp})_{m2}\tau_2 + \ldots + (I_{pp})_{mn}\sigma_m = \delta_{pm}\]

\[0 + 0 + \ldots + 0 + \ldots + 0 + [-A_{p1}\tau_1 + [-A_{p2}\tau_2 + \ldots + [-A_{bm}\sigma_m = -P_{pile}\]

\[
\text{where} \\
\delta_{pi} = \text{Displacement at pile node } i \\
p_i = \text{Contact stress under raft element } i
\]

5.2.3 \textit{Interaction of Soil - Raft - Pile Interface Conditions}

Combining the set of equations (5.7) & (5.11), the formulation for a piled raft system with \(n\) raft elements and \(m\) pile nodes may be expressed as:

\[\text{Piled Raft Foundation Systems in Expansive Soils}\]

\[\text{5-132}\]
\[(l_{rs})_{11} p_1 + (l_{rs})_{12} p_2 + \cdots + (l_{rs})_{1n} p_n - aT + (l_{pr})_{11} \tau_1 + (l_{pr})_{12} \tau_2 + \cdots + (l_{pr})_{1m} \sigma_m = \delta_{q1}\]
\[(l_{rs})_{21} p_1 + (l_{rs})_{22} p_2 + \cdots + (l_{rs})_{2n} p_n - aT + (l_{pr})_{21} \tau_1 + (l_{pr})_{22} \tau_2 + \cdots + (l_{pr})_{2m} \sigma_m = \delta_{q2}\]
\[(l_{rs})_{31} p_1 + (l_{rs})_{32} p_2 + \cdots + (l_{rs})_{3n} p_n - aT + (l_{pr})_{31} \tau_1 + (l_{pr})_{32} \tau_2 + \cdots + (l_{pr})_{3m} \sigma_m = \delta_{q3}\]

\[
\begin{align*}
(l_{rs})_{n1} p_1 + (l_{rs})_{n2} p_2 + \cdots + (l_{rs})_{nn} p_n - aT + (l_{pr})_{n1} \tau_1 + (l_{pr})_{n2} \tau_2 + \cdots + (l_{pr})_{nm} \sigma_m &= \delta_{qn} \\
(-A_r)_{1} p_1 + (-A_r)_{2} p_2 + \cdots + (-A_r)_{n} p_n - 0 + 0 + 0 + \cdots + 0 &= -P_{\text{tot}} \\
(l_{rp})_{11} p_1 + (l_{rp})_{12} p_2 + \cdots + (l_{rp})_{1n} p_n - 0 + (l_{pp})_{11} \tau_1 + (l_{pp})_{12} \tau_2 + \cdots + (l_{pp})_{1m} \sigma_m &= \delta_{p1} \\
(l_{rp})_{21} p_1 + (l_{rp})_{22} p_2 + \cdots + (l_{rp})_{2n} p_n - 0 + (l_{pp})_{21} \tau_1 + (l_{pp})_{22} \tau_2 + \cdots + (l_{pp})_{2m} \sigma_m &= \delta_{p2} \\
(l_{rp})_{m1} p_1 + (l_{rp})_{m2} p_2 + \cdots + (l_{rp})_{mn} p_n - 0 + (l_{pp})_{m1} \tau_1 + (l_{pp})_{m2} \tau_2 + \cdots + (l_{pp})_{mm} \sigma_m &= \delta_{pm} \\
0 + 0 + \cdots + 0 + (-A_{p1}) \tau_1 + (-A_{p2}) \tau_2 + \cdots + (-A_{bm}) \sigma_m &= -P_{\text{pile}}
\end{align*}
\]
The above equations are modified to introduce the "pile-raft-soil" interface compatibilities. In order to do so, the raft elements, which are connected to the pile head, are assigned displacements equals to the pile head displacements.

As these piled raft elements are not in contact with the soil, the influence of other pile elements on these raft elements through the soil medium is not considered in the analysis. Similarly, the influence of contact stresses underneath other raft elements on these piled raft elements through the soil medium is also not included in the analysis. As the piled raft element is an integral part of the raft, its structural continuity with other raft elements is maintained, excluding the effect of soil to soil interaction. The load on the pile, which is connected to the raft, is transferred from the connected raft element. Equations (5.12) are modified to incorporate the above conditions.

To describe this, the formulation for the $i$ th raft element connected with the first pile node may expressed as:

$$\delta_{ri} = \delta_{p1} \quad \text{..................5.13}$$

where,

$$\delta_{ri} = \text{displacement of } i \text{ raft element}$$

$$\delta_{p1} = \text{displacement of first pile node}.$$

Substituting for $\delta_{ri}$ from equation (5.1), the above equation may be expressed as

$$I_{rr} \ (-P) + a \ T + \delta_q = \delta_{p1}(=\delta_{ri}) \quad \text{........5.14}$$

Substituting the above equation in the $i$ th equation of the set of equation (5.12), the result may be expressed as:
\begin{align*}
(l_{rs})_{11} & \; p_1 + (l_{rs})_{12} \; p_2 + \ldots + (l_{rs})_{1n} \; p_n - a \; T + (l_{pr})_{11} \; \tau_1 + (l_{pr})_{12} \; \tau_2 + \ldots + (l_{pr})_{1m} \; \sigma_m = \delta_{q1} \\
(l_{rs})_{21} & \; p_1 + (l_{rs})_{22} \; p_2 + \ldots + (l_{rs})_{2n} \; p_n - a \; T + (l_{pr})_{21} \; \tau_1 + (l_{pr})_{22} \; \tau_2 + \ldots + (l_{pr})_{2m} \; \sigma_m = \delta_{q2} \\
(l_{rs})_{31} & \; p_1 + (l_{rs})_{32} \; p_2 + \ldots + (l_{rs})_{3n} \; p_n - a \; T + (l_{pr})_{31} \; \tau_1 + (l_{pr})_{32} \; \tau_2 + \ldots + (l_{pr})_{3m} \; \sigma_m = \delta_{q3} \\
(l_{rr})_{i1} & \; p_1 + (l_{rr})_{i2} \; p_2 + \ldots + (l_{rr})_{in} \; p_n - a \; T + 0 + 0 + \ldots + \delta_{ri} = \delta_{qi} \\
(l_{rs})_{n1} & \; p_1 + (l_{rs})_{n2} \; p_2 + \ldots + (l_{rs})_{nn} \; p_n - a \; T + (l_{pr})_{n1} \; \tau_1 + (l_{pr})_{n2} \; \tau_2 + \ldots + (l_{pr})_{nm} \; \sigma_m = \delta_{qn} \\
(-A_p)_1 & \; p_1 + (-A_p)_2 \; p_2 + \ldots + (-A_p)_n \; p_n - 0 + 0 + 0 + \ldots + 0 = -P_{tot} \\
(l_{rp})_{11} & \; p_1 + (l_{rp})_{12} \; p_2 + \ldots + (l_{rp})_{1n} \; p_n - 0 + (l_{pp})_{11} \; \tau_1 + (l_{pp})_{12} \; \tau_2 + \ldots + (l_{pp})_{1m} \; \sigma_m = \delta_{p1} \\
(l_{rp})_{21} & \; p_1 + (l_{rp})_{22} \; p_2 + \ldots + (l_{rp})_{2n} \; p_n - 0 + (l_{pp})_{21} \; \tau_1 + (l_{pp})_{22} \; \tau_2 + \ldots + (l_{pp})_{2m} \; \sigma_m = \delta_{p2} \\
(l_{rp})_{m1} & \; p_1 + (l_{rp})_{m2} \; p_2 + \ldots + (l_{rp})_{mn} \; p_n - 0 + (l_{pp})_{m1} \; \tau_1 + (l_{pp})_{m2} \; \tau_2 + \ldots + (l_{pp})_{mm} \; \sigma_m = \delta_{pm} \\
0 & + 0 + \ldots + 0 + (-A_{pl}) \; \tau_1 + (-A_{p2}) \; \tau_2 + \ldots + (-A_{bm}) \; \sigma_m = -P_{pile} \\
\end{align*}
The above set of equations are solved for both incompressible and compressible piles, as described in the following sections.

5.2.3.1 **INCOMPRESSIBLE PILE CONDITION**

To incorporate the incompressible pile condition, each pile node is assigned the same displacement. In other words, the pile compression is negligible in comparison to the deformation of the surrounding soil. Therefore, for a single incompressible pile, the nodal displacements may be expressed as:

\[ \delta_{p1} = \delta_{p2} = \delta_{p3} = \ldots = \delta_{pm} = \delta_p \ldots \ldots 5.16 \]

where,

\[ m = \text{Number of nodes} \]

Equations (5.15) may be modified to incorporate the incompressible pile condition as follows:
\[(I_{rs})_{11} p_1 + (I_{rs})_{12} p_2 + ... + (I_{rs})_{1n} p_n - a T + (I_{pr})_{11} \tau_1 + (I_{pr})_{12} \tau_2 + ... + (I_{pr})_{1m} \sigma_m = \delta_{q1}
\]

\[(I_{rs})_{21} p_1 + (I_{rs})_{22} p_2 + ... + (I_{rs})_{2n} p_n - a T + (I_{pr})_{21} \tau_1 + (I_{pr})_{22} \tau_2 + ... + (I_{pr})_{2m} \sigma_m = \delta_{q2}
\]

\[(I_{rs})_{31} p_1 + (I_{rs})_{32} p_2 + ... + (I_{rs})_{3n} p_n - a T + (I_{pr})_{31} \tau_1 + (I_{pr})_{32} \tau_2 + ... + (I_{pr})_{3m} \sigma_m = \delta_{q3}
\]

\[(I_{rr})_{i1} p_1 + (I_{rr})_{i2} p_2 + ... + (I_{rr})_{in} p_n - a T + 0 + 0 + ... + \delta_{ri} = \delta_{qi}
\]

\[(I_{rs})_{n1} p_1 + (I_{rs})_{n2} p_2 + ... + (I_{rs})_{nn} p_n - a T + (I_{pr})_{n1} \tau_1 + (I_{pr})_{n2} \tau_2 + ... + (I_{pr})_{nm} \sigma_m = \delta_{qn}
\]

\[(-A_r)_1 p_1 + (-A_r)_2 p_2 + ... + (-A_r)_n p_n - 0 + 0 + 0 + ... + 0 = -P_{tot}
\]

\[(I_{rp})_{i1} p_1 + (I_{rp})_{i2} p_2 + ... + (I_{rp})_{in} p_n - 0 + (I_{pp})_{i1} \tau_1 + (I_{pp})_{i2} \tau_2 + ... + (I_{pp})_{im} \sigma_m = \delta_p
\]

\[(I_{rp})_{21} p_1 + (I_{rp})_{22} p_2 + ... + (I_{rp})_{2n} p_n - 0 + (I_{pp})_{21} \tau_1 + (I_{pp})_{22} \tau_2 + ... + (I_{pp})_{2m} \sigma_m = \delta_p
\]

\[(I_{rp})_{m1} p_1 + (I_{rp})_{m2} p_2 + ... + (I_{rp})_{mn} p_n - 0 + (I_{pp})_{m1} \tau_1 + (I_{pp})_{m2} \tau_2 + ... + (I_{pp})_{mm} \sigma_m = \delta_p
\]

\[0 + 0 + ... + 0 + (-A_{pl}) \tau_1 + (-A_{p2}) \tau_2 + ... + (-A_{bm}) \sigma_m = -P_{pile}
\]
The load on the pile is transferred from the raft element, which is connected to the pile head. Therefore, the imposed load on the pile head may be expressed as:

\[ P_{\text{pile}} = (A_r)_i \times p_i \]  \hspace{1cm} (5.18)

where,

\((A_r)_i\) = area of the raft element, connected to the pile 

\(p_i\) = contact stress underneath the piled raft element

Substituting this in equations (5.17), it may be obtained as:
\( (I_{rs})_{11} p_1 + (I_{rs})_{12} p_2 + ..... + (I_{rs})_{in} p_n - a_T + (I_{pr})_{11} \tau_1 + (I_{pr})_{12} \tau_2 + ..... + (I_{pr})_{im} \sigma_m - 0 = \delta_{q1} \)

\( (I_{rs})_{21} p_1 + (I_{rs})_{22} p_2 + ..... + (I_{rs})_{2n} p_n - a_T + (I_{pr})_{21} \tau_1 + (I_{pr})_{22} \tau_2 + ..... + (I_{pr})_{2m} \sigma_m - 0 = \delta_{q2} \)

\( (I_{rs})_{31} p_1 + (I_{rs})_{32} p_2 + ..... + (I_{rs})_{3n} p_n - a_T + (I_{pr})_{31} \tau_1 + (I_{pr})_{32} \tau_2 + ..... + (I_{pr})_{3m} \sigma_m - 0 = \delta_{q3} \)

\( (I_{rr})_{i1} p_1 + (I_{rr})_{i2} p_2 + ..... + (I_{rr})_{in} p_n - a_T + 0 + 0 + ..... + \delta_r = \delta_{qi} \)

\( (I_{rs})_{n1} p_1 + (I_{rs})_{n2} p_2 + ..... + (I_{rs})_{nn} p_n - a_T + (I_{pr})_{n1} \tau_1 + (I_{pr})_{n2} \tau_2 + ..... + (I_{pr})_{nm} \sigma_m - 0 = \delta_{qn} \)

\( (-A_r)_1 p_1 + (-A_r)_2 p_2 + ..... + (-A_r)_n p_n - 0 + 0 + 0 + ..... + 0 - 0 = -P_{tot} \)

\( (I_{rp})_{11} p_1 + (I_{rp})_{12} p_2 + ..... + (I_{rp})_{1n} p_n - 0 + (I_{pp})_{11} \tau_1 + (I_{pp})_{12} \tau_2 + ..... + (I_{pp})_{im} \sigma_m - 0 = \delta_p \)

\( (I_{rp})_{21} p_1 + (I_{rp})_{22} p_2 + ..... + (I_{rp})_{2n} p_n - 0 + (I_{pp})_{21} \tau_1 + (I_{pp})_{22} \tau_2 + ..... + (I_{pp})_{2m} \sigma_m - 0 = \delta_p \)

\( (I_{rp})_{m1} p_1 + (I_{rp})_{m2} p_2 + ..... + (I_{rp})_{mn} p_n - 0 + (I_{pp})_{m1} \tau_1 + (I_{pp})_{m2} \tau_2 + ..... + (I_{pp})_{mm} \sigma_m - 0 = \delta_p \)

\( 0 + 0 + ..... + (A_r)_i \times p_i + ..... + 0 + (-A_p)_1 \tau_1 + (-A_p)_2 \tau_2 + ..... + (-A_hm) \sigma_m - 0 = 0 \)

The above set of equations may be expressed in a matrix form as below:

\[ \begin{bmatrix} \end{bmatrix} \]
\[
\begin{bmatrix}
(l_{rs})_{11} & (l_{rs})_{12} & \ldots & (l_{rs})_{1n} & a & (l_{pr})_{11} & (l_{pr})_{12} & \ldots & (l_{pr})_{1m} & 0 \\
(l_{rs})_{21} & (l_{rs})_{22} & \ldots & (l_{rs})_{2n} & a & (l_{pr})_{21} & (l_{pr})_{22} & \ldots & (l_{pr})_{2m} & 0 \\
(l_{rs})_{31} & (l_{rs})_{32} & \ldots & (l_{rs})_{3n} & a & (l_{pr})_{31} & (l_{pr})_{32} & \ldots & (l_{pr})_{3m} & 0 \\
\vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots & \vdots \\
(l_{rs})_{n1} & (l_{rs})_{n2} & \ldots & (l_{rs})_{nn} & a & (l_{pr})_{n1} & (l_{pr})_{n2} & \ldots & (l_{pr})_{nm} & 0 \\
(-A_{r})_{1} & (-A_{r})_{2} & \ldots & (-A_{r})_{n} & 0 & 0 & 0 & \ldots & 0 & 0 \\
(l_{pp})_{11} & (l_{pp})_{12} & \ldots & (l_{pp})_{1n} & 0 & (l_{pp})_{11} & (l_{pp})_{12} & \ldots & (l_{pp})_{1m} & -1 \\
(l_{pp})_{21} & (l_{pp})_{22} & \ldots & (l_{pp})_{2n} & 0 & (l_{pp})_{21} & (l_{pp})_{22} & \ldots & (l_{pp})_{2m} & -1 \\
\vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots & \vdots \\
(l_{pp})_{m1} & (l_{pp})_{m2} & \ldots & (l_{pp})_{mn} & 0 & (l_{pp})_{m1} & (l_{pp})_{m2} & \ldots & (l_{pp})_{mm} & -1 \\
0 & 0 & \ldots & (A_{p}) & 0 & 0 & (A_{p}) & \ldots & (A_{pm}) & 0 \\
\end{bmatrix}
\begin{bmatrix}
p_1 \\
p_2 \\
p_3 \\
p_4 \\
p_5 \\
p_6 \\
p_7 \\
p_8 \\
p_9 \\
p_{10} \\
\end{bmatrix}
= 
\begin{bmatrix}
\delta_q1 \\
\delta_q2 \\
\delta_q3 \\
\delta_{q1} \\
\delta_{q2} \\
\delta_{q3} \\
\delta_{q4} \\
T \\
\tau_1 \\
\tau_2 \\
\tau_m \\
\delta_p \\
\delta_{p1} \\
\end{bmatrix}
\]

(5.20)
The above matrix is solved for the following unknowns:

- Contact Stresses underneath each raft element \((P)\)
- Translation \((T)\)
- Shear stresses along the pile shafts \((\tau)\)
- Pile head settlement \((\delta_p)\)

The force vector is formed by combining the computed contact stresses \((P)\), with the imposed loading on the raft and is applied to the plate to determine the raft displacements and the moments as described in section 3.2.3.3.

### 5.2.3.2 Compressible Pile Condition

The pile compressibility is incorporated in a similar manner to that described in section 4.2.2. The structural compression of each pile element due to the shear stress is evaluated and added together to obtain the pile nodal displacement with the top of the pile as datum. The equations are modified as described in section 4.2.2 and are included in the main matrix to solve for the required unknowns.

### 5.2.4 Free Field Soil Movement Analysis

As discussed in previous chapters, the relative ground movement due to the process of swelling and shrinking of soils influences the raft due to the change in the supporting soil profile underneath it, whereas in case of a pile it induces a force along the shaft. In piled raft foundation systems, this phenomenon is rather more complex because of the structural connection between the piles and the raft and their respective arrangement. The relative movement of the ground has a significant impact on the behaviour of the piled raft foundation systems.

A piled raft constructed during the driest period, will experience an upward ground movement due to the swelling of soils in the following wet season. The edge of the raft and the outer piles will subject to higher upward ground movement compared to the central portion of the raft and inner piles. The swelling of the soils along the edge of the raft will be high because of its direct exposure to the seasonal temperature fluctuations. As discussed earlier, this type of soil movement is designated as "edge heave formation" underneath the raft. The upward movement of the ground causes an upward pressure underneath the raft as well an upward force along the pile shaft. The following cycles of the
wetting and drying processes cause a slow penetration of moisture towards the central portion of the raft and results in swelling over the course of time. The clay soil towards the central portion of the raft is not sensitive to the seasonal moisture fluctuation as it is not directly exposed to the atmosphere. However, the soil along the edge of the raft experiences swelling and shrinking effects with the seasonal moisture fluctuation. The cycles of moisture variation will cause further increase in the moisture intrusion towards the centre of the raft. As a result of this, the soil under the central raft will tend to swell and pushes the raft upward, but the piles may restrict the upward movement of the raft. This process may reduce the compressive stress at the pile-raft interface or in some situations and it may develop a tensile stress. These changes in the stress conditions causes a redistribution of stress in between various foundation components. The final result is dependent primarily on the following parameters:

- Geometry of the foundation components
- Number and location of the piles
- Soil parameters
- Magnitude and the depth of seasonal movement
- Loading conditions.

Similarly, a piled raft constructed in a wet ground condition, already rests on a swollen soil profile, which is flat initially, but with the following dry season, the soil along the edges of the raft experiences a shrinking effect and results in a downward movement. This process causes a downdrag force along the outer pile shafts. The moisture inside the central portion of the raft remains entrapped, because the raft acts as an impermeable barrier. This process causes a "central heave formation" underneath the raft.

Therefore, the initial state of the ground moisture condition dictates the formation soil profiles in the following cycles of the seasonal wetting and drying process. Therefore, it is necessary to incorporate these soil profiles in the analysis and design, so that the integrity of the raft, the piles and their inter-connection can be assessed under various seasonal conditions.

In order to incorporate these effects, a free field soil movement profile underneath the raft slab is assumed, as described in section 3.2.4.1. The movement of the ground at the pile heads ($S_p$) is specified, depending on the location of piles underneath the raft. The ground movements at the pile heads are distributed along the pile shafts as described in section 4.2.3. The ground movement distribution in piled raft systems is indicated schematically in Figure (5.3).
To incorporate the soil profile underneath the raft, equation (5.5) is modified as follows:

\[
[I_r] \{P\} - a \{\tau\} + [I_{pr}] \{\tau\} = \delta_q - \{S_r\} \quad \text{...............}(5.21)
\]

where,

\[
\{S_r\} = \text{vector of soil surface movement underneath the raft (positive for shrinking soil and negative for swelling soils)}
\]

Similarly, equation (5.8) is modified to include the ground movement along the pile shaft, to give:

\[
[I_p] \{P\} + [I_{pp}] \{\tau\} = \delta_p - \{S_z\} \quad \text{.................................}(5.22)
\]

where,

\[
\{S_z\} = \text{vector of soil surface movement along the pile shaft (positive for shrinking soil and negative for swelling soils)}
\]

The final set of equations is formulated by combining equations (5.6), (5.10), (5.21) and (5.22). The formulation of the final set of equations and their matrix form are indicated in equations (5.23) and (5.24), respectively.
\[(l_{rs})_{11} \ p_1 + (l_{rs})_{12} p_2 + \ldots + (l_{rs})_{1n} p_n - a_T + (l_{pr})_{11} \tau_1 + (l_{pr})_{12} \tau_2 + \ldots + (l_{pr})_{1m} \sigma_m - 0 = \delta_{q1} - S_{r1} \]

\[(l_{rs})_{21} p_1 + (l_{rs})_{22} p_2 + \ldots + (l_{rs})_{2n} p_n - a_T + (l_{pr})_{21} \tau_1 + (l_{pr})_{22} \tau_2 + \ldots + (l_{pr})_{2m} \sigma_m - 0 = \delta_{q2} - S_{r2} \]

\[(l_{rs})_{31} p_1 + (l_{rs})_{32} p_2 + \ldots + (l_{rs})_{3n} p_n - a_T + (l_{pr})_{31} \tau_1 + (l_{pr})_{32} \tau_2 + \ldots + (l_{pr})_{3m} \sigma_m - 0 = \delta_{q3} - S_{r3} \]

\[(l_{rr})_{11} p_1 + (l_{rr})_{12} p_2 + \ldots + (l_{rr})_{1n} p_n - a_T + 0 + 0 + \ldots + \delta_p = \delta_{q1} \]

\[(l_{rs})_{n1} p_1 + (l_{rs})_{n2} p_2 + \ldots + (l_{rs})_{nn} p_n - a_T + (l_{pr})_{n1} \tau_1 + (l_{pr})_{n2} \tau_2 + \ldots + (l_{pr})_{nn} \sigma_m - 0 = \delta_{qn} - S_{rm} \]

\[(-A_r)_1 p_1 + (-A_r)_2 p_2 + \ldots + (-A_r)_n p_n - 0 + 0 + 0 + \ldots + 0 - 0 = -P_{tol} \]

\[(l_{rp})_{11} p_1 + (l_{rp})_{12} p_2 + \ldots + (l_{rp})_{1n} p_n - 0 + (l_{pp})_{11} \tau_1 + (l_{pp})_{12} \tau_2 + \ldots + (l_{pp})_{1m} \sigma_m - 0 = \delta_p - S_{p1} \]

\[(l_{rp})_{21} p_1 + (l_{rp})_{22} p_2 + \ldots + (l_{rp})_{2n} p_n - 0 + (l_{pp})_{21} \tau_1 + (l_{pp})_{22} \tau_2 + \ldots + (l_{pp})_{2m} \sigma_m - 0 = \delta_p - S_{p2} \]

\[(l_{rp})_{m1} p_1 + (l_{rp})_{m2} p_2 + \ldots + (l_{rp})_{mn} p_n - 0 + (l_{pp})_{m1} \tau_1 + (l_{pp})_{m2} \tau_2 + \ldots + (l_{pp})_{mn} \sigma_m - 0 = \delta_p - S_{pm} \]

\[0 + 0 + \ldots + (A_r)_i \times p_i + \ldots + 0 + (-A_{p1}) \tau_1 + (-A_{p2}) \tau_2 + \ldots + (-A_{bm}) \sigma_m - 0 = 0 \]

(5.23)
\[
\begin{bmatrix}
(I_{s11}, I_{s12}) & \ldots & (I_{s1n}, a) & (I_{pr11}, I_{pr12}) & \ldots & (I_{pr1m}, 0) \\
(I_{s21}, I_{s22}) & \ldots & (I_{s2n}, a) & (I_{pr21}, I_{pr22}) & \ldots & (I_{pr2m}, 0) \\
\vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\
(I_{sji1}, I_{sji2}) & \ldots & (I_{sjin}, a) & (I_{prji1}, I_{prji2}) & \ldots & (I_{prji3m}, 0) \\
(I_{ri1}, I_{ri2}) & \ldots & (I_{rin}, a) & 0 & 0 & 0 & 0 & +1 \\
\vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\
(I_{ri1}, I_{ri2}) & \ldots & (I_{rin}, a) & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
(-A_r)_1, (-A_r)_2 & \ldots & (-A_r)_n & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
(I_{pr11}, I_{pr12}) & \ldots & (I_{pr1n}, 0) & (I_{pp11}, I_{pp12}) & \ldots & (I_{pp1m}, -1) \\
(I_{pr21}, I_{pr22}) & \ldots & (I_{pr2n}, 0) & (I_{pp21}, I_{pp22}) & \ldots & (I_{pp2m}, -1) \\
\vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\
(I_{pm1}, I_{pm2}) & \ldots & (I_{pmn}, 0) & (I_{ppm1}, I_{ppm2}) & \ldots & (I_{ppmn}, -1) \\
0 & 0 & \ldots & (A_r)_i & \ldots & 0 & 0 & (-A_p1) & (-A_p2) & \ldots & (-A_pm) & 0 \\
\end{bmatrix}
\]
\[
\begin{bmatrix}
p_1 \\
p_2 \\
p_3 \\
p_i \\
p_n \\
T \\
\tau_1 \\
\tau_2 \\
\tau_m \\
\delta_p \\
\end{bmatrix}
= \begin{bmatrix}
\delta_{q1} - S_{r1} \\
\delta_{q2} - S_{r2} \\
\delta_{q3} - S_{r3} \\
\delta_{qn} - S_{mn} \\
-p_{\text{tot}} \\
-S_{p1} \\
-S_{p2} \\
\vdots \\
-S_{pm} \\
0 \\
\end{bmatrix}
\]

\text{(5.24)}
These equations are solved for various unknowns, as mentioned in previous sections.

5.2.5 **OTHER CONDITIONS**

The analyses are extended to incorporate the effect of elastic as well as non-linear conditions. These conditions are described below:

5.2.5.1 **ELASTIC CONDITION**

The following conditions are adopted to incorporate the elastic condition in the analysis:

- *No separation between the raft and the soil*
- *Bearing capacity of soil underneath the raft and the pile base is infinite*
- *Pile - soil interface is capable of resisting any shear stress which can develop i.e. no interface slip.*

The procedure adopted to incorporate these conditions is described in sections 3.2.5.1 and 4.2.4.1.

5.2.5.2 **NON-LINEAR CONDITIONS**

The following conditions are considered to incorporate non-linear effects in the analysis:

- *Lift off underneath the raft*
- *Local soil yield underneath the raft*
- *Slip at the soil-pile interface*
- *Yielding of soil under the pile base*

The procedures adopted to incorporate the above conditions are described in sections 3.2.5.2 and 4.2.4.2.
5.3 DEVELOPMENT OF COMPUTER MODEL

A computer program \textit{PRAES} (\textit{Piled Raft Analysis in Expansive Soils}) has been developed by using FORTRAN 77 in order to carry out the necessary mathematical formulations. The various stages of the computer programming are outlined below:

1. **INPUT DATA**:

In this stage, the required data, defining the whole foundation system are read. The input block of the program is structured to read the following data to carry out the required analysis:

- Number of nodes and elements of the raft
- Geometrical properties of the raft
- Material properties of the raft and the soil
- Boundary conditions
- Imposed loads on the raft elements
- Number of Piles and its locations
- Geometrical properties of Piles
- Material properties of pile
- Number of pile elements and its sizes
- Maximum soil free field soil movement due to swelling or shrinking of soils underneath the raft
- Extent of the ground movement along the pile shaft

- Required analysis type
  - Elastic
  - Non-linear

If elastic analysis is required, the following conditions are incorporated:

- No Lift off underneath the raft
- No local soil yield under the raft
- No Soil - Pile slip
- No yielding of soil under the pile base

If non-linear analysis is required, the following conditions are allowed:

- Lift off underneath the raft
- Local soil yield under the raft
• Soil - Pile slip
• Yielding of soil under the pile base.

2. **Generating Soil Profile:**

The vertical soil profile under each raft element and along the pile shaft, as discussed in section 5.2.4, is generated and stored in an array for later application.

3. **Formation of Stiffness Matrix:**

Stiffness matrix is formed for all individual raft elements and is coupled to obtain the global stiffness matrix, as described in section 3.3(3). It is denoted as \([K]\) in equation (3.6).

4. **Formation of Load Vector due to Imposed Loading:**

The load vector is formed with the imposed load on the raft. This is a column matrix, consisting of \((4 \times \text{no. of nodes})\) number of rows.

5. **Boundary Conditions:**

As discussed in section 3.3(5), the equations are modified for the pinned condition, rotations and twist.

6. **Solving for Pinned Raft Displacement for Imposed Load:**

The matrix is solved for the displacement \(\delta_q\), as described in equation (3.17).

7. **Formation of Load Vector by Applying Unit Loading:**

The force vector is determined by applying an unit load on the raft. The method of obtaining this matrix is similar to that as described in step (4).

8. **Boundary Conditions:**

The above load vector is modified for the specified boundary conditions.

9. **Solving for Raft Interaction Factors:**

These factors are determined by using the equations, as described in section 3.3(9).
10. **SOIL INFLUENCE MATRIX:**

This matrix is developed, as described in section 3.3(10).

11. **FORMATION OF TOTAL LOAD EQUATION FOR RAFT:**

The vertical applied load on the raft \( P_{tot} \) is equated to the contact pressures, multiplied by the relevant element areas, as indicated in equation (5.6).

12. **PILE TO PILE INFLUENCE FACTORS:**

These factors \( I_{pp} \) are determined as shown in equation (5.8) and are placed in the set of equations (5.23).

13. **RAFT TO PILE INFLUENCE FACTORS:**

These factors \( I_{rp} \) are determined as indicated in equation (5.8) and are placed in the set of equations (5.23).

14. **PILE TO RAFT INFLUENCE FACTORS:**

These factors \( I_{pr} \) are determined as discussed in section (5.2.1) and are placed in the set of equations (5.23).

15. **FORMULATION OF TOTAL LOAD EQUATIONS FOR PILES:**

The force along the pile shaft and at the base are equated with the load transferred from the raft element, as indicated in equation (5.10).

16. **INCORPORATING VERTICAL SOIL PROFILE UNDER THE RAFT:**

The equations are modified to incorporate the vertical soil movement vector \( S_v \), which is determined in step 2. This is discussed in section 5.2.4.
17. **Introducing Soil Movement Along the Pile Shaft:**

The equations are modified to incorporate the vertical soil movement vector \( (S_y) \) along the pile shaft as determined in step 2. The procedure is discussed in section 5.2.4.

18. **Modifying The Matrix For Non-Linear Conditions:**

This section of the program is activated after the first run and undertakes following processes:

**For Raft:**
- *Lift Off under the raft*
- *Local yielding of soil underneath the raft*

The procedure to incorporate the above conditions is discussed in section 3.3(13).

**For Pile:**
- *Soil Pile Slip*
- *Yielding of soil under the pile base*

The procedure to include these conditions is discussed in section 4.3(6).

It should be noted that these conditions are not considered in the case of an elastic analysis.

19. **Solving The Main Matrix:**

The set of equations shown in eq.5.23 is solved for the following unknowns

- *Contact Stresses underneath the raft*
- *Translation*
- *Shear stresses along the pile shaft*
- *Pile displacements.*

20. **Screening Stage:**

In this part of the program, the following aspects are screened and processed, as discussed in sections 3.2.5 & 4.2.4 for rafts & piles, respectively:
For Rafts:

- **Tensile Stresses underneath the raft elements exceeding the specified limit**
- **Compressive contact Stresses exceeding the specified limit**

For Piles:

- **Positive shear stresses exceeding the specified stress limit**
- **Stress at the pile base exceeding the specified limit**

The program run is diverted to stage 18 in order to incorporate the above conditions for various raft and pile elements, if there is any.

After processing the above conditions for all elements, the following condition is screened

- **Negative shear stress along the pile shaft exceeding the specified limit**

The program run is diverted to stage 18 to incorporate the above condition, if any pile element is screened in this category and the whole procedure is processed until all these conditions are satisfied.

21. **FORMATION OF THE FORCE VECTOR:**

The force vector \([F]\) is formed with the contact stresses determined in step 19 for the raft together with the imposed loading on the raft.

22. **SOLVING FOR Unknowns:**

The nodal variables of raft such as displacements, rotations & twists are determined. These nodal variables are used to determine the moment per unit length for the raft. The procedure is described in section 3.3(17).

The computer model **PRAES** also provides the facility of incorporating the following conditions:

- **Variable raft thicknesses**
- **Different pile lengths**
The model may be also extended for the following conditions:

- Any shape of raft
- Various pile diameters
- Incremental loading

5.4 COMPARISON OF RESULTS WITH AVAILABLE WORK

In this section, the basic behaviour of the piled raft foundation systems has been analysed and compared with available work carried out by various authors, in order to demonstrate the accuracy of the approach developed herein. The analysis has been carried out with and without free field soil movement conditions.

5.4.1 WITHOUT FREE FIELD SOIL MOVEMENTS

Piled raft systems resting on a flat ground surface are analysed for various raft stiffness values. The influence of the pile compressibility on the overall foundation performance is also studied and discussed. These values are finally compared with available solutions, published in the literature.

In order to study non-linear effects on the foundation behaviour, analyses are undertaken for various pile capacities. In other words, the capacities of piles are limited to a specific value. The load on the pile is limited to its ultimate capacity and any further load on pile exceeding the capacity is redistributed in other elements of the foundation system, which are still in an elastic state. In the present analysis, the pile capacities are incorporated by limiting the shear stress along the pile shaft and also by limiting the stress at the base of the pile. The ultimate pile capacities are considered in terms of the total load applied to the raft. The group efficiency is considered as 1. A relative load capacity factor $\eta$ (Hain and Lee, 1978) is denoted to present the pile capacities as follows:

$$\eta = \frac{n \ P_{ult}}{q \ L_R \ B_R}$$

where,

- $n$ = Number of piles
- $q$ = Load intensity on raft
- $P_{ult}$ = Pile ultimate capacity
\[ L_R \& B_R = \text{Length and width of raft} \]

For example, \( \eta = 1 \) means

\[ P_{\text{eq}} = q \frac{L_R}{n} B_R \]

ie. the capacity of a single pile is equal to the total imposed load divided by the number of piles.

The above factor \( \eta \) is adopted in the analysis to limit the pile capacities and also to facilitate the comparison with Hain and Lee (1978), as it has been adopted in their paper. The analysis is carried out with the various pile capacities corresponds to \( \eta = 0.75, 1.5 \& 3 \) and its influence on the foundation behaviour is analysed and discussed.

### 5.4.1.1 Incompressible Piles

The analysis is carried out with incompressible piles i.e. the pile compression is neglected in comparison with the surrounding soil deformation. In order to undertake the analysis, a 50D×50D square raft with 36 piles is considered, as indicated in Figure (5.4). The length-diameter \( (L_R/D) \) ratio of each pile is adopted as 100. The condition of symmetry is considered in the analysis to reduce the computer memory requirement as well as to reduce the run time. The analysis is carried out for "elastic" as well as for "non linear" conditions. The influence of "soil-pile" slip on foundation behaviour is discussed and presented.

### 5.4.1.1 Load Shared by Piles

The load shared by various piles with varying raft stiffnesses are presented in Figure (5.6). In the case of a rigid raft \( (K_R =10) \), the corner piles are subject to the maximum load and the piles in the close proximity to the center of the raft are subject to minimum load. The load distribution in various piles is significantly influenced by the raft stiffness. With a decrease in the raft stiffness, the load on the corner pile reduces and that on the central piles increases. Other piles share in-between values.

In the case of non-linear condition \( (\eta = 1.5) \), for the rigid raft \( (K_R =10) \), the shear stress along the shaft of the corner piles is subject to the limitation of stress and these elements are allowed to slip (Figure 5.8). As a result of this, the load on the corner pile reduces and
the overall settlement of the raft increases in comparison to the elastic condition (Figure 5.7). Therefore, it is understood that the load carried by the corner piles is higher in the case of the elastic condition and is subject to a reduction with non-linear effects. The reduction of load on the corner piles causes a redistribution effect in the overall foundation system and as a result of that, the load on the other piles increases. In the case of a flexible raft, the non-linear conditions exhibit similar behaviour as the elastic condition, the load distribution on the piles is approximately similar, and shear stresses along the pile shaft are within the limiting value in both cases.

Above results generally agree fairly well with Hain and Lee (1978). Some differences are observed due to the method adopted to incorporate the non-linear effects, as discussed above. In addition, both solutions may involve some inaccuracies due to discretisation.

5.4.1.1.2 Settlement Analysis

The influence of the number of piles on the total settlement is studied and presented in Figure (5.7) for a rigid raft \((K_r=10)\). It can be seen that the settlement is reduced with the increase in the number of piles up to a point, beyond which any further increase in the number of piles is not effective in reducing the settlement. Under elastic conditions, the reduction in settlement is higher compared to the non-linear condition, because the elastic analysis considers the pile capacity to be infinite, whereas the non-linear analysis limits the pile capacity to its ultimate value. In other words, whenever the shear stress along the pile shaft exceeds its limiting "adhesion value", a slip occurs at the soil-pile interface. This slip causes higher pile settlement in comparison with the elastic condition.

The settlement profiles of the piled raft system along BB are shown for \(K_r=0.01\) and \(K_r=10\) in Figures (5.9) and (5.10), respectively. The settlement of an unpiled raft is also presented to indicate the influence of piles in reducing the settlement. For \(\eta = 1.5\), the total settlements are reduced by 38% and 43% in comparison with the unpiled raft for \(K_r=0.01\) and \(K_r=10\). The reduction in settlement is also dependent on the \(L_r/D\) (length to diameter ratio) & \(S/D\) (spacing to diameter ratio). Higher \(L_r/D\) and lower \(S/D\) ratios cause a higher reduction in differential settlement (Figure 5.12).

The differential settlement of an unpiled raft may be reduced by providing piles at strategic locations, generally at the central portion of the raft. It can be seen from Figure (5.9) that the differential settlement of an unpiled raft \((K_r = 0.01)\) is reduced by 60% by incorporating piles underneath it.
These results are compared with those of Hain and Lee (1978) and are found in to be in fairly good agreement. It should be noted that there is a some difference in values, caused because of the different pattern of analysis adopted to limit the pile capacities. In the present analysis, the piles are divided into various elements and the interaction effect in between pile elements is considered, whereas in Hain & Lee (1978), the interaction effect among full piles is considered. In the Hain and Lee paper, the restriction in the pile capacity is incorporated when the load on the pile reaches its ultimate capacity, whereas in the present analysis, the restriction is incorporated by limiting the shear stress along the pile shaft. It should be noted that the stress distribution along the pile shaft is not uniform and some portions of the shaft experience higher stress than others. Therefore, the slip occurs at those portions of the shaft much earlier than when it is loaded to its ultimate capacity. This may induce the non linear effect in the pile behaviour in advance of that in the method adopted by Hain & Lee (1978). As a result of this, some difference in results is expected.

5.4.1.1.3  \textbf{Moment Analysis}

The bending moment distribution along BB (Figure 5.4) has been analysed and is presented in Figure (5.11). The moment distribution in the unpiled raft is also shown in order to compare the influence of piles. The presence of piles induces negative moment in the raft. These moment values are compared with Hain & Lee (1978) and generally agree well. The difference in values may be due to the mesh adopted in the analysis. It should be noted that in the 1970's, the limited capacity of computers was a constraint in analysing a finer mesh, which may influence the accuracy of the earlier moment values quite significantly.

5.4.1.2  \textbf{Compressible Piles}

The performance of piled raft systems is dependent on the load sharing and distribution features of the various components. The structural compression of the pile increases the pile head settlement and influences the load sharing effect quite significantly. Therefore, it is important to address the issue of pile compression in piled raft behaviour.

5.4.1.2.1  \textbf{Settlement Analysis}

The increase in pile compressibility causes higher maximum settlement in a piled raft foundation system, because of its structural compression in relation to the surrounding soil.
The increase in pile compressibility causes higher maximum settlement in a piled raft foundation system, because of its structural compression in relation to the surrounding soil. The influence of the pile compression on the maximum displacement is carried out for two length-diameter ratios ($L_p/D=25$ and 50) and the results are presented in Figure (5.13). It can be seen that for $L_p/D=50$, the increase in the maximum settlement due to the pile compressibility from $K_p=10^3$ to 10 is approximately 20%, whereas for $L_p/D=25$, the increase in the maximum settlement is approximately 12% for the pile raft arrangement analysed. Therefore, the relative compression of the piles is quite significant for higher $L_p/D$ (pile length to diameter) ratios. For longer piles, the pile compression is a significant component of the total settlement in comparison with the incompressible piles. The analysis indicates that the maximum settlement reduces with an increase in the raft stiffness (Figure 5.13). It can also be seen from Figure (5.12) that a higher $S/D$ (space to diameter) ratio is less effective in reducing the maximum settlement of the raft. The settlement of the piled raft with $S/D$ as 25 is approximately similar to the unpiled raft, whereas for $S/D=5$, the piles are more effective in reducing the settlement.

The differential settlement is greatly influenced by the $L_p/D$ ratios, for the various raft stiffnesses. The differential settlement reduces with increasing $L_p/D$ ratio. These aspects of the foundation behaviour have been analysed and discussed in later chapters. The behaviour in a flat ground condition as well as in swelling and shrinking soils is studied and presented.

### 5.4.1.2.2 Load Shared by Piles

Analysis have been carried out for rigid and flexible rafts and the influence of spacing/diameter ($S/D$) ratio on the percentage of load shared by the piles has been studied and is presented below.

It can be seen from Figure (5.14) that the load taken by piles reduces significantly as the $S/D$ ratio increases.

The load shared by the piles increases with an increase in the $K_p$ value. Relatively incompressible piles experience lesser compression and share higher load. It can be seen from Figure (5.15) that, beyond a certain value of $K_p$, any further increase causes no effect on the load sharing behaviour of piles. In other words, the compressibility of the piles is negligible in comparison with the surrounding soil. Conversely, the load carried by the piles reduces as the pile becomes more compressible, because of the increase in the total settlement, as discussed earlier. Therefore, higher compression of pile with respect to surrounding soil causes transfer of load to the raft, and as a result, the load taken by the piles reduces.
5.4.2 With Free Field Soil Movement Condition

A rigid raft supported by four identical piles is considered for the analysis. The piled raft arrangement is shown in Figure (5.5). The behaviour of the piled raft foundation is analysed for the upward and downward ground movement due to the effect of consolidating and swelling soils, respectively. The ground movement may occur due to the change in the effective stress or suction in the soil. The induced force due to the movement of ground is analysed and the its influence on the foundation settlement is presented. The obtained results are compared with those of Poulos (1993).

The following parameters have been adopted to carry out the analysis:

<table>
<thead>
<tr>
<th><strong>Features</strong></th>
<th><strong>Specified Cut Off Values</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Compressive loading (kPa)</td>
</tr>
<tr>
<td>Shear stress at soil-pile interface</td>
<td>50</td>
</tr>
<tr>
<td>Stress at the pile base</td>
<td>450</td>
</tr>
<tr>
<td>Bearing capacity of soil underneath the raft</td>
<td>300</td>
</tr>
</tbody>
</table>

5.4.2.1 Piled Raft in Consolidating Soils

In this section, the foundation behaviour is analysed with the downdrag force induced along the pile shaft by the consolidating soils. The analysis is carried out with the piled raft arrangement as shown in Figure (5.5). The influence of the consolidating soil on the foundation settlement is analysed and presented in Figure (5.16). It can be seen that the settlement of the raft increases with an increase in the ground movement. The raft is rigid, therefore, the settlement is uniform.

In Figure (5.17) the load distribution pattern along the pile shaft for a "flat ground" condition and also due to the effect of consolidating soils is presented. It can be seen that the load on the pile increases due to the induced force along the pile shaft and also due to the changed ground support condition underneath the raft. The load on the pile increases with the increase in the ground movement up to a certain extent, and then stabilises, as slip takes place between the soil and the pile. In Figure (5.17), the load on the piles stabilises at a ground movement of about 40 mm and any further ground movement is unable to induce any further load because of the slip. The maximum load on the pile heads is equal to the superimposed load and the dead weight of the raft.
5.4.2.2  **PILED RAFT IN SWELLING SOILS**

The behaviour of the foundation system has been analysed, incorporating the effect of the upward ground due to swelling soils. The settlement and the stress distribution along the pile shaft for various ground movements are analysed and presented in Figures (5.18) and (5.19) respectively. It can be seen from Figure (5.18) that the initial settlement of the raft is reduced due to the upward force induced by the ground movement. At approximately 80mm of ground movement, the raft experiences upward settlement and increases further with an increase in the ground movement. The upward force on the foundation system is generated by the thrust of the ground underneath the raft and the upward moving soil along the pile shaft.

The upward ground movement induces an uplift force along the pile shaft as well as from the base of the raft. The uplift force along the pile shaft reduces the positive shear stress and beyond an extent, the full pile experiences negative shear stress. It can be seen from Figure (5.19) that at \( S_o = -40 \text{ mm} \), the full pile shaft subjects to downward stress (tensile force) due to the swelling soils.

The analysed results are compared with Poulos (1993) and are found to be in reasonably good agreement.

5.5  **SUMMARY**

The concept of introducing piles underneath a raft is well-accepted, but a real understanding under various ground conditions still requires some attention to achieve an optimised system. Generally, in most cases, the piles are designed to withstand the total superimposed load, neglecting the sharing effect from the raft, but in actual situations, the raft also shares part of the load. Therefore, in order to optimise the foundation system, the effect of mutual interaction between the foundation components should be undertaken and the load-sharing characteristics between the pile and the raft should be established. This may reduce the required number of piles.

The load distribution pattern between various piles is dependent on the relative stiffness of the raft. In the case of a rigid raft, under "elastic" conditions, the corner piles experience the maximum load and the piles in the close proximity to the centre of the raft are subject to the minimum load. For "non-linear" conditions, the corner piles experience slip and
redistribute the load to the other foundation components. Conversely, for a flexible raft under uniform loading, the loads on various piles are approximately uniform, but the settlement is non-uniform with the maximum settlement occurring at the center of the raft.

The load on various piles is also dependent on the spacing (S/D ratio) and the length (Lp/D ratio) of piles. For the analysed case of a 62 pile group, the percentage of load taken by piles for Lp/D=25 is approximately 29% for S/D = 10 and is reduced to 8% for S/D = 20. For the same pile group, in case of a rigid raft, the percentages of load shared by the piles for Lp/D = 100, 50 and 25 are 40%, 20% and 8% respectively, for S/D = 20.

The maximum settlement of a piled raft system may be reduced by introducing piles at strategic locations. The reduction in settlement is dependent on the number of piles up to an extent, beyond which, any further increase in the number of piles is not effective in reducing the settlement. The reduction in settlement is also dependent on the pile capacities. Elastic analysis, which considers the pile capacity as infinite, gives the greatest reduction in settlement, in contrast to non-linear analysis, which limits the raft and the pile capacities to finite values. Piles with lower ultimate capacities, are less effective in reducing the settlement. For the analysed case of a 62 pile group, under elastic conditions, the reduction in settlement for a rigid raft is 52% and 43% with respect to an unpiled raft for elastic and non-linear (η=1.5) conditions, respectively. Thus the reduction in the settlement is 9% less due to the non-linear effects.

The analysis procedure to limit the pile capacities also influences the accuracy of the result. It should be noted that the method of incorporating the ultimate pile capacity, after the load on the pile reaches its ultimate value, will consider foundation behaviour to be elastic until the load "cut off" is incorporated, whereas the method adopted herein, to restrict the pile capacity by limiting the shear stress along the shaft, will give a foundation behaviour which is non-linear much before the pile is loaded to its ultimate capacity. It is understood that the stress distribution along the pile shaft is not uniform and some portions of the shaft experience higher stress than others. Therefore, slip occurs at those portions of the shaft at loads below the ultimate capacity.

The differential settlement of a raft may be reduced by introducing piles at various strategic locations. There are many cases where piles are adopted as settlement reducers. As discussed in previous sections, the differential settlement may be a concern for semi-rigid or flexible rafts. The number of piles and spacing (S/D ratio) may be derived for the required reduction in differential settlement. The pile length to diameter (Lp/D) ratio also influences the settlement reduction feature of the foundation. The differential settlement of
the foundation may be reduced either by adopting a smaller $S/D$ ratio or by considering longer piles.

The percentage load taken by the piles is also influenced by the pile compressibility ($K_p$). Compressible pile experiences higher settlement due to the structural compression in relation to the surrounding soil, and carry lesser loads compared to incompressible piles.

Piled raft behaviour in shrinking and swelling soils has been analysed and discussed in greater detail in Chapters 6 & 7.
Figure 5.1a: Interaction between pile elements

Figure 5.1b: Interaction from stress under raft element to pile elements

Figure 5.1c: Interaction from pile element to soil under raft element

Legend:
- \( I_{pp} \): Pile element to pile element influence factors
- \( I_{rp} \): Raft element to pile element influence factors
- \( I_{pr} \): Pile element to raft element influence factors
- \( I_{ss} \): Soil to soil influence factors
- \( I_{rr} \): Raft element to raft element influence factors (not shown here), as discussed in section 3.2.3.2.

Figure 5.1: Interaction between foundation elements
Figure 5.1a: Interaction between pile elements

Figure 5.1b: Interaction from stress under raft element to pile elements

Figure 5.1c: Interaction from pile element to soil under raft element

Figure 5.1d: Interaction between soil to soil

Legend:

$L_{pp}$: Pile element to pile element influence factors
$L_{rp}$: Raft element to pile element influence factors
$L_{pr}$: Pile element to raft element influence factors
$L_{ss}$: Soil to soil influence factors
$L_{rr}$: Raft element to raft element influence factors (not shown here), as discussed in section 3.2.3.2.

Figure 5.1: Interaction between foundation elements
Figure 5.2a: Raft elements and nodes

Figure 5.2b: Pile elements and nodes

Figure 5.2: Piled Raft arrangement (node and element numbering)
Figure 5.3a: Piled Raft - Plan

Section: X - X

Ground movement under raft

Initial ground condition

Ground movement along pile shaft

Free field soil profile along the pile shaft (pile: 1)

\( \{S_{p_{i}}\} = \{S_{p1}, S_{p2}, S_{p3}, S_{p4}, S_{p5}\} \)

Figure 5.3b: Free field soil movement distribution for shrinking soils

Initial ground condition

Ground movement under raft

Section: X - X

Ground movement along pile shaft

Free field soil profile along the pile shaft (pile: 1)

\( \{S_{p_{i}}\} = \{-S_{p1}, -S_{p2}, -S_{p3}, -S_{p4}, -S_{p5}\} \)

Figure 5.3c: Free field soil movement distribution for swelling soils

Figure 5.3: Free field soil movement distribution (Schematic only)
Raft:

Length \((L_R)\) = 50 \(D\)
Width \((B_R)\) = 50 \(D\)
Poisson's ratio = 0.15

Pile group:

Number = 36
Length = 100 \(D\)
Diameter = \(D\)
Spacing = 8.33 \(D\)

Loading condition: Uniformly distributed load

Figure 5.4: Piled Raft arrangement
Figure 5.5: Piled Raft arrangement
3: Indicates Pile 3

Figure 5.6: Load sharing characteristics of piles

Figure 5.7: Effect of number of piles on settlement
Figure 5.8: Distribution of pile loads with load cut off ($\eta = 1.5$)

Figure 5.9: Distribution of settlement along section BB of Figure 5.4; $K_R = 0.01$
Figure 5.10: Distribution of settlement along section BB of Figure 5.4; $K_r = 10$

Figure 5.11: Distribution of moment along section BB of Figure 5.4
Figure 5.12: Influence of $L_p/D$ and $S/D$ on maximum displacement

Figure 5.13: Effect of pile compression on maximum displacement
Figure 5.14: Percentage of load taken by \( \delta^2 \) pile group

Figure 5.15: Effect of pile compressibility on load sharing characteristics of piles
Figure 5.16: Piled Raft settlement in consolidating soils

Figure 5.17: Stress distribution along pile shaft - Piled Raft in consolidating soils
Figure 5.18: Piled Raft settlement in swelling soils

Figure 5.19: Stress distribution along pile shaft - Piled Raft in swelling soils
CHAPTER - 6

BEHAVIOUR OF PILED RAFT SYSTEMS IN SHRINKING SOILS

6.1 INTRODUCTION

In recent years, considerable attention has been drawn towards the methods of analysis to optimise piled raft foundation systems. In the majority of cases, the piles are designed to withstand the whole superimposed load i.e. the load carried by the raft is totally neglected. However, in the actual situation, the raft also shares part of the load. Therefore, the load sharing between the pile and the raft may be taken into account in order to achieve an effective and economic foundation system. The concept of using the piles as settlement reducers underneath the raft is also increasingly popular and effective (Burland et al, 1977). The strategic placement of piles underneath the raft may be an effective solution to restrict the excessive differential settlement in a raft foundation. Under the normal ground condition (flat ground condition), these practices are generally understood, but in a reactive soil environment, the effect of the cyclic drying and wetting process due to the seasonal moisture variation should be addressed and its influence on the foundation should be thoroughly investigated. The ground movement due to the process of swelling or shrinking of soils changes the state of stress in various parts of the foundation, which may adversely effect the foundation behaviour and its integrity.

In this chapter, the analysis of piled raft foundation systems have been carried out, including the effect of the ground movement due to the process of shrinking of soils. The influence of swelling soils on the foundation behaviour has been analysed and is presented in Chapter 7. As discussed in previous chapters, both piles and rafts are quite significantly affected by the ground movement. The piles are influenced due to the ground movement along the shaft, whereas the raft is influenced due to the relative ground movement underneath its surface. In piled raft systems, this behaviour is rather more complex, because of the interaction between various foundation components. The ground movement induces force in the pile as well as changes in the state of stress under the raft and, due to their structural connection, these forces are redistributed in the various parts of the foundation, depending on their relative stiffnesses. These features influence the foundation behaviour quite significantly and may cause an adverse effect to the foundation performance. Therefore, it is imperative to undertake a thorough investigation to achieve an effective foundation solution.
The load sharing behaviour between the pile and the raft is analysed for a wide range of ground movement. The stress distributions under the raft and along the pile shaft are studied and presented. The influence of the ground movement on the settlements and the moments are also analysed and compared with the "flat ground" condition.

The analysis is also undertaken for a free standing pile group (i.e. a piled raft system with a gap in between the raft and the soil surface) and its behaviour is compared with the piled raft system (i.e. raft in contact with the soil surface). The free standing pile groups are analysed for the uniform ground movement as well as for a mound shaped soil profile. The merits and demerits of these two foundation systems are analysed and presented.

The analysis has been carried out by using the computer program PRAES (Piled Raft Analysis in Expansive Soils), as described in Chapter 5. The behaviour of the piled raft systems, which are constructed in wet ground condition, has been analysed for the subsequent shrinking process due to the drying effect. The relative ground movement underneath the raft and along the pile shaft are considered in the analysis. The movement of the ground under the raft is adopted to indicate varying degrees of the seasonal drying effect along the edge of the raft. Along the pile shaft the ground movement is extended up to a depth below which no seasonal moisture change occurs, and hence no volume change takes place. This is known as "depth of seasonal movement". The soil profile underneath the raft and along the pile shaft are adopted as described in sections 3.2.4.1 and 4.2.3 respectively. An initial analysis is also carried out to indicate the effect of "elastic" and "non-linear conditions". Further analyses are undertaken, including the effect of non-linear conditions at the "soil-structure" interface.

Finally, attention is drawn towards the potential practical implications due to these ground movements and, their effect on the structural integrity of the foundation is briefly discussed and presented.

In order to explain the foundation behaviour at various stages, some aspects, which have been mentioned previously, are readdressed in appropriate sections in order to clarify the description of the foundation behaviour.

6.2 CHARACTERISTICS OF BEHAVIOUR

The analysis is carried out for $2^2$, $4^2$ and $6^2$ pile groups as indicated in Figures 6.1, 6.15 and 6.27, respectively. A comparison between the piled raft system and free standing pile group
is presented and their merits and demerits are discussed in terms of their performance in shrinking soils. The free standing pile group is analysed for both uniform ground movement as well as for a mound shaped soil profile. In the case of a well ventilated soil surface underneath the raft of the free standing pile group, the ground movement is adopted as uniform. Conversely, if there is a lack of ventilation, the soil profile is adopted as mound shaped. The respective movement of the ground at the pile head is distributed along pile shaft up to the depth of seasonal variation. For the purpose of the analyses herein, it is taken as 5m and its distribution with depth to be linear. The parameters, adopted to incorporate the non-linear conditions, are indicated in Table 6.1.

An initial analysis has been carried out to indicate the influence of the soil surface movement on rafts of various dimensionless stiffnesses ($K_R$). Further detailed analyses are presented for $K_R = 1$ and 0.01, which are typical of rigid and flexible raft conditions respectively. The piled raft foundation analysis is undertaken for two load conditions i.e. $P = 0$ and 0.25 $P_{ult}$, where $P = 0$ (unloaded raft) signifies the extreme condition of a lightly loaded foundation and $P = 0.25 P_{ult}$ indicates a normal loading condition. These loads are distributed uniformly over the raft. The influence of various load intensities is also analysed and presented for various ground movements.

6.2.1 ELASTIC AND NON-LINEAR EFFECTS

In order to indicate the non-linear effects on the piled rafts, the results are presented for a $2^2$ pile group (rigid raft) with $L_p/D = 15$ and 50 in Figures 6.2 and 6.3. The analyses are carried out for a wide range of ground movement ($S_o$). As previously discussed, the elastic condition considers the strength at the soil-structure interface as infinite, whereas the non-linear effects consider the following conditions in the analysis:

- A separation takes place between the raft and the soil, when the tensile stress exceeds the limiting value.
- Soil yields locally underneath the raft and at the pile base, when the contact stress exceeds the limiting bearing value.
- A slip takes place at the soil-pile interface, when the shear stress exceeds the limiting value.

In order to incorporate these non-linear effects, the values adopted in the analysis are indicated in Table 6.1. It can be seen from Figure 6.2 that under the lower load condition i.e. low $P/P_{ult}$, the elastic settlement is higher than the settlement under the non-linear condition, because under the application of lower load, the tensile stress in between the raft and the soil and the down drag force along the pile shaft is very high, because no limitation is placed on stresses, whereas under the non-linear condition, the slip and the detachment between the raft and soil cause reduction in the downdrag load. In the case of higher
imposed loads, the non-linear settlement is higher than the elastic settlement, because the settlement due to the superimposed load is dominating the settlement. Under higher applied load, due to the higher settlement, the negative shear stress along the pile shaft, due to the ground movement, changes to positive shear stress as the settlement of the pile at any depth exceeds the ground movement. As a result of this, a redistribution of stress takes place and an excessive stress develops in the stable portion of the pile shaft, which is positive (resistive in nature) and a slip occurs. Due to this, the foundation experiences higher settlement, compared to the elastic condition, because the elastic condition allows no slip in the stable portion of the pile shaft and causes higher resistance against the imposed load.

In real situations, the stresses at the soil - structure interface are finite and the non-linear conditions incorporate this behaviour by accounting for these finite limitations. In contrast, the elastic analysis considers these stresses as infinite, which is not the actual condition. Therefore, further analyses have been carried out, including the effect of non-linear conditions with the limiting values as indicated in Table 6.1.

<table>
<thead>
<tr>
<th>Features</th>
<th>In compression in kPa</th>
<th>In tension in kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil - pile slip</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Soil yielding under raft</td>
<td>300</td>
<td>-</td>
</tr>
<tr>
<td>Soil yielding at pile base</td>
<td>450</td>
<td>-</td>
</tr>
<tr>
<td>Tensile stress under raft</td>
<td>-</td>
<td>20</td>
</tr>
</tbody>
</table>

### 6.2.2 PILED RAFT FOUNDATION SYSTEMS

As mentioned above, the piled raft systems are analysed for three arrangements. These arrangements are indicated in Figures 6.1, 6.15 and 6.27. The objective of adopting these three piled raft arrangements is to indicate the influence of number of piles and their respective location underneath the raft. These cases are analysed for the mound shaped soil profile and presented in the following sections.

### 6.2.2.1 2² PILE GROUP ANALYSIS

A piled raft foundation system with a 2² pile group is analysed, including the effect of the ground movement caused by shrinking soils. The configuration is shown in Figure (6.1). The limiting values adopted to incorporate these non-linear conditions are indicated in Table 6.1. An initial brief analysis is carried out to indicate the influence of the soil surface
movement on various raft stiffnesses ($K_r$). The analysis is undertaken for a wide range of ground movement and the influence on the loads shared by various piles is presented in section 6.2.2.1.1. Further detailed analyses are presented for $K_r = 1 \& 0.01$, which generally represent the rigid and flexible raft conditions.

6.2.2.1.1 Load Sharing Characteristics of Piles

It can be seen from Figure 6.4 that the load on piles increases with an increase in the ground movement. The downward movement of the soil causes a reduction in the contact stress and transfers the load to the piles. In case of a rigid raft, the increase in load is less, compared to a flexible raft, because of the following reasons:

- In case of a rigid raft, the piles share higher load, compared to a flexible raft, in a flat ground ($S_o = 0$) condition. Therefore, the transfer of additional load on the pile due to the ground movement causes the piles to settle more. As a result of this, the load is transferred back to the raft, resulting in higher contact stress underneath the raft. It is discussed in Chapter 4 that the additional settlement of a pile due the downward ground movement is high under the application of higher imposed loads, whereas in the case of a flexible raft, the load on the pile is less in the flat ground condition. Therefore, under a similar ground movement, the piles experience less settlement and share higher load. In other words, the pile is still capable of withstanding the downdrag force as well as the additional imposed load transferred, due to the redistribution effect.

- Due to relatively higher ground movement towards the edge of the raft, the rigid raft transfers higher load towards the central portion of the raft and causes lesser transfer of load to the piles, in comparison to a flexible raft. In the case of a flexible raft condition, the raft is unable to transfer load towards the central portion due to its flexibility. Therefore, the effect is localised and the extent of redistribution is less. As a result of this, the imposed load on the raft is transferred to the piles in the closest proximity.

Under the application of $P = 0.25 \, P_{ult}$, for a flat ground condition, the loads on the pile heads are 42% and 22% of its ultimate capacity for $K_r = 1$ and $K_r = 0.01$, respectively. Due to 80 mm ground movement, the additional loads transferred to the pile are 7% and 42% with respect to the flat ground condition, for rigid ($K_r = 1$) and flexible rafts ($K_r = 0.01$), respectively. As discussed above, the pile in the rigid rafts ($K_r = 1$), under a flat ground ($S_o = 0$) condition, carry higher load, compared to the flexible rafts ($K_r = 0.01$). As a result of this, they take less additional load due to the ground movement.
The stress distribution underneath a rigid raft \((K_R=1)\) are shown in plates (6.1) and (6.2) for 0.25 \(P_{ult}\) load condition. These contours represent a flat \((S_o = 0)\) ground condition as well as the ground surface subjected to 80 mm \((=S_o)\) movement. The increase in the pile load is clearly evident due to the movement of the ground.

It should be noted that these are the loads at the pile heads. The load distribution along the pile shaft with the ground movement is rather more complex and is discussed in more detail in the following section.

6.2.2.1.2 Settlement Analysis

In order to study the settlement behaviour of the foundation, the settlements at the pile head and at the centre of the raft have been analysed and presented. In the case of a rigid raft \((K_R = 1)\), the settlement at the pile head and at the centre of the raft increase with the increase in the ground movement (Figure 6.6). The rigidity of the raft causes the raft to move uniformly and as a result of this, the differential settlement is less and the redistribution of stresses towards the central portion of the raft is high. Therefore, due to the effect of redistribution, the contact stress at the centre of the raft increases (Figure 6.9). As a result of this, the moment at the centre of the raft also increases dramatically (Figure 6.11), which is discussed later.

In the case of a flexible raft \((K_R = 0.01)\) in a flat ground condition, the settlement at the centre of the raft is high in comparison to the pile head. It can be seen from Figure (6.6) that the central settlement increases with the increase in the ground movement. The downward movement of the ground causes higher transfer of load towards the centre of the raft, which causes an increase in the central settlement. The imposed load in the proximity of the pile location is directly transferred to the pile, because the raft is unable to redistribute the stress due to its flexibility. Therefore the transfer of load towards the central portion of the raft is dependent on the raft stiffness. In the case of a flexible raft \((K_R = 0.01)\), for a ground movement of 120 mm, the increase in the pile head settlement is 57% of the central settlement. This phenomenon also causes an increase in the differential settlement.

The displacement contours for a rigid raft \((K_R=1)\) are shown in plates (6.5) and (6.6), representing a flat \((S_o = 0)\) ground condition as well as the ground surface subjected to 80 mm \((=S_o)\) movement. The increase in the settlement is evident due to the downward movement of the ground, as discussed earlier.
It should be noted that in a ground movement situation, the settlement of piled raft foundation systems is dependent on the number of piles and their locations. The presence of piles towards the centre of the raft has a great influence on the settlement profile of the raft. In later sections, it is demonstrated that the settlement of the raft is significantly dependent on the pile arrangement.

### 6.2.2.1.3 Load Distribution along Pile Shaft

The ground movement due to the shrinking soils causes a down drag force along the pile shaft. The pile behaviour in a piled raft systems is quite similar to a single pile as discussed in Chapter 5, but because of its structural connection with the raft, its behaviour is influenced by the raft and its stiffness.

It can be seen from Figure (6.7a) that for an unloaded raft \( P = 0 \), the down drag force along the pile shaft causes a tensile stress at the "pile-raft" interface. The increase in the ground movement causes a minor reduction in the tensile stress. The reduction is due to the change in the soil surface profile underneath the raft. Higher ground movement causes a reduction in the contact area under the raft (towards the centre), which causes higher settlement due to the tensile force generated by the piles. However, the piles experience no further increase in force due to the ground movement, as a slip takes place between the pile and soil. Due to this higher settlement at the centre, a reduction in the tensile stress takes place, which is approximately 18% for a rigid raft \( K_R = 1 \) subject to a ground movement \( S_o \) variation from 40\text{mm} to 120\text{mm}.

In the case of \( P = 0.25 P_{ul} \), it can be seen from Figure (6.7b) that for a rigid raft \( K_R = 1 \), the load on the piles, under a flat ground condition, is higher than that for the flexible raft condition \( K_R = 0.01 \). It has been discussed in section 6.2.2.1.6 that the remaining capacity of a heavily loaded pile may not be adequate to withstand the additional load induced by the ground movement. Therefore, additional load is redistributed to the other foundation components, depending on the raft stiffness. This is a very important feature of the foundation behaviour and is discussed in section 6.2.2.1.6 in detail.

### 6.2.2.1.4 Contact Stress Distribution

The contact pressure under the raft is significantly influenced by the effect of redistribution. The analysis is undertaken for \( P = 0 \) (unloaded raft) and 0.25 \( P_{ul} \) (loaded raft) and the results are presented in Figures (6.8 and 6.9). Due to the ground movement towards the edge of the raft, the load is partly transferred to the piles and towards the central portion of
the raft. The increase in the contact stress towards the central portion of the raft is dependent on the raft stiffness.

It can be seen from Figure (6.8) that an unloaded raft \((P = 0)\) may also experience contact stress due to the tensile force, generated by the piles. These tensile stresses are generated due to the downward movement of the ground along the pile shaft. In the case of an unloaded rigid raft \((K_R = 1)\), the contact stress at the centre of the raft is higher, compared to a flexible raft \((K_R = 0.01)\). It can be seen from Figure (6.8) that the increase in the contact stress for a rigid raft \((K_R = 1)\) is approximately 42% higher with respect to the flexible raft \((K_R = 0.01)\) due to a ground movement of 120 mm. The ground movement induces force along the pile shaft until slip occurs and no further load is transferred to the pile, but the changing profile of the ground underneath the raft alters the ground support condition underneath the raft, which in turn influences the contact stresses quite significantly.

A loaded raft \((P=0.25 \, P_{ulb})\) also experiences an increase in the contact stress at the centre (Figure 6.9) with increasing ground movement. The increase in stress is dependent on the raft stiffness. The increase in the contact stress, for a ground movement of 120 mm, is approximately 12 times and 2 times higher with respect to the contact stresses under a flat ground condition \((S_e=0)\) for rigid \((K_R=1)\) and flexible \((K_R=0.01)\) conditions, respectively. The increase in the contact stress at the centre of the rigid raft is higher in comparison to the flexible raft, because the rigid raft causes a higher transfer of edge stress towards the central portion of the raft. In contrast, the flexible raft is unable to redistribute the stress, and as a result, it experiences a small increase in the central contact stress.

It is obvious that the presence of piles towards the central portion of the raft will affect the increase in the contact stress very significantly. This feature of the foundation behaviour is discussed in later sections of this chapter.

6.2.2.1.5 Moment Analysis

The maximum moment and the moment along section BB (Figures 6.10 to 6.13) are analysed and presented for a wide range of ground movements. In the case of \(P = 0.25 \, P_{ulb}\), it can be seen from Figure (6.11) that for the rigid \((K_R = 1)\) and flexible \((K_R = 0.01)\) rafts, the negative moment in the proximity of the pile increases with the increasing ground movement. However, the positive moment reduces in the case of a rigid raft, and increases for a flexible raft condition. The movement of the ground along BB causes the flexible raft to deflect downward in between the two-pile support due to the imposed load. As a result of this, the positive moment increases with the increasing ground movement. In the case of the rigid raft, the whole raft settles uniformly due to its rigidity and transfers the load towards
the centre of the raft, resulting in a reduction in positive moment. The increases in the negative moment at the pile head for the rigid and flexible rafts due to 120mm ground movement are approximately 40% and 66% in comparison to the flat ground condition (Figure 6.11). As discussed earlier, in the case of a flexible raft condition, the pile experiences a local effect and as a result of this, the influence on moment at the pile head is higher, compared to the rigid raft.

It can be seen from Figure (6.13) that for a rigid raft, the maximum positive moment decreases by 1.3% and maximum negative moment increases by 24% with the imposition of 120 mm ground movement. For a flexible raft, the maximum positive and negative moments increase by 103% and 80%, respectively for a ground movement of 120 mm.

The contours of moment in X-direction \( M_{xx} \) for a rigid raft \( K_R=1 \) are shown in plates (6.3) and (6.4), representing a flat \( (S_o = 0) \) ground condition as well as the ground surface subjected to 80 mm \( (=S_o) \) movement. It can be seen from these plates that a reduction in the positive moment takes place due to the ground movement as indicated in Figure (6.11).

It is important to note that the presence of piles towards the central portion of the raft will change the behaviour of positive moment. In the present situation, the flexible raft experiences an increase in the positive moment with increasing ground movement, because of the downward moving soil and the imposed load on it. The presence of piles in the central portion of the raft will provide a support under the raft and will change the behaviour of the positive moment. Therefore, the pile arrangements and their locations influence the moments and their respective settlements quite significantly. These aspects are discussed in detail in later sections.

6.2.2.1.6 INFLUENCE OF LOADING

An analysis has been carried out for an imposed load of \( P = 0.5 \) \( P_{ult} \) and the effect of higher loading on the foundation behaviour is analysed and discussed. It can be seen from Figure (6.14) that for a rigid raft \( (K_R = 1) \), under the application of \( P = 0.5 \) \( P_{ult} \) load, the load on the pile under the flat ground condition is very high, with nearly 84% of its capacity utilised. Therefore, the pile is not capable of withstanding the additional load induced by the ground movement. It can be seen from Poulos (1990) that the additional settlement of a single pile due to the ground movement is very high under the application of higher imposed load. The pile tends to settle individually, but because of its structural connection and the rigidity of the raft, the pile movement is restricted and the load is redistributed to other foundation components. It can also be seen that due to the tendency for higher settlement, no significant downdrag force is evident along the pile shaft at higher ground
movement. As discussed earlier, as the pile settlement exceeds the value of the ground movement at any depth, the negative shear stress becomes positive.

In the case of a flexible raft \( (K_R = 0.01) \), the load on pile, under a flat ground condition \( (S_o = 0) \), is less in comparison to the rigid raft, and therefore the pile is able to withstand the additional downdrag force induced by the ground movement. In this case, a higher load is transferred towards the centre of the raft, because of the higher load on the raft. The contact stress exceeds the ultimate bearing stress of soil \( (300 \text{ kPa}) \), and as a result of this, the soil yields and redistributes the load to other foundation components. In this process, the raft experiences higher settlement and some "lifted" raft elements resume contact with the soil.

Therefore, in shrinking soil, the effectiveness of piles is dependent both on the imposed load and the raft stiffness. The loads on the various foundation components under a flat ground condition govern the foundation behaviour in shrinking soils.

### 6.2.2.2 4² Pile Group Analysis

In this section, a 4² pile group is analysed, including the effect of the ground movement caused by shrinking soils. The piled raft arrangement is shown in Figure (6.15) and the limiting values adopted are indicated in Table 6.1.

An initial analysis has been carried out to indicate the influence of the soil surface movement on various raft stiffnesses \( (K_R) \). The analysis is undertaken for a wide range of ground movement and the influence on the loads shared by various piles is presented in section 6.2.2.2.1. Further detailed analyses are presented for \( K_R = 1 \) & 0.01, which generally represent typical rigid and flexible raft conditions.

#### 6.2.2.2.1 Load Sharing Characteristics of Piles

It can be seen from Figure 6.16 that for a rigid raft \( (K_R = 10) \), under a flat ground condition \( (S_o = 0) \), the maximum and the minimum loads are shared by corner (pile 3) and central (pile 1) piles, respectively. The difference in load distribution on various piles reduces as the raft becomes more flexible. In the case of a rigid raft, the load on the central pile (pile 1) increases rapidly with the increase in the ground movement, whereas other piles (piles 2 & 3) experience an increase approximately up to \( S_o = 40\text{mm} \) and then a reduction in load takes place.

The downward ground movement along the edges of the raft causes a reduction in the contact stress and a redistribution of stresses in between various foundation components.
As a result of this, additional load is transferred to the piles as well as to the central portion of the raft. Initially the reduction in the contact stress causes an increase of load on all piles \((S_o = 40\text{mm})\), but further increase in the ground movement causes higher load transfer towards the central pile (pile 1), because of the following two reasons:

- Central piles carry minimum load under the flat ground condition \((S_o=0)\), and therefore they have higher remaining capacity to share the additional load induced by the ground movement.

- Central piles experience lesser ground movement in comparison to outer piles.

The above reasons cause the central pile (pile 1) to be stiffer in comparison to other piles, and therefore it attracts higher load. In contrast outer piles experience higher ground movement and beyond a certain movement (about \(S_o=40\text{mm}\)), a reduction in load takes place and the load is transferred to the inner piles.

The downward ground movement also induces additional force along the pile shaft, which plays an important role in redistributing the stresses between various foundation components. As the ground movement is higher towards the outer periphery of the raft, the outer piles (piles 2 & 3) are subjected to higher downdrag forces, compared to the inner piles (pile 1). The movement of the rigid raft is uniform, and therefore, it can be seen from Figure 6.16 that the increase in load on the outer piles (pile 3) is less, compared to the central piles (pile 1). The central pile (pile 1) attracts higher load, because of its higher stiffness, as discussed above. In the case of a flexible raft \((K_r = 0.01)\), the redistribution of load towards the central portion of the raft is not significant, because of the raft flexibility. Therefore, it can be seen from Figure 6.16 that the load on pile 1 is not significantly affected by the ground movement, for the flexible raft condition, in comparison to a rigid raft. The piles towards the outer periphery of the raft are influenced locally due to the lack of the redistribution effect.

The stress distribution between the piles and raft for a rigid raft \((K_r=1)\) condition are shown in plates (6.7) and (6.8), representing a flat \((S_o = 0)\) ground condition as well as the ground surface subjected to \(80\ \text{mm} \ (=S_o)\) movement.

### 6.2.2.2 Settlement Analysis

In order to analyse the settlement behaviour of the foundation, the analysis has been undertaken for \(P = 0\) and \(0.25\ P_u\) load conditions. It can be seen from Figure (6.17) that for an unloaded raft \((P = 0)\), the ground movement causes higher settlement at pile 3 and
lesser settlement at the raft centre. As expected, in case of a rigid raft \( (K_r=1) \) the difference in settlements are less in comparison to the flexible raft \( (K_r=0.01) \). The rigidity of the raft causes the raft to move uniformly, resulting in less differential settlement. As a result of this, higher contact stress develops towards the centre of the raft, in comparison to a flexible raft (Fig 6.21). The ground movement induces a downdrag force along the pile shaft and these forces are transferred to the central portion of the raft, depending on the raft stiffness. The outer piles (piles 2 & 3) experience tensile forces at the pile raft connection (Fig 6.19), as the rigidity of the raft restricts the downward movement of the pile, although, the central pile (pile 1) experiences compressive stress due to the downward force exerted by the external piles (pile 2 and 3). The load distribution aspects are discussed in a later section.

In the case of a loaded raft \( (P = 0.25 P_{ult}) \), it can be seen from Figure 6.18 that for a flexible raft condition \( (K_r = 0.01) \), the difference in settlement between the centre of the raft and at pile head 3 is very high compared to the rigid raft \( (K_r = 1) \). The central settlement is higher than the corner settlement in a flat ground condition \( (S_o = 0) \). The settlement at the pile head 3 increases with increasing ground movement, whereas at the centre, the settlement remains nearly unaffected, because of the following two reasons:

- the centre of the raft is less affected by the shrinking soils and
- the effect of the ground movement along the edge of the raft is not transferred towards the central part of the raft, because of the raft flexibility.

It can also be seen that the difference in settlement reduces with the increase in the soil surface movement and becomes zero at approximately \( S_o = 60mm \) (Figure 6.18). Beyond this point, the difference again increases with further increase in the ground movement, but in the reverse fashion i.e. the displacement at the corner pile (pile 3) becomes higher than the central displacement. In the case of the rigid raft condition \( (K_r=1) \), the central displacement also increases with an increase in the ground movement as the influence of the surface movement is transferred towards the central portion of the raft, due to the raft rigidity.

In the case of a flexible raft condition \( (K_r=0.01) \), the increase in the central settlement and the settlement at the pile head 3 are 3\% and 51\% for a ground movement of 120 mm (Figure 6.18).

The patterns of displacement for a rigid raft \( (K_r=1) \) are shown in plates (6.11) and (6.12). These plates indicate the overall increase in the displacement due to the movement of the ground.
6.2.2.2.3 Load Distribution along Pile Shaft

As discussed in previous sections, in the case of an unloaded raft (\(P = 0\)), the central pile (pile 1) experiences compressive force due to the downdrag force on the outer piles (piles 2 and 3). The outer piles (pile 2 and 3) experience downdrag force due to the downward ground movement along the pile shaft. The ground movement towards the edge of the raft is higher than the centre, and therefore the downdrag force along the outer pile shaft is higher in comparison to the inner piles. It can be seen from Figure (6.19) that the downdrag force along the external pile (pile 3) is higher in the case of a flexible raft (\(K_R = 0.01\)) in comparison to a rigid raft (\(K_R = 1\)). The increase in the downdrag forces for 40mm, 80mm and 120mm ground movements, for a flexible raft, is approximately 25\%, 17\% and 16\% higher in comparison to the rigid raft condition. These forces arise from downward (negative) shear stresses along the pile shaft, caused by the downward ground movement. In the case of a flexible raft condition, the edge piles are loaded more than the rigid raft, because there is less redistribution and a more localised effect. Therefore, it can be seen in Figure (6.19) that the central pile (pile 1) is less loaded compared to a rigid raft.

In the case of a loaded raft (\(P = 0.25 P_{ab}\)), it can be seen from Figure 6.20 that for a rigid raft (\(K_R = 1\)) on a flat ground condition (\(S_o = 0\)), the load on the corner pile (pile 3) is the maximum and is very close to its ultimate capacity; therefore any further load induced by the shrinking soils will cause the pile to settle more, and as a result, the negative shear stress (downdrag force) will become positive. Therefore, it can seen that the downdrag force is absent in pile 3. Under the application of higher imposed load, the additional settlement due to the ground movement is very high. As the pile settlement exceeds the value of the ground movement at any depth, the negative shear stress becomes positive. Although in a piled raft situation, the individual pile may tend to behave independently, because of its structural connection to the raft, it may experience a restriction and as a result of this, a redistribution of stress takes place, which transfers the load on the inner piles (pile 1). Therefore, it can be seen that the increment of load on the inner pile (pile 1) is rapid with the increase in the soil surface movement. The increase in load is higher in the case of a rigid raft (\(K_R = 1\)), compared to a flexible condition (\(K_R = 0.01\)). As discussed, the inner pile (pile 1) experiences lesser ground movement as well as less initial imposed load from the raft, and therefore it acts as a stiffer component compared to the outer piles, thus attracting higher load as a result of redistribution.

It has already been discussed earlier that in the case of a flexible raft, due to its flexibility, the raft is unable to redistribute the stresses towards the central portion of the raft or pile. As a result of this, the piles are influenced locally. Pile 3 is influenced more than pile 1, as it experiences higher ground movement. The increase in load along the pile shaft stabilises
at a limit, beyond which any further increase in the ground movement causes a slip at the "soil-pile" interface and transfers no further load to the pile shaft. The percentage of load shared by the raft decreases with the increase in the ground movement.

6.2.2.2.4 Contact Stress Distribution

The contact stress distribution underneath the raft along AA (Figures 6.21 and 6.22) is presented for various ground movements caused by shrinking soils. In the case of an unloaded rigid raft \((P = 0, K_R = 1)\), the ground movement induces downdrag force along the pile shaft, which redistributes load towards the central portion of the raft (Figure 6.21) and the central pile (pile 1). As discussed earlier, the transfer of load is higher for a rigid raft \((K_R = 1)\) in comparison to a flexible raft \((K_R = 0.01)\). The increase in the ground movement transfers higher load towards the central portion of the raft and causes lift off (i.e. zero contact stress) towards the outer parts of the raft.

For a loaded rigid raft \((P = 0.25 P_{ult}, K_R = 1)\), the contact stress at the centre of the raft is less compared to the flexible raft for a flat ground condition, and therefore, the ground movement causes a rapid increase in the contact stress at the central portion of the raft and a decrease towards the edge. The stress distribution in a flexible raft is similar to the rigid raft, but of lesser magnitude. The increases in the contact stresses at the centre of the raft, for the ground movement of 120 mm are approximately 3.3 and 1.2 times that for the flat ground condition, for a rigid \((K_R = 1)\) and flexible \((K_R = 0.01)\) raft, respectively.

6.2.2.2.5 Moment Analysis

The maximum moment and the moment along section BB in Figure (6.23) are analysed for a wide of range ground movements. In the case of an unloaded raft \((P = 0)\), the increase in moment due to the ground movement is very high for a rigid raft \((K_R = 1)\), compared to a flexible raft \((K_R = 0.01)\). The increase in the contact pressure is high towards the central portion of the raft due to the downdrag force along piles and, due to this effect, the raft experiences tension at the top (i.e. hogging effect), causing an increase in the negative moment. Due to the tensile stress, the raft experiences positive moment in the proximity of the outer piles (pile 3). A minor reduction in the positive moment takes place with the increase in the ground movement, because of the reduction in the tensile stress at pile head 3 (Figure 6.23), as discussed in section 6.2.2.2.3.

In the case of a loaded raft \((P = 0.25 P_{ult})\), the positive moment along BB reduces (Figure 6.24) with an increase in the ground movement. As discussed earlier (Figure 6.18), the difference in settlement between the pile head (pile 3) and the raft centre reduces with the
increase in the ground movement up to a point and again increases with the further increase in the ground movement, but in the reverse fashion. The reverse settlement profile will cause a change in sign of the moment. It can be seen from Figure (6.24) that the positive moment reduces and becomes negative after a certain value of the ground movement. However, there may be some points in the raft where some positive moment will still be present.

In the case of an unloaded raft ($P = 0$), the increase in the maximum negative moment with increasing ground movement is very high up to a point (about $S_o = 40\text{mm}$). Beyond this, the rate of increase is less (Figure 6.25) for both rigid and flexible raft conditions. The initial ground movement causes very high down drag force along the pile shaft; because no slip takes place at the soil - pile interface, therefore the transfer of force to the pile is high. However, further increase in the ground movement causes a slip, causing no further increase in the downdrag force along the pile shaft. As a result of this, the increase in the moment with the ground movement is less.

It can be seen from Figure (6.26) that for a loaded raft ($P = 0.25 \, P_{ab}$), the positive moment reduces and the negative moment increases as the increase in the ground movement. The reductions in the maximum positive moment for a rigid ($K_R = 1$) and flexible ($K_R = 0.01$) raft conditions are 99% and 80%, respectively for a ground movement variation from 0 to 120mm. For the same ground movement, the increase in the maximum negative moment for rigid ($K_R = 1$) and flexible ($K_R = 0.01$) rafts are 85% and 82%, respectively.

The distributions of moment for a rigid raft ($K_R=1$) are shown in plates (6.9) and (6.10). These plates indicate the increase in the negative moment as shown in Figure (6.24).

6.2.2.3 $6^2$ PILE GROUP ANALYSIS

In this section, a $6^2$ pile group is analysed, including the effect of the ground movement caused by shrinking soils. The piled raft arrangement is shown in Figure (6.27) and the limiting values adopted to incorporate the non-linear conditions are indicated in Table 6.1.

An initial brief analysis is carried out to indicate the influence of the soil surface movement on raft behaviour, for various raft stiffnesses ($K_R$). The analysis is undertaken for a ground movement of 80mm and the influence on the loads shared by various piles is presented in section 6.2.2.3.1. Further detailed analyses are presented for $K_R = 1 \, & \, 0.01$, which again are typical of rigid and flexible raft conditions.
6.2.2.3.1 Load Sharing Characteristics of Piles

It can be seen from Figure 6.28 that for a rigid raft \((K_R = 10)\), under a flat ground condition \((S_o = 0)\), the maximum and the minimum loads are shared by corner (pile 6) and central (pile 1) pile, respectively. The difference in load between piles reduces as the raft becomes more flexible. The ground movement causes an increase in load on all piles. The corner piles, in the case of a rigid raft, reach a value of load very close to the ultimate capacity, because of the initial high load in a flat ground condition. Therefore, the loads are transferred towards the central piles, because the central piles are less influenced by the ground movement, as discussed in section 6.2.2.2.1 for a 4\(^2\) pile group. In the case of a flexible raft, as the corner piles are less loaded in a flat ground condition, therefore they attract a higher load in comparison to the rigid raft condition. Generally the mechanism is similar to the 4\(^2\) pile group condition. It should be noted that these are the loads on the pile head only; the distribution of load along the pile shaft is discussed in detail in the following sections.

The downward ground movement induces additional force along the pile shaft, which plays a very important role in redistributing the stresses between various foundation components. As the ground movement is higher towards the periphery of the raft, the outer piles (piles 3, 5 and 6) are subjected to higher down drag force, compared to the inner pile (pile 1). The rigid raft movement is uniform, and therefore, it can be seen from Figure 6.28 that the increase in load on the outer piles is less, compared to the central piles. In the case of a flexible raft \((K_R = 0.01)\), the redistribution of load towards the central portion of the raft is not significant because of the raft flexibility, and also because the relative movement of the ground is less towards the central portion of the raft. Therefore, it can be seen from Figure (6.28) that the load in pile 1 is not significantly affected by the ground movement, for the flexible raft condition.

The stress distribution between the raft and the piles for a rigid raft \((K_R=1)\) are shown in plates (6.13) and (6.14), representing a flat \((S_o = 0)\) ground condition as well as the ground surface subjected to 80 mm \((=S_o)\) movement.

6.2.2.3.2 Settlement Analysis

In order to analyse the settlement behaviour, the analysis is undertaken for \(P = 0\) (unloaded raft) and 0.25 \(P_{ult}\) (loaded raft) load conditions. The settlements at the centre of the raft and at various pile heads are analysed for a wide range of ground movements. It can be seen from Figure (6.29) that for an unloaded raft \((P = 0)\), the ground movement causes higher settlement at pile 6 and lesser settlement at the centre of the raft for rigid as well as for
flexible raft conditions. In the case of a rigid raft \((K_R=1)\), the difference in settlements is less in comparison to the flexible raft \((K_R=0.01)\), obviously, due to the rigidity of the raft. As a result of this, higher contact stress develops towards the centre of the raft, compared to a flexible raft (Fig 6.33). It can be seen from Figure (6.29) that in the case of a flexible raft, the central settlement is almost unaffected by the ground movement, as the raft is unable to transfer the induced forces towards the central portion of the raft, due to its flexibility. The ground movement induces a downdrag force along the pile shaft and these forces are transferred to the central portion of the raft, depending on the raft stiffness. The outer piles (piles 3, 5 and 6) experience tensile stresses at the pile-raft connection (Fig 6.31), as the rigidity of the raft restricts the downward movement of the pile. Due to these tensile stresses at the outer piles, the central pile (pile 1) experiences a compressive stress.

In the case of a loaded raft \((P = 0.25 P_{ult})\), it can be seen from Figure 6.30 that for the flexible raft condition \((K_R = 0.01)\), the difference in settlement between the centre of the raft and at pile 6 is very high compared to the rigid raft \((K_R = 1)\). The central settlement is higher than the corner settlement in a flat ground condition \((S_o = 0)\). The head settlement at pile 6 increases rapidly with the increase in ground movement, whereas at the centre, the increase is much less, because of the reasons discussed in section 6.2.2.2.2.

It can also be seen from Figure (6.30) that the difference in settlements between the raft centre and at the corner pile (pile 6) reduces with the increase in the soil surface movement and becomes zero at approximately \(S_o = 90 \text{ mm}\). Beyond this point, the difference in settlement again increases with the further increase in the ground movement, but in the reverse fashion i.e. the displacement at pile 6 becomes higher than the central displacement. In the case of a rigid raft \((K_R=1)\), the central displacement also increases with an increase in the ground movement as the influence of the surface movement is transferred towards the central portion of the raft, due to its rigidity.

It is important to note that the pattern of settlement is quite similar to the \(4^2\) pile group, but significantly different to the \(2^2\) pile group. The presence of piles towards the centre of the raft influences the settlement of the raft very significantly.

The increase in displacement due to a ground movement of 80 mm for a rigid raft \((K_R=1)\) is shown in plates (6.17) and (6.18).

\subsection*{6.2.2.3.3 Load Distribution along Pile Shaft}

As discussed in previous sections, in the case of an unloaded raft \((P = 0)\), the central piles experience compressive stress due to the downdrag at the outer piles. The ground
movement towards the edge of the raft is high, and therefore the downdrag force along the outer pile shaft is higher in comparison to inner piles. The transfer of load towards the central piles is dependent on the raft stiffness as well as on the ground movement, as discussed earlier. Generally, the behaviour of the foundation system is similar to the $4^2$ piled raft system, but the load distribution is significantly influenced by the location and the number of piles. It can be seen from Figure (6.31) that for an unloaded raft, pile 2 experiences compressive force due to the downdrag force in the outer piles, up to a ground movement of 40 mm. Further increase in the ground movement profile influences this pile and induces a downdrag force, causing a rapid reduction in the compressive force and a development of tensile stress at the “raft-pile” interface. This is a very important feature of the foundation behaviour and requires attention for an effective foundation system. In other words, a pile may experience compressive force in an initial ground movement condition, but later with the higher ground movement, may be subject to a tensile force. This phenomenon should be addressed for a partly constructed foundation, which is not fully loaded and is subjected to a full cycle of seasonal drying and wetting. The tensile stress at the pile-raft interface may cause damage to the foundation system, which may remain unnoticed but may influence its performance adversely.

As discussed earlier, a rigid raft ($K_R = 1$) experiences a higher load increase on the central pile (pile 1). It can be seen from Figure (6.31) that the load on pile 1, due to a ground movement of 120 mm, is approximately 62% higher, in comparison to a flexible raft ($K_R = 0.01$).

In case of a loaded raft ($P = 0.25$ Pult), it can be seen from Figure (6.32) that for a rigid raft ($K_R = 1$) on a flat ground condition ($S_o = 0$), the load on the corner pile (pile : 6) is a maximum and is very close to the ultimate capacity; therefore any further load induced by the shrinking soils will force the pile to settle more, and as a result of that the negative shear stress (downdrag force) will become positive. Therefore, it can be seen that the downdrag force is not evident along the shaft of the pile. The general behaviour of this piled raft arrangement is similar to that discussed in section 6.2.2.2.3.

As discussed earlier, in the case of a flexible raft ($K_R = 0.01$), the raft is unable to redistribute the stresses towards the central portion of the raft or piles. As a result of this, the outer piles are influenced locally. This behaviour can be observed in Figure (6.32).

6.2.2.3.4 Contact Stress Distribution

In the case of an unloaded rigid raft ($K_R = 1$), the ground movement induces downdrag force along the pile shaft, which redistributes load towards the central portion of the raft.
(Figure 6.33) and the central pile (Figure 6.31). The transfer of load is higher for the rigid raft. The increase in the ground movement causes higher load transfer towards the central portion of the raft and causes lift off (i.e. zero contact stress) towards the outer parts of the raft.

For a rigid loaded raft ($P = 0.25 \, P_{\text{ult}}, \, K_R = 1$), it can be seen from Figure (6.33) that the contact stress at the centre of the raft is less compared to a flexible raft for a flat ground condition, and therefore this portion of the raft acts more stiffly and the ground movement causes a rapid increase in the contact stress, which causes a decrease of contact stress towards the edge of the raft. In the case of a flexible raft ($K_R = 0.01$), the load towards the central part of the raft is comparatively higher than for the rigid raft ($K_R = 1$) for a flat ground condition ($S_o = 0$).

6.2.2.3.5 Moment Analysis

In case of an unloaded raft ($P = 0$), the increase in moment due to ground movement is very high for a rigid raft (Figure 6.34), compared to a flexible raft. Similar behaviour has been observed in section 6.2.2.2.5 for the $4^2$ pile group. As discussed previously, the increase in the contact pressure is high towards the central portion of the raft due to the downdrag force along piles and due to this effect, the raft experiences tension at the top (hogging effect), causing higher negative moment.

In the case of a loaded raft ($P = 0.25 \, P_{\text{ult}}$), the positive moment reduces with an increase in the ground movement (Figure 6.35). As discussed earlier, the difference in settlement between the pile head (pile 6) and the raft centre reduces with the increase in the ground movement up to a point, and again increases with a further increase in the ground movement, but in the reverse fashion. It should be noted that the reverse settlement profile will cause a change in sign of the moment. It can be seen that the positive moment reduces and becomes negative after a certain value of the ground movement.

In the case of an unloaded raft ($P = 0$), the increase in the maximum negative moment with the increasing ground movement is very high up to about ($S_o = 80$mm); beyond this, the rate of increase is less (Figure 6.37). The reason for this behaviour is similar to that discussed in section 6.2.2.2.4 for a $4^2$ pile group.

For a loaded raft ($P = 0.25 \, P_{\text{ult}}$), the maximum positive moment reduces and the maximum negative moment increases with increasing ground movement. The reductions in the maximum positive moment for rigid ($K_R = 1$) and flexible ($K_R = 0.01$) rafts are 97% and 86% respectively with respect to the flat ground condition, for a ground movement of
120mm. The corresponding increases in the maximum negative moment for a rigid \((K_R = 1)\) and flexible \((K_R = 0.01)\) raft are 355% and 97% respectively, with respect to the flat ground condition.

The moment contours for a rigid raft \((K_R = 1)\) subjected to 80 mm ground movement are shown in plates (6.15) and (6.16). It can be seen that the negative moment increases at the pile locations, as discussed in relation to Figure (6.35).

### 6.2.3 Free Standing Pile Groups and Pile Raft Systems

- A Comparative Analysis

In this section, the behaviour of a piled raft foundation system is compared with a free standing pile group. As mentioned previously, free standing pile groups can be considered as pile raft systems in which there is no contact between the raft base and the soil surface. The behaviour of free standing pile groups is expected to be different in a reactive soil environment, because of the gap between the raft and the soil surface. The downward ground movement due to a shrinking soil influences the pile shaft only, not the raft, directly. Therefore, the load distribution pattern shall be dependent primarily on the pile behaviour, although the raft stiffness plays an important role in redistributing the effect on the other piles. In contrast, in the case of piled raft foundation systems, the ground movement causes a reduction in the contact stress under the raft as well as influencing the stress along the pile shaft.

It should also be noted that the ultimate capacity of a free standing pile group is less in comparison to piled raft foundation system, because the capacity offered by the raft is absent.

The free standing pile group is analysed for the uniform ground movement as well as for the soil mound. A uniform soil surface movement is adopted to represent a well ventilated gap, where the change in moisture occurs uniformly underneath the raft. Conversely, if the gap is not ventilated, a mound-shaped ground movement is considered to represent the situation.

The purpose of the analysis is to indicate the advantages and disadvantages of maintaining a gap between the raft and the soil surface. The analysis is undertaken for the same load (56 kPa) for the piled raft and free standing pile groups, which is approximately 25% of ultimate capacity \(P = 0.25 P_{ul} \) of the free standing pile group.
The analysis is carried out with the piled raft arrangements indicated in Figure (6.39).

6.2.3.1 Settlement Analysis

In order to analyse the settlement, the analysis is undertaken for rigid ($K_R=1$) and flexible ($K_R=0.01$) raft conditions. It can be seen from Figure (6.40) that for a rigid raft ($K_R=1$) on a flat ground condition ($S_o=0$), the central settlement and the settlement at pile head 3 of the free standing pile group are approximately 4.8% and 5.1% higher than for the piled raft system. In the case of a flexible raft condition ($K_R=0.01$), it can be seen from Figure (6.41) that the settlements are 3% and 5.4% higher with respect to the piled raft system.

In the case of a rigid raft, the free standing group with a soil mound of 40 mm ($S_o=40$ mm) is subject to similar settlements as the piled raft systems. Due to the similar mound conditions, the piles in both foundation systems are subjected to similar downdrag forces. The only difference occurs at the centre of the raft; in the case of piled raft system, the central part of the raft remains in contact of the soil surface, whereas there is no contact in free standing groups. Due to this contact, the piled raft system experiences a minor contact stress due to the presence of central piles. However towards the edge of the raft, both systems experience a similar ground support condition.

It can be seen from Figure (6.41) that for a flexible raft ($K_R=0.01$), the settlement at the centre of the piled raft system is less in comparison to the free standing group. This occurs because, in the case of a piled raft system, the centre of the raft remains in contact of the mound, which reduces the settlement, whereas there is no contact in the case of a free standing group. The outer piles experience similar settlement, because they are subjected to similar ground movements and the influence of the edge condition is not transferred towards the centre of the raft. Therefore, the settlement of the outer pile (pile 3) is nearly same in both foundation systems, when subjected to the mound-shaped soil profile.

The settlement of the free standing group experiencing uniform ground movement is higher, compared to the piled raft systems as well as for the free standing groups subjected to the mound-shaped soil profile. It can be seen from Figure (6.41) that for a flexible raft ($K_R=0.01$) on a free standing group subjected to uniform ground movement, the increase in the central settlement is much higher, compared to the free standing group and a piled raft system subjected to mound shaped soil surface. For a ground movement of 120mm, the central settlement of the free standing group subjected to an uniform ground movement is respectively 9% and 7% higher with respect to the pile raft and the free standing pile group subjected to soil mound condition. For the same ground movement, the settlement at the corner pile (pile 3) of the free standing group subjected to uniform ground movement is
14% and 8% higher than the piled raft and free standing groups subjected to a mound shaped soil movement. The pile towards the edge of the raft experiences similar ground movement in all cases, due to the direct exposure to the atmosphere. In the case of a flexible raft ($K_R = 0.01$), the raft is unable to transfer the effect from the centre to the edge, therefore pile 3 experiences similar settlement. However, in the case of a rigid raft ($K_R = 1$), due to the effect of redistribution, the behaviour is little different.

**6.2.3.2 Load Distribution along the Pile Shaft**

It can be seen from figure (6.42) that, in the case of a rigid raft ($K_R=1$) on a flat ground condition ($S_o = 0$), the corner piles and the central piles of the free standing pile group experience maximum and minimum loads, respectively. The piled raft system also exhibits similar behaviour, but the magnitude of load taken by piles is less in comparison to a free standing group, because the raft also shares part of the imposed load. The inner piles in a free standing pile group with the rigid raft act in a stiffer manner than the outer piles, for the following two reasons:

- the central piles carry minimum load under the flat ground condition ($S_o=0$), therefore they have higher remaining capacity to withstand the additional load induced by the ground movement
- the central piles experience lesser ground movement in comparison to outer piles, in the case of mound shaped ground movement.

As result of this, the mound shaped ground movement causes higher transfer of load towards the central piles.

As discussed earlier, in the case of a mound-shaped ground movement, the raft and the piles towards the edge of the raft experience similar magnitude of the ground movement in both types of foundation. Therefore it can be seen from Figure (6.42) that the load distributions along piles 2 and 3 are nearly the same for both types of foundation system. In contrast, the load distribution along pile 1 is different, because of the difference in the central support condition. In the case of a piled raft system, the central portion of the raft remains in contact with the ground surface, and therefore the ground surface shares some load and transfers the rest of the load to the piles, mainly to the central piles. For free standing pile groups, due to the gap between the raft base and the soil surface, a greater load is transferred to the inner piles.

In the case of uniform ground movement under the free standing pile group, it can be seen from Figure (6.42) that the increase in the load on piles due to the ground movement is
higher than for the piled raft system as well as the free standing group subjected to mound-shaped ground movement. The uniform ground movement causes the same downdrag force in all piles, but the pile behaviour is dictated by the initial load imposed from the raft under a flat ground condition. As discussed previously, for the same ground movement condition, a single pile, under the application of a higher imposed load, experiences higher additional settlement in comparison to a single pile under lower load. Therefore, the piles, which experience lower loads in a flat ground condition, act in a stiffer manner in comparison to other piles and attract higher load. Therefore, it can be seen from Figure (6.42) that the inner pile (pile 1), in the case of a rigid raft ($K_R = 1$), experiences higher downdrag force in comparison to piles 2 & 3, because the inner pile is less loaded under a flat ground condition.

In summary, the uniform ground movement induces a higher downdrag force in comparison to mound-shaped ground movement in a free standing pile group.

6.2.3.3 Moment Analysis

In this section, the maximum moment and the moment along section BB (see Figure 6.44) are analysed for a wide range of ground movements. The analysis is undertaken for rigid ($K_R = 1$) and flexible ($K_R = 0.01$) raft conditions. A comparison between the piled raft foundation system subjected to mound-shaped ground movement, and a free standing pile group subjected to uniform and mound shaped ground movement, is presented below.

As discussed in previous sections, in the case of a piled raft foundation system, due to the load sharing characteristics of the raft, piles are less loaded in comparison to free standing group under a flat ground condition. Therefore they experience smaller settlements (Figures 6.40 and 6.41) and lesser imposed loads (Figures 6.42 and 6.43). As a result of this, it can be seen from Figure (6.44) that the positive moment for a piled raft system along BB is higher in comparison to a free standing pile group for the flat ground ($S_o = 0$) condition. Conversely, the negative moment at pile 3 in the piled raft system is less than for the free standing pile group.

In the case of a piled raft system subjected to the mound-shaped ground movement, for a rigid raft ($K_R = 1$), the reduction in the positive moment with increasing ground movement, is higher than for the free standing pile group, because of the support provided by the mound at the raft centre, which is absent in the free-standing pile group. The moment along BB is quite similar in all cases (see Figure 6.45), because the edge of the raft and piles experience similar ground support conditions.
In the case of a rigid raft ($K_r = 1$) on a flat ground condition ($S_c = 0$), the maximum positive moment for the piled raft system is 19% higher than for the free standing pile group, because of the development of contact stress underneath the raft. With 80 mm increase in the ground movement, the maximum positive moment in the piled raft system is 5% and 38% lower with respect to the free standing pile groups subjected to the mound and uniform ground movements, respectively. The reduction in the positive moment in the case of a free standing pile group experiencing uniform ground movement is less than for the mound shaped ground movement.

6.3 PRACTICAL IMPLICATIONS

The movement of the ground due to the shrinking effect induces a downdrag force along the pile shaft and also changes the ground support condition underneath the raft. This process influences the stress condition of the foundation quite significantly. The extent of influence is dependent on various soil and foundation parameters, but the raft thickness and the imposed loading also play a very important role in these situations. The stiffness of the raft controls the distribution of stress between various foundation components and the imposed load counteracts the possibility of negative tensile stress at the pile-raft interface. These features influence the load distribution between various piles as well as the settlement, contact stress underneath the raft and the moment. In some extreme cases, a tensile stress may be induced at the pile raft interface. The analysis and discussion indicate the following aspects, which may influence the overall foundation performance in a practical point of view:

- The relative movement of the ground influences the settlement profile of the foundation very significantly. Under normal loading conditions, the settlement at the centre of the foundation is higher than the corner settlement, but with the increasing ground movement, the settlement towards the corner increases. There may be a situation in which the corner settlement becomes higher than the central settlement, in other words, the raft experiences a total change in the settlement profile. Obviously, the changing state of the foundation settlement profile will affect the superstructure and may cause some consequent structural settlement damage. Therefore, during analysis of such a foundation system, proper consideration should be given to this phenomenon.

- The analysis also indicates that an unloaded or lightly loaded raft may experience a tensile stress at the pile-raft interface. This draws the attention towards the cases, where the construction is carried out up to the foundation level and then stopped for a significant period of time, for various reasons. This condition may be regarded as a
lightly loaded foundation condition. In this situation, the foundation may experience a full cycle of seasonal wetting and drying, causing the supporting soil to be influenced by the shrinking effect. This may induce a tensile stress at the pile-raft interface causing damage to the piled-raft connection, which may remain visually undetected. These tensile stresses are significantly high in a swelling soil condition, which is discussed in the next chapter.

- The above case may cause the raft to deflect prior to the actual superstructure loading. Therefore, the superstructure components may be constructed on a deflected raft, which will undergo additional movements under the imposed load from the superstructure. This may induce undesirable stresses in the structure. Therefore, these features should be addressed during the analysis stage when the foundation system is in a reactive soil environment.

6.4 SUMMARY

The ground movement due to the shrinking soils has a significant impact on the foundation behaviour. The relative movement of the ground under the raft causes a change in the state of the contact stress as well as the distribution of stress along the pile shaft. This phenomenon influences the load sharing pattern between the raft and the piles very significantly. The difference in behaviour is dependent on the geometry of the foundation, and the number and location of the piles underneath the raft. The size of the raft influences the shape of the mound formation under the raft and the location of the piles influences the magnitude of the downdrag force along the pile shaft. A pile towards the edge of the raft experiences higher downdrag force in comparison to an inner pile. These downdrag forces influence the foundation components in terms of their performance and effectiveness very significantly. In order to indicate the influence of the number of piles and their respective location, various pile groups have been analysed in this chapter. The settlement of piles increases with increasing ground movement due to the shrinking soil, but the pattern of settlement depends on the location of piles and the raft stiffness.

The change in the state of stress due to the ground movement causes a redistribution of load in between various foundation components. In the case of a loaded raft, the load on the pile increases with an increase in the ground movement. The downward movement of the ground causes a reduction in the contact stress underneath the edge of the raft, and as result of this the imposed load is transferred to the piles as well as towards the central portion of the raft, depending on the raft stiffness. Due to this effect, an increase in the contact stress and the load on piles towards the centre of the raft takes place. In case of a $2^2$ pile group,
where there is no piles towards the central portion of the raft, the redistributed load is
directly transferred to the ground, resulting in a dramatic increase in the contact stress. In
these situations, the presence of piles towards the centre of the raft attracts the majority of
redistributed load towards it, as the pile components act more stiffly in comparison to the
ground surface and result in a lesser increase in contact stress at the central raft. The
distribution of load on various piles is dependent on the location of the piles as well as on
the initial load condition of the piles in a flat ground condition ($S_0 = 0$). A less loaded pile
experiences higher transfer of load towards it, because higher additional load carrying
capacity of any foundation component results in a stiffer response which attracts higher
load towards it.

As the contact stress reduces due to the ground movement and becomes negative, it may
exceed the value of adhesion between the raft and the soil, and a detachment takes place at
the "soil-raft" interface.

In the case of a flexible piled raft in a flat ground condition ($S_0 = 0$), the central settlement
is higher than the corner settlement under uniform imposed load condition. The settlement
of the outer piles increases with increasing ground movement, whereas at the centre, the
settlement is less affected by the ground movement, because of the raft flexibility. The
differential settlement reduces with an increase in the soil surface movement and becomes
zero at some value of the ground movement. Beyond this point, the difference in settlement
again increases with the further increase in the ground movement, but in the reverse fashion
i.e. the displacement of the outer pile becomes higher than the central displacement. In the
case of the rigid raft condition, the central displacement also increases with an increase in
the ground movement, because the influence of the ground movement at the edge is
transferred towards the central portion of the raft, due to the raft rigidity.

The movement of the ground along the pile shaft induces a downdrag force, which
influences the pile shaft as well as the whole foundation behaviour due to the effect of
redistribution. The magnitude of the downdrag force along the pile shaft is dependent on
the ground movement, its extent and the imposed load on the pile in the flat ground
condition, as discussed earlier.

The behaviour of an unloaded raft is significantly different in comparison to a loaded raft.
The downward movement of the ground induces a downdrag force along the pile shaft and
due to this effect, the piles experience downward movement. The corner piles experience
higher ground movement in comparison to the central piles. Therefore, in the case of a rigid
raft, the raft restricts the movement of the corner pile, because of its rigidity and, as a result
of this, tensile stresses develop at the "pile-raft" interface of the corner pile. These tensile
stresses impose downward force on the raft, which is transferred towards the central portion of the raft as well as towards the central piles. In other words, the downdrag force along the corner pile shaft imposes a downward load on the raft, which induces compressive stress on the central pile as well as to the central part of the raft. The transfer of load towards the central portion of the raft is dependent on the raft rigidity, as discussed previously. A rigid raft transfers higher load in comparison to a flexible raft.

The movement of the ground causes a reduction in the maximum positive moment and an increase in the maximum negative moment. The variation in moment is dependent on the raft stiffness as well as on the number and the location of the piles. In a flat ground condition, the moment in a loaded raft at the corner pile head is generally negative, and increases with the increase in the ground movement.

In a flat ground \((S_o=0)\) condition, the load on piles in the case of free standing group is higher than the piled raft system, because the raft shares no load. Therefore, the settlement is higher in the case of a free standing group.

In the case of mound-shaped ground movement, the central portion of the piled raft remains in contact with the soil surface, whereas the raft in the free standing pile group has no contact. Therefore, in the free standing pile group, the load on the central pile is higher, compared to piled raft foundation system, for rigid as well as for flexible raft conditions. The piles in both the foundation systems experience similar magnitude of the ground movement, excepting that the raft at the centre is subject to a contact stress in the piled raft system, which is not present in free standing pile group. Due to the small load sharing effect at the central part of the raft, for a rigid raft condition \((K_R=1)\), the settlement of the piles in a free standing group is little higher compared to the piled raft system. In contrast, in the case of a flexible raft \((K_R=0.01)\), the outer piles experience the same settlement for both foundation systems, but the settlement at the raft centre is high for the free standing pile group in comparison to the piled raft system.

The behaviour of a free standing pile group subjected to a uniform ground movement is significantly different in comparison to the piled raft systems subjected to mound-shaped ground movement. The downdrag force along the central pile (pile 1) is very high in case of the free standing pile group, because it experiences the same magnitude of the ground movement as the outer piles. The outer piles in both foundation systems experience a similar effect, because they are subject to a similar magnitude of ground movement. However, the net effect is dependent on the raft stiffness, which redistributes the effect to various foundation components. In this situation, for a rigid raft, the settlements of the piles in the case of a free standing pile group are nearly the same due to the raft rigidity, and
these settlements are higher than the piled raft system. In case of a flexible raft, the outer piles experience similar settlements in both foundation systems, as they subject to the same ground movement, whereas the central pile of the free standing group experiences higher settlement in comparison to the piled raft system and this effect is not distributed due to the raft flexibility. In the case of uniform ground movement, the settlements of piles for the free standing pile group are higher in comparison to the piled raft system, because the central piles are also subject to the same ground movement as the outer piles.

In the case of a free standing pile group subjected to uniform ground movement, the central piles experience the same magnitude of ground movement as the corner piles, whereas, for mound shaped ground movement, the central piles experience lesser ground movement in comparison to the corner piles. As a result of this, the central piles act more stiffly and share higher load after redistribution. In a flat ground condition ($S_o = 0$), the central piles are less loaded in a rigid raft condition, and therefore they act more stiffer and withstand a higher downdrag force. In the case of a flexible raft, the imposed load on the central pile is higher than the rigid raft, and therefore, it experiences higher settlement and withstands lesser downdrag force.

The positive moment along a section AA near the centre of a piled raft system is higher than in the free standing pile group in a flat ground condition, whereas the negative moment is less in the piled raft system. With the increasing ground movement in a mound shaped profile, the difference in moment between the piled raft system and the free standing group reduces, because both the foundations experience similar ground movements along AA, any differences are due to the contact of the piled raft system with the soil, which also shares the load. The edge of the foundation exhibits similar behaviour, because in both cases, the raft is uplifted due to the ground movement. Therefore, there is little or no difference, either in moment or in the settlement.
Piled Raft Analysis

$2^2$ Pile Group Analysis
Soil parameters:

- Poisson's ratio = 0.3
- Modulus = 20 MPa

Pile:

- Length of pile = 20 m
- Diameter = 1 m
- Pile compression = 10^1
- Pile modulus = 2 × 10^4 MPa

Raft:

- Length of raft = 7 m
- Width of raft = 7 m
- Raft modulus = 2 × 10^4 MPa

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**Figure 6.1 a:** Plan (Quarter portion only)

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**Figure 6.1 b:** Section XX (Quarter portion only)

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**Figure 6.1:** Piled Raft Arrangement (2^2 pile group), Quarter portion only
Figure 6.2: Elastic and Non Linear Analysis \((L_p/D = 15)\)

Figure 6.3: Elastic and Non Linear Analysis \((L_p/D = 50)\)
2² Pile Group Analysis

Figure 6.4: Load sharing characteristics of pile

2² pile group analysis

Figure 6.5: Settlement Analysis (Applied load = 0, K_R = 1)
2^2 Pile Group Analysis

Shrinking Soil Analysis
Settlement Analysis
Applied Load = 0.25 \( P_{ult} \)

 Settlement at raft centre
 Settlement at pile head

Figure 6.6: Settlement at raft centre and at pile head

\[ L = L_0 \]

2^2 Pile Group Analysis

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Figure 6.7: Distribution of load along pile shaft
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Figure 6.9: Distribution of contact stress along AA (Applied load = 0.25 $P_{uk}$)
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Figure 6.11: Moment distribution along BB (Applied load = 0.25 \(P_{ub}\))
2^2 pile group analysis

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2^2 pile group analysis

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**Plate 6.6: Displacements in metres** ($P = 0.25 P_{ub} S_o = 80\text{mm}$)
Piled Raft Analysis

4² Pile Group Analysis
Soil Parameters:
- Soil Poisson's ratio = 0.3
- Soil modulus = 20 MPa

Pile:
- $L_p =$ Length of pile = 20 m
- $D =$ Pile diameter = 1 m
- $K_p =$ Pile compression = $10^3$
- $E_p =$ Pile modulus = $2 \times 10^4$ MPa

Raft:
- $L_R =$ Length of raft = 20 m
- $B_R =$ Width of raft = 20 m
- $E_R =$ Raft modulus = $2 \times 10^4$ MPa

**Figure 6.15a**: Plan (Quarter portion only)

**Figure 6.15b**: Section XX (Quarter portion only)

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Figure 6.17: Settlements at raft centre and at pile head (Applied load = 0)
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Fig. 6.19a: Load distribution along pile 1
Fig. 6.19b: Load distribution along pile 2
Fig. 6.19c: Load distribution along pile 3

Figure 6.19: Load distribution along pile shafts (Applied load = 0)

Fig. 6.20a: Load distribution along pile 1
Fig. 6.20b: Load distribution along pile 2
Fig. 6.20c: Load distribution along pile 3

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4. Pile group analysis

**Figure 6.23**: Moment along section BB (Applied load = 0)

4. Pile group analysis

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Piled Raft Analysis

$6^2$ Pile Group Analysis
Soil parameters:

- Soil Poisson's ratio = 0.3
- Soil modulus = 20 MPa

Pile:

- \( L_p \) = Length of pile = 25 m
- \( D \) = Pile diameter = 1 m
- \( K_p \) = Pile compression = \( 10^3 \)
- \( E_p \) = Pile modulus = \( 2 \times 10^4 \) MPa

Raft:

- \( L_R \) = Length of raft = 45 m
- \( B_R \) = Width of raft = 45 m
- \( E_R \) = Raft modulus = \( 2 \times 10^4 \) MPa

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Figure 6.27b: Section XX

Figure 6.27: Piled Raft Arrangement (6² pile group), Quarter portion only
6\textsuperscript{2} Pile group analysis

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6\textsuperscript{2} Pile group analysis

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**Fig. 6.31c:** Load distribution along pile 3

**Fig. 6.31d:** Load distribution along pile 4

**Fig. 6.31e:** Load distribution along pile 5

**Fig. 6.31f:** Load distribution along pile 6

**Figure 6.31:** Load distribution along pile shafts (Applied load = 0)
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- Fig. 6.32c: Load distribution along pile 3

- Fig. 6.32d: Load distribution along pile 4
- Fig. 6.32e: Load distribution along pile 5
- Fig. 6.32f: Load distribution along pile 6

Figure 6.32: Load distribution along pile shafts (Applied load = 0.25 P_{ult})
6\textsuperscript{2} Pile group analysis

**Figure 6.32**: Contact pressure distribution along AA (Applied load = 0)

6\textsuperscript{2} Pile group analysis

**Figure 6.33**: Contact pressure distribution along AA (Applied load = 0.25 \( P_{ab} \))
$d^2$ Pile group analysis

![Graph showing moment analysis for different soil analyses.](image)

**Figure 6.34**: Moment analysis along BB (Applied load = 0)

$6^2$ Pile group analysis

![Graph showing moment analysis for different soil analyses.](image)

**Figure 6.35**: Moment analysis along BB (Applied load = 0.25 $P_{ult}$)
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Figure 6.37: Maximum moment analysis (P = 0)

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Plate 6.14: Stress distribution (in kPa) between piles and raft ($P = 0.25 P_{ult} S_o = 80\text{mm}$)
Plate 6.15: Moment (kN-m/m) in X-direction ($P = 0.25 \, P_{ult} \, S_o = 0$)

Plate 6.16: Moment (kN-m/m) in X-direction ($P = 0.25 \, P_{ult} \, S_o = 80\text{mm}$)
Plate 6.17: Displacements in metres ($P = 0.25 \ P_{atm} \ S_o = 0$)

Plate 6.18: Displacements in metres ($P = 0.25 \ P_{atm} \ S_o = 80 \ mm$)
Piled Raft Foundation System

and

Free Standing Pile Group

A Comparative Study
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Behaviour of Piled Raft Systems in Shrinking Soils

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CHAPTER - 7

BEHAVIOUR OF PILED RAFT SYSTEMS IN SWELLING SOILS

7.1 INTRODUCTION

In this chapter, the behaviour of piled raft foundation systems in swelling soils has been analysed and presented in order to address their performance in soil subjected to relative ground movement. The response of the foundation system is studied, incorporating the movement of the ground under the raft as well as along the pile shaft. As discussed in the previous chapter, in recent years, a great deal of attention is drawn towards the methods of analysis to optimise the piled raft foundation systems. In situations where foundation is constructed in a dry state of soil and the swelling of the soil is a potential issue, the "soil-structure" interaction should be thoroughly investigated in the foundation analysis. The upward movement of the ground support underneath the raft as well as along the pile shaft should be accounted for. Due to the swelling of soils, the raft experiences an upward thrust from the bottom and the piles are subject to an upward force along the shaft. This phenomenon may influence the behaviour of the foundation dramatically.

The performance of piles used as settlement reducers may also be influenced significantly by the upward movement of the soil due to the induced stress along the shaft and the thrust underneath the raft. Due to this effect, the settlement reducing objective of these piles may not be achieved.

In this chapter, the analysis of the piled raft foundation system is carried out, including the effect of ground movement due to the process of swelling of soils. As discussed in previous chapters, piles and rafts are quite significantly affected by the upward ground movement caused by swelling soils. The piles are influenced due to the relative ground movement along the shaft, whereas the raft is influenced due to the relative ground movement underneath its surface. In piled raft systems, this behaviour is rather more complex, because of the complicated interaction between various foundation components. The ground movement induces an upward force in the pile and also changes the state of stress under the raft. Due to the structural connection between the piles and the raft, these forces are redistributed in the various part of the foundation, depending on their relative stiffnesses. These features influence the foundation behaviour quite significantly and may
cause an adverse effect on the foundation performance. Therefore, it is imperative to undertake a thorough investigation to achieve an effective foundation solution in these situations.

The load sharing behaviour between the pile and the raft is analysed for a wide range of ground movement. The stress distributions under the raft and along the pile shaft are studied and presented. The influence of the ground movement on the settlements and the moments is also analysed and compared with the flat ground condition.

The analysis is also undertaken for a free standing pile group and its behaviour is compared with a piled raft system. The free standing pile groups are analysed for a uniform ground movement as well as for an edge heave soil profile. The merits and demerits of these two foundation systems are analysed and presented.

The analysis is carried out by using the computer program PRAESP (Piled Raft Analysis in Expansive Soils), as described in Chapter 5. The behaviour of the piled raft foundation systems, which are constructed in a dry state of soil, has been analysed for the subsequent wetting of soils, which causes swelling. The relative movement of the soil surface under the raft and along the pile shaft have been considered in the analysis. The swelling effect along the pile shaft has been extended up to the “depth of seasonal movement”. The soil profiles underneath the raft and along the pile shaft are adopted as described in sections 3.2.4.1 and 4.2.3 respectively. An initial analysis is also carried out to indicate the effect of elastic and non-linear conditions. Further analyses are undertaken to assess the effect of non-linear conditions at the soil structure interface.

Finally, attention is also drawn towards the potential practical implications due to these ground movements, and their effect on the structural integrity of the foundation, is briefly discussed.

In order to explain the foundation behaviour at various stages, some aspects which have been mentioned previously, are readdressed in order to detail the behaviour of the foundation.

### 7.2 CHARACTERISTICS OF BEHAVIOUR

The analysis is carried out for $2^2$, $4^2$ and $6^2$ pile groups as indicated in Figures 7.1, 7.14 and 7.26, respectively. A comparison between the piled raft system and free standing pile group is presented and their merits and demerits are discussed in terms of their performance in the
swelling soils. The free standing pile group is analysed for uniform as well as an edge heave ground movements. In the case of a well ventilated soil surface underneath the raft and the free standing pile group, the ground movement is adopted as uniform. Conversely, if there is a lack of ventilation, the soil profile is adopted as an edge heave. The respective movement of the ground at the pile head is distributed along pile shaft up to the depth of seasonal variation. For the purpose of the analyses herein, it is taken as 5m and its distribution with depth to be linear. The parameters adopted to incorporate the non-linear conditions are indicated in Table 7.1.

An initial analysis has been carried out to indicate the influence of the soil surface movement on rafts of various dimensionless stiffnesses \( K_R \). Further detailed analyses are presented for \( K_R = 1 \) and 0.01, which generally are typical of rigid and flexible raft conditions. The piled raft foundation analysis is undertaken for two load conditions i.e. \( P = 0 \) and 0.25 \( P_{\text{ul}} \), where \( P = 0 \) (unloaded raft) signifies the extreme condition of a lightly loaded foundation and \( P = 0.25 P_{\text{ul}} \) indicates a normal loading condition.

### 7.2.1 Elastic and Non-linear Effects

In order to indicate the non-linear effects on the foundation system, the results are presented for a 2\(^2\) pile group with \( L/p/D = 15 \) and 50 in Figures 7.2 and 7.3. The analysis is carried out for a wide range of ground movement \( (S_e) \). As previously discussed, the elastic condition considers the stresses at the soil-structure interface to have no upper limit, whereas the non-linear effects consider the following conditions in the analysis:

- A separation takes place between the raft and the soil, as the tensile stress exceeds the limiting value.
- Soil yields locally underneath the raft and at the pile base, in the case where the bearing stress exceeds the limiting value.
- A slip occurs at the soil-pile interface, as the shear stress exceeds the limiting value.

In order to incorporate these non-linear conditions, the values adopted in the analysis are indicated in Table 7.1. The swelling soil exerts a thrust underneath the raft and also induce an upward force along the pile shaft. Under low imposed load condition \( (P \approx 0) \), the swelling force causes an upward foundation displacement, because the low imposed load is unable to counteract the upward force. With increasing imposed load, the swelling force is counteracted and the upward displacement is nullified. It can be seen from Figure 7.2 that in the case of an unloaded foundation i.e. \( P/P_{\text{ul}} = 0 \), the upward displacement is a maximum, and reduces with an increase in imposed load. In the case of the non-linear condition, the following phenomena occur:

- A slip takes place at the soil-pile interface in the swelling as well as in the stable zone.
  - Slip in the swelling zone reduces the uplift force along the pile shaft, whereas the slip in
the stable zone reduces the pile resistance against the uplift force. The stable zone is defined as the portion of the pile which is not directly subjected to the ground movement.

- Yielding of the soil takes place underneath the raft, as the contact stress exceeds the limiting value. The contact stress increases due to upward movement of the swelling soils.

The combination of the above non linear effects influences the foundation quite significantly, compared to the elastic condition. In the case of a loaded raft (Figures 7.2 and 7.3), the settlement under the elastic condition is less than for the non linear condition, because there is no slip at the soil - pile interface and no yielding of soil under the raft, which results in a higher upward thrust to the foundation. Conversely, in the case of the non linear condition, due to the slip and the yielding of soil, less upward force is exerted on the foundation, and as a result, the downward displacement is high. The difference in settlement increases with the increase in the imposed load. It can be seen from Figure (7.2) that the non linear settlement is 25% higher, compared to the elastic settlement for a ground movement of 80 mm, under the application of 0.75\( P_{ult} \). Under an imposed loading of 0.25 \( P_{ult} \), for the same ground movement, the non linear settlement is 20% higher compared to the elastic condition. Therefore, it can be seen that the elastic and the non linear systems exhibit significantly different behaviour.

In real situations, the stresses at the soil-structure interface are finite and the non linear conditions incorporate this behaviour by accounting for these limitations. Therefore, the remaining analyses in this chapter include the effect of non-linear conditions, with the limiting values as indicated in Table 7.1.

| Table : 7.1 |

<table>
<thead>
<tr>
<th>Features</th>
<th>In compression in kPa</th>
<th>In tension in kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil - pile slip</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Soil yielding under raft</td>
<td>300</td>
<td>-</td>
</tr>
<tr>
<td>Soil yielding at pile base</td>
<td>450</td>
<td>-</td>
</tr>
<tr>
<td>Tensile stress under raft</td>
<td>-</td>
<td>20</td>
</tr>
</tbody>
</table>
7.2.2 Piled Raft Foundation Systems

The three piled raft arrangements analysed are indicated in Figures 7.1, 7.14 and 7.26. These cases are presented in the following sections.

7.2.2.1 \(2^2\) Pile Group Analysis

In this section, a piled raft foundation system with a \(2^2\) pile group is analysed, including the effect of the ground movement caused by the swelling soils. The piled raft arrangement is shown in Figure (7.1). An initial brief analysis is carried out to indicate the influence of the soil surface movement on various raft stiffnesses \((K_R)\). The analysis is undertaken for a wide range of ground movement and its influence on the overall foundation performance is presented and discussed. Further detailed analyses are again presented for \(K_R = 1 \& 0.01\).

7.2.2.1.1 Load Sharing Characteristics of Piles

It can be seen from Figure 7.4 that the load on the piles reduces with the increase in the ground movement. In the case of a rigid raft \((K_R=10)\) on a flat ground surface \((S_o = 0)\), the piles carry higher load (Figure 7.4) compared to the flexible raft \((K_R=0.001)\). It can be seen from Figure (7.4) that the high compressive stress at the pile raft interface reduces the potential development of tensile stress, whereas a less-heavily loaded pile increases the possible occurrence of a high tensile load at the pile head. Therefore, in the case of a flexible raft, the pile experiences higher tensile load with increasing ground movement, because the low compressive stress on a flat ground surface \((S_o = 0)\) is unable to counteract the upward force induced by swelling soils. The change in the state of stress at the interface takes place due to the combination of the following two aspects:

- The upward force induced by the ground movement along the pile shaft, which reduces compressive stress
- The swelling soil exerts an upward thrust underneath the raft, causing a reduction in the compressive load on the pile. The piles located towards the edge of the raft experience a greater magnitude of ground movement compared to the inner piles, because the soil towards the outer periphery of raft experiences higher exposure to moisture and temperature fluctuations. These features are discussed in the sections dealing with \(4^2\) and \(6^2\) pile raft groups, where piles are located towards the inside and outside of the raft.

It can be seen from Figure (7.4) that, under the application of \(P = 0.25 P_{ult}\) for a flat ground condition, the loads on the pile heads are 42% and 22% of the ultimate capacity for the
rigid \((K_R = 1)\) and flexible \((K_R = 0.01)\) raft conditions, respectively. Due to 80 mm ground movement, the reductions in the pile loads are 76% and 124% with respect to the flat ground condition, respectively. In other words, the piles which carry higher load in the flat ground condition, experience lesser reduction. Conversely, the piles experiencing a lower load, are subject to higher reduction in load and also experience a tensile load due to the upward thrust exerted by the swelling soils. The distribution of load along the pile shaft with the ground movement is discussed in the following section in detail.

The stress distribution underneath a rigid raft \((K_R = 1)\) are shown in plates (7.1) and (7.2) for the \(0.25\ P_{ult}\) load condition. These contours represent a flat \((S_o = 0)\) ground condition as well as the ground surface subjected to 80 mm \((=S_o)\) movement due to swelling soils. It can be seen from these plates that a decrease in the pile load and an increase in the upward thrust along the edge of the raft take place due to the movement of the ground.

7.2.2.1.2 

**Settlement Analysis**

In order to study the settlement behaviour of the foundation, the settlements at the pile head and at the centre of the raft are analysed and presented. In the case of a rigid raft \((K_R = 1)\), the settlements at the pile head and at the centre of the raft experience a reduction with increasing ground movement (Fig 7.6). As discussed previously, the upward ground movement causes a thrust underneath the raft as well as inducing an upward force along the pile shaft, and this causes a reduction in settlement towards the edge of the raft. In the case of a rigid raft \((K_R = 1)\), due to its rigidity, the edge effect is transferred throughout the raft and the whole raft experiences similar settlement. As a result of this, the differential settlement is less in comparison to a flexible raft \((K_R = 0.01)\). In other words, the rigid movement of the raft causes higher redistribution of stress towards the central portion of the raft. For a ground movement of 80 mm, the reduction in settlement for a rigid loaded raft \((P = 0.25\ P_{ult})\) is approximately 27% (Figure 7.6) with respect to the flat ground surface. In the case of an unloaded raft subjected to a ground movement variation of 40 to 80 mm, the increase in the upward displacement is approximately 66%, (Figure 7.5). As a result of this, a rapid reduction in the contact stress takes place towards the centre of the raft (Figure 7.9). In this process, the contact stress towards the edge of the raft increases and reaches its limiting value (300 kPa). In other words, due to the uniform movement of the rigid raft \((K_R = 1)\), the contact stress reduces towards the inner portion of the raft and increases at the raft edge.

It can be seen from Figures (7.5 and 7.6) that the settlement at the centre of the flexible raft increases up to a point with the ground movement, and then reduces. The high contact stress towards the edge of the raft causes a downward deformation of the soil profile at the
centre of the raft due to the interaction which pulls the raft down due to the adhesion between the raft and the soil. With further increase in the ground movement, the tensile stress between the raft and the soil exceeds the adhesion limit (20 kPa), and as a result, "lift-off" takes place. This phenomenon causes an upward displacement in the case of an unloaded flexible raft.

The above behaviour is not evident for the rigid raft, as the adhesive force between the raft and the soil surface is not able to pull the raft down, due to its rigidity.

The displacement contours for a rigid raft ($K_R=1$) are shown in plates (7.5) and (7.6), representing a flat ($S_o = 0$) ground condition as well as the ground surface subjected to 80 mm (= $S_o$) movement. The decrease in the overall settlement is evident due to the upward movement of the ground, as discussed earlier.

It should be noted that, in a ground movement situation, the settlement of the foundation system is dependent on the number of piles and their respective locations. The presence of piles towards the centre of the raft influences the settlement profile quite significantly, as it restricts the uplifting effect. In later sections, it is indicated that the foundation settlement profile is significantly dependent on these parameters.

### 7.2.2.1.3 Load Distribution along Pile Shaft

The upward movement of the ground due to the swelling effect causes an upward force along the pile shaft and also exerts a thrust underneath the raft. Although the pile behaviour in a piled raft system is quite similar to that of a single pile discussed in chapter 5, due to its structural connection to the raft, the behaviour is significantly influenced by the raft and its stiffness.

It can be seen from Figure 7.7 that for an unloaded raft ($P = 0$), the pile experiences a tensile stress at the pile - raft interface due to the upward induced force along the pile shaft and the thrust underneath the raft. An increase in the ground movement causes further increase in the tensile stress. These tensile stresses are counteracted by the resistance offered by the pile shafts in the stable zone. In other words, the shear stress along pile shaft in the stable zone becomes negative (downward), in order to balance the tension at the pile head.

In the case of an unloaded foundation subjected to a ground movement increase from 40 to 120 mm, the increase in the tensile stress is 86% and 80% (Figure 7.7a) for rigid ($K_R=1$) and flexible raft ($K_R=0.01$) conditions, respectively. In the case of a rigid raft, the tensile
stress is higher due to its rigidity, which causes higher transfer of swelling force towards the pile. In the case of a flexible raft, due to its flexibility, the swelling soil causes less thrust (contact stress) towards the raft edge, compared to the rigid raft. Therefore, it can be seen from Figure (7.8) that the contact stress underneath the edge of the rigid raft is higher in comparison to the flexible raft. It should be noted that the presence of piles towards the centre of the raft will influence the performance quite significantly.

In the case of a loaded raft, the load sharing characteristics between the pile and the raft are dependent on the raft stiffness. A rigid raft exerts higher load on the pile in comparison to a flexible raft. As discussed earlier, the swelling soil causes a reduction in the compressive stress at the pile - raft interface. A less loaded pile may be subject to a tensile stress due to this effect. It can be seen from Figure (7.7b) that, in the case of a flexible raft (\(K_R = 0.01\)), the load on the pile under a flat ground condition is less than the rigid raft condition (\(K_R = 1\)). Therefore, for flexible rafts, the swelling force causes a reduction in compressive stress at the pile head up to \(S_o = 40\) mm, but a further increase in the ground movement causes a development of tensile stress at the pile raft interface. In the case of rigid raft, the pile experiences a higher load in a flat ground condition, and therefore the tensile stress at \(S_o = 120\) mm is much less compared to the flexible raft. The reduction in stress in a rigid and flexible raft condition, for a ground movement of \(120\) mm, is approximately \(101\%\) and \(151\%\) with respect to the flat ground condition. A percentage, higher than \(100\%\), indicates that the compressive stress is totally nullified and the pile head is in tension.

### 7.2.2.1.4 Contact Stress Distribution

The contact pressure under the raft is significantly influenced by the effect of redistribution, which is dependent on the raft stiffness. The analysis is undertaken for \(P = 0\) (unloaded raft) and \(0.25 P_{ult}\) (loaded raft) loading conditions and the results are presented in Figures 7.8 and 7.9. Due to the upward ground movement towards the edge of the raft, a rapid increase in the contact stress takes place along the edge and the effect is transferred towards the central portion of the raft, depending on the raft stiffness.

In case of an unloaded rigid raft (\(K_R = 1\)), the increase in the contact stress is rapid, compared to the flexible raft. As a result of this, the central portion of the rigid raft experiences lift-off. For a flexible raft (\(K_R = 0.01\)), the effect of swelling soil towards the central part of the raft is less. It can be seen from Figure (7.8) that, for a flexible raft, the initial ground movement (\(S_o = 40\) mm) causes the central raft to adhere to the soil, in other words, the bottom of the raft experiences a tensile stress between the raft bottom and the soil. With further upward movement of the ground, the tensile contact stress (tensile) exceeds the limiting value, causing a detachment (lift off) between the raft and the soil. In
the case of a rigid raft, the detachment takes place well before the flexible raft, because of the greater rigidity.

In the case of a loaded raft, the contact stress distribution is dependent on the raft stiffness. It can be seen from Figure (7.9) that, in comparison to a flexible raft, a rigid raft experiences high contact stress towards the edge and less stress in the central portion of the raft, in a flat ground condition. Due to the low contact stress towards the centre of the raft and the rigidity of the raft, the compressive stress decreases very rapidly and exceeds the limiting adhesion value, resulting in lift off. In case of a flexible raft, the contact stress towards the central raft is high (Figure 7.9). Therefore, no lift off takes place with the increase in the ground movement, because of the initial high contact stress in a flat ground condition as well as due to the flexibility of the raft, which is unable to transfer the edge effect towards the central portion of the raft.

Due to the ground movement, the contact stress along the edge of the raft increases very rapidly and exceeds the limiting bearing stress. Further increase in the ground movement causes a transfer of stress towards the central part of the raft.

7.2.2.1.5 Moment Analysis

The moment, the maximum moment and the moment along a section BB are presented and discussed for a wide range of ground movements in Figures (7.10) to (7.13). In swelling soils, it can be seen from Figure (7.11) that, for a loaded raft, the negative moment at the proximity of the pile location reduces with an increase in the ground movement. The upward thrust under the edge of the raft causes a reduction in the compressive stress on the pile head (Figure 7.7b) and as a result, the negative moment reduces and eventually becomes positive. The positive moment at the centre of the BB axis is also influenced by the edge ground movement. In the process of swelling of soil, the following phenomena take place:

- Higher ground movement between the edge and the pile tends to induce positive moment towards the central portion of the axis BB.
- The upward movement of the ground between piles causes a reduction in the positive moment.

The influence of the above features influence the moment behaviour of the raft along BB. It can be seen from Figure (7.11) that, for a rigid raft, the positive moment at the centre of section BB increases with the ground movement of 40 mm and then reduces with the further increase in the ground movement. The initial ground movement towards the end of
BB will be very high, which will reduce the imposed load on the pile and as well it will create positive moment towards the centre of BB. As section BB is some distance away from the edge of the raft, the influence of ground movement towards the centre of BB is less. Therefore, in the case of a rigid raft, the initial movement of the ground causes an increase in the positive moment, because the effect of the end ground movement is transferred towards the centre of BB. Further increase in the ground movement causes a thrust under the raft towards the centre of BB. Due to this, the positive moment reduces with the increase in the ground movement. In the case of a flexible raft, the initial effect of the ground movement is not very significant, because the edge effect is not transferred towards the centre of the BB axis due to the raft flexibility. For the rigid raft, the increase in the positive moment at the centre of BB for a ground movement of 40 mm is 3% and with the further increase in the ground movement from 40 mm to 120 mm, the decrease in the positive moment is 45%. In the case of the flexible raft, the positive moment experiences a continuous reduction, reaching 24% for a ground movement of 120 mm.

In the case of an unloaded raft (Figure 7.10), the movement of the ground along the edge of the raft causes an upward thrust underneath the raft. As a result of this, positive moment develops at the pile head and negative moment occurs towards the centre of the section BB with the increasing ground movement. The positive moment at the pile head is caused by the upward thrust between the pile and the edge of the raft, whereas the negative moment at the centre is due to the upward ground movement along the edge. This phenomenon induces a tensile stress at the pile-raft interface. The increase in moment is higher in the case of a rigid raft than for a flexible raft.

The maximum positive moment in a rigid loaded raft \( (P = 0.25 P_{\text{ult}}) \) reduces with the increasing ground movement (Figure 7.13). The upward thrust along the edge of the raft is redistributed to the whole raft due to its rigidity and due to this effect, the reduction in the maximum positive moment takes place with the increase in the ground movement. The maximum negative moment also experiences a reduction due to the ground movement. In a flat ground condition \( (S_o = 0) \), the negative moment occurs at the pile head, which reduces with the increasing upward thrust caused by the swelling soils. In the case of a flexible loaded raft \( (K_r = 0.01) \), the maximum positive moment increases with the increasing ground movement, because the movement along the edge of the raft has much less influence at the centre of the raft. The reduction in the maximum negative moment is 92% and 94% for flexible and rigid raft conditions (Figure 7.13) respectively, for a ground movement of 80 mm. For a similar ground movement, the maximum positive moment reduces by 20% for the rigid raft and increases by 34% for the flexible raft.
In the case of an unloaded raft, the maximum negative moment increases with increasing ground movement. The increase is approximately 7 times and 10 times for rigid and flexible raft conditions respectively, for a ground movement from of 40 to 120mm (Figure 7.12). The rigid raft experiences higher increase in the negative moment due to its rigidity in comparison to a flexible raft. The maximum positive moment also increases with the increasing ground movement due to the upward thrust in the proximity of the pile head.

The contours of moment along X-axis for a rigid raft ($K_R=1$) are shown in plates (7.3) and (7.4), representing a flat ($S_o = 0$) ground condition as well as the ground surface subjected to 80 mm ($S_o$) movement. It can be seen from these plates that at the pile locations the negative moment reduces and becomes positive with the ground movement as shown in Figure (7.11).

7.2.2.2 $4^2$ PILE GROUP ANALYSIS

In this section, a $4^2$ pile group is analysed, including the effect of the ground movement, caused by the process of swelling of soils. The piled raft arrangement is shown in Figure (7.14) and the limiting values to incorporate the non linear effects are indicated in Table 7.1.

An initial analysis is carried out to indicate the influence of the soil surface movement on various raft stiffnesses ($K_R$). The analysis is undertaken for a wide range of ground movement and the influence on the loads shared by various piles is presented in section 7.2.2.2.1. Further detailed analyses are presented again for $K_R = 1 & 0.01$, which generally represent the “rigid” and “flexible” raft conditions.

7.2.2.2.1 LOAD SHARING CHARACTERISTICS OF PILES

It can be seen from Figure 7.15 that for a rigid raft ($K_R = 10$), under a flat ground condition ($S_o = 0$), the maximum and the minimum loads are shared by the corner (pile 3) and central (pile 1) piles, respectively. The difference in load distribution on various piles reduces as the raft becomes flexible. The load on all piles experiences a reduction due to the swelling effect, excepting the inner piles, which are subject to a minor increase in load in a flexible raft situation.

The movement of the ground underneath the raft causes an upward thrust towards the edge of the raft and also induces an upward force along the pile shaft. The raft experiences maximum thrust along its edge, because of the direct exposure to the seasonal moisture
variation. The movement along the pile shaft is dependent on the location of the piles underneath the raft. A pile (such as pile 3) towards the edge of the raft will be subject to higher ground movement in comparison to a pile (such as pile 1) towards the centre of the raft. These features influence the pattern of load distribution between the piles and the raft. Due to the structural connection between the raft and the pile, the raft stiffness affects the distribution of load very significantly. It can be seen from Figure (7.15) that, for a rigid raft ($K_R = 10$), all piles experience a reduction in load. The reduction in load at the outer pile (pile 3) is due to the upward thrust under the raft caused by the swelling soils. The reduction in load in the inner pile (pile 1) takes place due to the rigidity of the raft, which causes the raft to move uniformly and redistributes the upward effect all over. The percentage reduction of load in piles for various raft stiffness subjected to a wide range of ground movement is presented in Table 7.2. A percentage higher than 100% indicates the pile head experiences tensile stress, whereas, less than zero (negative) indicates an increase in the pile load. It should also be noted that these percentages are relative values, for example, for $K_R = 10$, the reduction of load in pile 1 is 83% means that the $\{(\text{load on pile 1 at } S_o = 0) - (\text{load on pile 1 at } S_o = 40 \text{ mm}) / \text{load on pile 1 at } S_o = 0\}$. Therefore, these figures should not be used to compare the magnitude reduction in between two piles.

**Table : 7.2**

(Percentage reduction in pile load)

<table>
<thead>
<tr>
<th>$S_o$</th>
<th>40 mm</th>
<th>80 mm</th>
<th>120 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pile : 1</td>
<td>Pile : 2</td>
<td>Pile : 3</td>
</tr>
<tr>
<td>$K_R$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>83</td>
<td>64</td>
<td>52</td>
</tr>
<tr>
<td>1</td>
<td>62</td>
<td>63</td>
<td>56</td>
</tr>
<tr>
<td>0.1</td>
<td>2</td>
<td>58</td>
<td>89</td>
</tr>
<tr>
<td>0.01</td>
<td>-15</td>
<td>50</td>
<td>132</td>
</tr>
</tbody>
</table>

In a rigid raft condition, the reduction in load in the corner pile (pile 3) is higher in comparison to the central pile (pile 1). The corner pile experiences the maximum load on a flat ground condition ($S_o = 0$) compared to the central pile, therefore the upward thrust causes higher reduction of load. The load distribution pattern in a flexible raft is different, because of the lack of distribution of the swelling effect towards the central pile. It can be seen from Figure (7.15) that the influence of swelling soils on the central pile (pile 1) is significantly less, whereas the corner pile (pile 3) is influenced very significantly and is subjected to tensile stress at the raft pile interface.

It is important to note from Table 7.2 that the corner pile (pile 3) experiences higher tensile stress in the case of a flexible raft compared to a rigid raft. In a flexible raft condition, the influence of the swelling soil is not transferred towards the central portion of the raft, since
the effect is localised and the swelling force is transferred to the pile in the closest proximity. The swelling force is a maximum at the corner, and therefore the corner pile (pile 3) is most affected. The reduction in the corner pile (pile 3) load is 249% (Table 7.2), i.e. the pile experiences very high tension at the pile raft interface. The influence is localised and the central pile (pile 1) remains nearly unaffected.

Due to the restriction created by the raft and the pile, the free field soil surface profile is distorted and high contact stress is developed along the edge of the raft. The free field soil surface profile at the centre of the raft experiences a “depression” due to the change in the stress condition at the “raft-pile-soil” interface. Due to this depression, an adhesion force develops between the raft and the soil, which induces force on the central pile (pile 1). Therefore, it can be seen from Table 7.2 that, in the case of a flexible raft, the central pile (pile 1) experiences little increase in load.

The stress distribution between the raft and the pile for a rigid raft ($K_R=1$) are shown in plates (7.7) and (7.8), representing a flat ($S_o = 0$) ground condition as well as the ground surface subjected to 80 mm ($=S_o$) movement.

It should be noted that these are the loads at the pile heads; the Distribution of load along the pile shaft is discussed in later chapters.

7.2.2.2 Settlement Analysis

In order to analyse the settlement behaviour of the foundation, the analysis is again undertaken for $P = 0$ (unloaded raft) and 0.25 $P_{ult}$ (loaded raft) load conditions. The settlement at the raft centre as well as at various pile heads are analysed for a wide range of ground movements. It can be seen from Figure (7.16) that for an unloaded raft ($P = 0$), the ground movement causes higher upward displacement at the corner pile (pile 3) and lesser displacement at the central pile (pile 1). The difference in displacement in between pile 1 and 3 is less in the case of a rigid raft compared to a flexible raft. The increase in the upward displacement at the corner pile (pile 3), due to the ground movement variation from 40mm to 80mm, is 101% and 200%, respectively for the rigid and flexible rafts. For the same ground movement, the increase in upward displacement at pile 1 is 101% and 38% for rigid and flexible raft conditions, respectively. In the case of a rigid raft, due to the uniform movement of the raft, the thrust along the edge of the raft is redistributed to the central piles. As a result of this, the central pile (pile 1) is significantly influenced by the swelling effect. Therefore, due to the redistributed sharing effect by the other piles, the displacement of pile 3 is reduced. In the case of a flexible raft, due to the lack of redistribution, the effect is localised. The upward thrust along the edge of the raft is mostly transferred to the corner
pile (pile 3) which experiences very high upward displacement in comparison to a rigid raft condition. The inner pile (pile 1) remains less affected, because of the smaller transfer of the swelling effect towards the central portion of the raft. It can also be seen from Figure (7.22) that the moment in the case of a flexible raft is significantly less influenced compared to the rigid raft. As discussed earlier, the centre of the flexible raft experiences downward settlement due to the downdrag force created by the adhesion in between the raft bottom and the soil, as a result of this, the centre of the raft experiences minor downward settlement. In the case of a rigid raft, due to the raft rigidity, this downdrag force has no influence on the central portion of the raft.

In the case of a loaded raft ($P = 0.25 P_{ult}$), for the rigid raft condition, the settlement of the foundation reduces with increasing ground movement. For a flexible raft, the corner pile (pile 3) experiences a higher reduction in settlement, but the central pile (pile 1) remains almost unaffected. For a ground movement of 80mm, the reduction in the corner pile settlements (pile 3) are 21% and 30% for rigid and flexible raft, respectively, whereas, for the similar ground movement, the inner pile (pile 1) experiences only 19% and 2% reduction in settlement for the rigid and flexible raft conditions.

The pattern of displacement for a rigid raft ($K_{R} = I$) is shown in plates (7.11) and (7.12). These plates indicate the overall reduction in the displacement due to upward movement of the ground. The swelling soil causes a reduction in the pile settlement, depending on the pile location and the raft stiffness.

### 7.2.2.2.3 LOAD DISTRIBUTION ALONG PILE SHAFT

In the case of an unloaded rigid raft ($P = 0$), due to the upward thrust, the piles experience tensile stress at the pile raft interface. The magnitude of tensile stress is dependent on the pile location. The maximum and the minimum tensile stresses are developed in the corner (pile 3) and the central pile (pile 1), respectively. The ground movement induces an upward force along the pile shaft as well as it causes a thrust underneath the raft. Due to this effect, the pile shaft in the swelling zone experiences an upward stress along the pile shaft and the pile shaft in the stable zone causes a downward stress to counteract the tensile force. In the case of a flexible raft (Figure 7.18), the outer piles (piles 2 and 3) experience tensile force, whereas, the inner pile (pile 1) experiences a minor compressive force. As discussed earlier, the compressive force is exerted towards the central pile (pile 1) due to the vertical deformation of the free field soil surface under the central raft, which drags the raft down due to the adhesion between the raft and the soil. This causes a development of compressive force in the inner pile. This feature is significant in the flexible raft as it deforms and is influenced by the adhesion. In contrast, a rigid raft is not influenced. It can
be seen from Figure (7.20 and 7.21) that the flexible raft experiences a negative contact stress, whereas, for a rigid raft, the contact stress exceeds the limiting tensile stress and is subject to detachment (lifting off). Therefore, all piles in a rigid raft condition experience tensile stress (Figure 7.18).

In the case of a rigid loaded raft ($P = 0.25 \, P_{ult}$), under a flat ground condition ($S_o = 0$), the maximum and the minimum loads are taken by the outer and inner piles, respectively. Due to the raft rigidity, the edge swelling force is transferred towards the central piles. As the central pile is less loaded in a flat ground condition, the swelling force nullifies the compressive stress at the pile raft interface and generates tensile stress. The outer piles are subject to tensile stress at a later stage of the ground movement, because the higher imposed load on the corner pile counteracts the swelling effect. Therefore, the corner pile experiences tensile stress at $S_o = 120\, mm$, whereas, the inner pile is subject to tensile stress at $S_o = 80\, mm$. In the case of a flexible raft, the central pile is less influenced, and the corner pile (pile 3) is most affected by the ground movement along the edge of the raft.

**7.2.2.2.4 CONTACT STRESS DISTRIBUTION**

Piled raft systems in swelling soils experience very high contact stress along the edges of the raft. The increase in the contact stress is due to the relative upward movement of soil surface, caused by the swelling soil. The presence of piles towards the raft edge restricts the upward movement of the raft and as a result, high contact stress builds up at the corner as well as along the edge of the raft. It can be seen from Figure (7.20) that the contact stress towards the edge of the raft increases with the increasing ground movement. The contact stress can exceed the ultimate bearing capacity ($300 \, kPa$, Table 7.1), causing yielding of soil locally.

It can be seen from Figure (7.20) that, in the case of an unloaded raft, the increase in the contact stress along the edge of the raft is quite rapid for a rigid raft, compared to a flexible raft. Therefore, in a rigid raft, the contact stress exceeds the ultimate capacity limitation and causes redistribution towards the central part of the raft. For a rigid raft, the central portion experiences a tensile stress only up to $S_o = 40\, mm$, after that, a detachment takes place between the raft and the soil. For a flexible raft ($K_R = 0.01$), the centre of the raft remains in contact of the soil, because the tensile stress between the raft and the soil still remains within the limit of $20 \, kPa$.

It can be seen from Figure (7.21) that, in the case of a loaded flexible raft, under a flat ground condition, the contact stress in higher towards the centre of the raft in comparison to the rigid raft. The contact stress at the centre reduces with increasing ground movement. As
the contact stress at the centre is less in case of a rigid raft, the increasing ground movement causes tension between the raft and the soil. In contrast, in the case of a flexible raft, due to the raft flexibility, less swelling effect is transferred towards the centre as well as due to the higher initial stress (in the flat ground condition), part of the swelling effect is counteracted. As a result, the central part of the raft is less influenced by the swelling soils. The stress at the edge of the rigid raft reaches the limiting value at $S_o = 80mm$, further increase in the ground movement causes the redistribution effect and the contact stress increases towards the centre of the raft.

7.2.2.2.5 **MOMENT ANALYSIS**

The maximum moment and the moment along section BB are analysed for a wide range ground movement. The upward movement of the ground causes a thrust underneath the raft towards the edge as well as it induces upward stress along the pile shaft. Outer piles experience higher ground movement in comparison to the inner piles. As a result of this, the outer periphery of the raft subjects to upward contact stress. The influence of the ground movement on the inner pile is less than on the outer pile, therefore, these piles experience less upward force due to the ground movement, compared to the outer piles. The influence of the ground movement is dependent on the imposed loading on the raft.

It can be seen from Figure (7.22) that, in the case of an unloaded raft, the positive moment at the pile heads increases with the increasing ground movement. The increase in moment in a rigid raft ($K_R = 1$) is higher in comparison to a flexible raft ($K_R = 0.01$). The maximum positive moment also increases with increasing ground movement (Figure 7.24). The maximum positive moment occurs at the pile head locations.

In the case of a loaded raft ($P = 0.25 P_{ult}$), the negative moment along section BB reduces and the positive moment increases with increasing ground movement (Figure 7.23). A reduction in the negative moment at the corner pile head takes place due to the upward pressure caused by the swelling soil. It is very important to note that the high negative moment changes to the positive moment in the raft slab at the pile head locations. This may be a serious point of concern in designing the raft slab. The maximum positive moment also increases with increasing ground movement, whereas the maximum negative moment decreases. For 80 mm ground movement, the increase in the maximum positive moment is approximately 142% and 211% for rigid and flexible raft conditions. For the same ground movement, the reduction in negative moment is 93% and 91% for rigid and flexible rafts, respectively.
The distributions of moment for a rigid raft ($K_R=1$) are shown in plates (7.9) and (7.10). These plates indicate the reduction in the negative moment at the corner pile locations as presented in Figure (7.23).

### 7.2.2.3 6² PILE GROUP ANALYSIS

In this section, the 6² pile group shown in Figure (7.26) is analysed, including the effect of the ground movement caused by the swelling soils.

An initial brief analysis is carried out to indicate the influence of the soil surface movement on various raft stiffnesses ($K_R$). The analysis is undertaken for a ground movement of 80mm and the influence on the loads shared by various piles is presented in section 7.2.2.3.1.

#### 7.2.2.3.1 LOAD SHARING CHARACTERISTICS OF PILES

It can be seen from Figure 7.27 that for a rigid raft ($K_R = 10$), under a flat ground condition ($S_o = 0$), the maximum and the minimum loads are shared by pile 6 and pile 1, respectively. The difference in load between piles reduces as the raft becomes more flexible.

As discussed earlier, the ground movement profile underneath the raft causes an upward thrust towards the edge of the raft and also induces an upward force along the pile shaft. The raft experiences maximum thrust along the edge of the raft, because of the direct exposure to the seasonal moisture variation. The ground movement along the pile shaft is dependent on its respective location underneath the raft. Pile 6 towards the edge of the raft experiences higher ground movement in comparison to a pile (pile 1) towards the centre of the raft, and this influences the load distribution between the piles, as well as in the raft. It can be seen from Figure (7.27) that, in the case of a rigid raft ($K_R = 10$), all piles experience a reduction in load. The outer pile (pile 6) experiences the reduction due to the upward thrust under the raft, caused by the swelling soils. The reduction in load in the inner pile (pile 1) takes place due to the rigidity of the raft, which redistributes the swelling effect towards it. The percentage reduction in load in piles is indicated in Table (7.3) for ground movements of 40 and 80 mm.
In a rigid raft, the reduction of load in the corner pile (pile 6) is higher in comparison to the central pile (pile 1). The load distribution in a flexible raft condition is different and it can be seen from Figure (7.27) and Table (7.3) that the influence of swelling soils on the central pile (pile 1) is significantly less.

The corner pile (pile 6) experiences higher tensile stress in case of a flexible raft compared to a rigid raft. In a flexible raft condition, the influence of the swelling soil is not transferred towards the central portion of the raft, because of the raft flexibility. As a result of that the corner piles (piles 3, 5 and 6) are most affected. The influence becomes localised and the central pile (pile 1) remains unaffected, as discussed earlier.

The stress distributions between the raft and the piles for a rigid raft ($K_R=1$) are shown in plates (7.13) and (7.14), representing a flat ($S_o=0$) ground condition as well as an 80 mm ($=S_o$) movement. It can be seen that the piles 2 and 4 are subjected to a tensile stress at the pile-raft interface due to the ground movement, which is also indicated in Figure (7.27).

### 7.2.2.3.2 SettlemenT Analysis

In order to analyse the settlement behaviour of the foundation, the analysis is again undertaken for $P=0$ (unloaded raft) and 0.25 $P_{ult}$ (loaded raft) load conditions. It can be seen from Figure (7.28) that for an unloaded raft ($P=0$), the ground movement causes higher upward displacement at pile 6 and a lesser displacement at pile 1. The corner and edge piles (piles 3, 5 and 6) experience higher upward displacement due to the upward thrust caused by the swelling soils along the edge of the raft. The increase in the upward displacements at pile 6, due to the movement of the ground from 40mm to 80mm, are 125% and 183%, respectively for the rigid and flexible rafts (Figure 7.28). For the same ground movement, the increase in upward displacement at pile head 1 is 117% and 32%, for rigid and flexible rafts. In a flexible raft, the corner pile (pile 6) experiences very high upward displacement in comparison to a rigid raft, as discussed in previous sections. In the case of a rigid raft, the thrust along the edge of the raft is redistributed to other foundation...
components, therefore, due to the resistance offered by the other piles, the displacement of pile 6 is reduced. In the case of a flexible raft condition, due to the lack of redistribution, the effect is localised. The upward thrust along the edge of the raft is mostly transferred to the corner pile (pile 6). The inner pile (pile 1) is nearly not effected, because of the smaller influence of the ground movement towards the central portion of the raft.

In the case of a rigid loaded raft \((P = 0.25 \, P_{ult})\), the settlement of the whole foundation reduces with increasing ground movement. For a flexible raft, the corner piles (pile 5 & 6) experience higher reduction in settlement and the central pile (pile 1) remains nearly unaffected. For a ground movement of 80 mm, the reduction in the corner pile settlements (pile 6) is 16% and 23% for rigid and flexible raft, respectively. For similar ground movement, the inner pile (pile 1) experiences a reduction in settlement by 14% and 3%, for the rigid and flexible raft conditions. It can also be seen from Figures (7.28 & 7.29) that the differential settlement also increases with an increase in the ground movement.

The reduction in the overall displacement due to the movement of the ground by 80 mm for a rigid raft \((K_R=1)\) is shown in plates (7.17) and (7.18).

### 7.2.2.3.3 Load Distribution along Pile Shaft

It can be seen from Figure (7.30) that the basic mechanism, as discussed in section 7.2.2.2.3 for \(d^2\) pile group, holds good for the unloaded raft, only the magnitude changes due to the greater number of piles and their respective locations. The magnitude of tensile stress is dependent on the pile location and the raft stiffness. In the case of a rigid raft \((K_R = 1)\), the maximum and the minimum tensile stresses develop at the corner (pile 6) and the central pile (pile 1) heads, respectively. In the case of a flexible raft, all piles except pile 1 (central pile) experience tensile stress. The central pile is subjected to minor compressive force for the reason discussed in section 7.2.2.2.1.

In the case of a rigid loaded raft \((P=0.25 \, P_{ult})\), the swelling force counteracts imposed loads on the piles and causes a reduction in load (Figure 7.31a). For a flexible raft, due to the lack of redistribution effect, the corner piles (3, 5 and 6) experience maximum reduction due to the localised nature of the effect.

### 7.2.2.3.4 Contact Stress Distribution

The influence of the ground movement on the contact stress distribution is similar to that discussed in section 7.2.2.2.4. The greater number of piles effect the foundation behaviour quite significantly, but the basic mechanism remains the same.
It can be seen from Figure (7.32) that, in the case of an unloaded raft, the increase in the contact stress along the edge of the raft is quite rapid for a rigid raft, compared to a flexible raft. The contact stress exceeds the ultimate capacity limitation at the corner of the raft (Plate 7.14) and causes redistribution to other foundation components.

It can be seen from Figure (7.33) that, in the case of a flexible loaded raft, under a flat ground condition, the contact stress in higher towards the central part of the raft in comparison to the rigid raft. The contact stress at the centre of the raft reduces with increasing ground movement. As the contact stress at the centre is less in case of a rigid raft, the increasing ground movement causes a rapid decrease in the contact stress. In the case of a flexible raft, due to the raft flexibility, less swelling effect is transferred towards the centre as well as due to the higher initial contact stress, and part of the swelling effect is counteracted. The contact stress at the corner of the rigid raft reaches the limiting value (300 kPa) at \( S_0 = 120 \text{mm} \), and further increase in the ground movement causes some redistribution and the contact stress increases towards the centre of the raft.

The presence of piles near the central part of the raft influences the contact stress quite significantly. It can be seen from Figure (7.9) that, in case of \( 2^2 \) pile group, the contact stress is higher in comparison to \( 4^2 \) or \( 6^2 \) pile groups (Figures 7.21 and 7.33), because the central piles cause lesser transfer of load to the central soil surface. The behaviour of a \( 2^2 \) pile group is significantly different to \( 4^2 \) and \( 6^2 \) pile groups, due to the presence of piles towards the centre.

**7.2.2.3.5 Moment Analysis**

It can be seen from Figure (7.34) that, in the case of an unloaded raft, the positive moment increases along section BB with increasing ground movement. The increase in moment in a rigid raft \( (K_R = I) \) is higher than in a flexible raft \( (K_R = 0.01) \). The maximum positive moment also increase with the increasing ground movement. For a ground movement of 80 mm, the increase in the positive moment is approximately 124% and 25% for rigid and flexible raft conditions, with respect to the flat ground condition.

In the case of a loaded raft \( (P = 0.25 P_{w0}) \), the negative moment along BB reduces and the positive moment increases with increasing ground movement. The maximum positive moment also increases with the increasing ground movement, whereas the maximum negative moment reduces. For a ground movement of 80 mm, the increase in the maximum positive moment is approximately 74% and 37% for rigid and flexible raft conditions. For the same ground movement, the reduction in negative moment is 58% and 62% for rigid and flexible rafts, respectively.
The moment contours for a rigid raft \((K_R=1)\) subjected to 80 mm ground movement are shown in plates (7.15) and (7.16). It can be seen that the negative moment reduces and becomes positive at the corner pile location as discussed in Figure (7.35).

7.2.3 **Free Standing Pile Groups and Pile Raft Systems - A Comparative Analysis**

In this section, the behaviour of a piled raft foundation is compared with that of a free standing pile group. The behaviour of free standing pile groups is expected to be different in a reactive soil environment, because of the gap between the raft and the soil. Due to the gap, the upward ground movement influences the pile shaft only, without any thrust underneath the raft. Therefore, the load distribution pattern shall be dependent on the pile behaviour, although the raft stiffness plays an important role in redistributing the effect to other foundation components. In the case of piled raft foundation systems, the ground movement causes an upward thrust underneath the raft as well as inducing forces along the pile shaft.

It should also be noted that the ultimate compressive capacity of a free standing pile group is generally less in comparison to a piled raft foundation system, because the capacity offered by the raft is absent.

The free standing pile group is analysed for the case of "uniform ground movement" as well as for the "edge heave" condition. It is assumed in the analysis that the soil profile under the raft never comes in contact with the bottom of the raft. The uniform soil surface movement is adopted to represent a well ventilated gap, where the change in moisture occurs uniformly underneath the raft. Conversely, if the gap is not ventilated, an edge heave ground movement is considered to represent the situation.

The purpose of the analysis is to indicate the advantages and disadvantages of maintaining a gap between the raft and the soil surface. The analysis is undertaken for the same loading \((56 \text{ kPa})\) for the piled raft and free standing pile groups, which is approximately 25% of ultimate capacity \((P = 0.25 P_{ub})\) of the free standing pile group.

The analysis is carried out with the arrangements indicated in Figure (7.38).

7.2.3.1 **Settlement Analysis**

In order to analyse the settlement behaviour, the analysis is undertaken for the rigid \((K_R=1)\) and the flexible \((K_R=0.01)\) raft conditions. It can be seen from Figure (7.39) that for a rigid
raft \((K_R=1)\) on a flat ground condition \((S_o=0)\), the settlement of the free standing pile group is approximately 5% higher than the piled raft system. The settlement of the foundation reduces with an increase in the upward ground movement. In the case of a piled raft system, the reduction in settlement is large in comparison to the free standing pile groups, because the piled raft system experiences an upward thrust underneath the raft, causing a higher reduction in settlement. The free standing pile groups are affected by the ground movement only due to the induced force along the pile shaft.

In case of the free standing pile groups subjected to uniform ground movement \((S_o = 80\ mm)\), the decrease in the settlements for the central pile (pile 1) are 3.4% and 4% for rigid and flexible raft conditions (Figures 7.39 and 7.40). For the same ground movements, the reduction in settlements for the corner pile (pile 3) are 3.6% and 3.5% for rigid and flexible raft conditions respectively.

The central pile (pile 1) in a free standing pile group, subjected to an edge heave ground movement \((S_o = 80\ mm)\), experiences 2% and 1% reduction in settlement for rigid and flexible raft conditions (Figures 7.39 and 7.40). For the same ground movement, the reduction in settlement for the corner pile (pile 3) is 3% and 4% for rigid and flexible raft conditions, respectively.

It can be seen from Figure (7.40) that the uniform ground movement causes a higher reduction in the settlement of the central pile (pile 1), compared to the edge heave soil profile, whereas the corner pile (pile 3) settlement remains nearly unaffected; however, the difference is not very significant.

For a ground movement of 80mm, the central pile (pile 1) in a piled raft system experiences 34% and 4% reduction in settlement for rigid and flexible raft conditions. For the same ground movement, the corner pile (pile 3) experiences settlement reduction of 39% and 65% for rigid and flexible raft conditions, respectively. As mentioned earlier, the piled raft system is analysed for the edge heave soil profile only.

It can be seen from the above comparison that the ground movement due to the swelling soils has a very significant impact on piled raft systems, compared to free standing pile groups. The free standing group is less influenced by the ground movement because of the gap between the raft and the soil surface, due to which, no upward thrust is exerted on the raft.

It can also be seen from Figures (7.39) and (7.40) that, in the case of free standing pile groups, the reduction in settlement stabilises beyond a certain value of ground movement,
because of the slip which develops at the soil pile interface, so that no further transfer of load occurs on the pile shaft. In piled raft systems however, due to the thrust exerted under the raft, the settlement continues to reduce as the ground movement increases.

7.2.3.2 Load Distribution along the Pile Shaft

It can be seen from Figures (7.41) and (7.42) that the distribution of load along the pile shaft is dependent on the raft-soil-pile interface conditions. In the case of free standing groups, the upward ground movement influences the foundation, only by inducing an upward force along the pile shaft. Therefore, after full slip has developed at the soil pile interface, no further load is transferred to the pile shaft, and the stress along the pile shaft stabilises. In the case of the piled raft foundation system, the stress along the pile shaft is dictated by the upward thrust caused by the swelling soils. It counteracts the compressive stress at the pile heads and in some situations, a tensile stress may develop due to these forces. These tensile forces are anchored by the downward shear stress in the stable zone. In certain situations, the whole pile length experiences tension and negative shear stress due to the high tensile stress at the pile head.

It can be seen from Figures (7.41) and (7.42) that, in the case of the free standing pile group, the central pile (pile 1) when subjected to uniform ground movement, experiences a higher upward force compared to edge heave soil profile. The corner pile (pile 3) experiences a similar effect, because in both cases, the piles experience the same ground movement. It can be seen from Figure (7.41a) that for a rigid raft, the maximum upward force in the central pile (pile 1), due to the ground movement of 80 mm, is 772 kN and 85 kN for uniform and edge heave soil profiles, respectively. For the similar ground condition, the corner pile (pile 3) experiences upward forces of 786 kN and 786 kN for uniform and edge heave soil profiles, respectively.

7.2.3.3 Moment Analysis

The maximum moment and the moment along section BB are presented in Figures (7.43) to (7.46). In the case of free standing pile groups subjected to 80 mm ground movement, for rigid rafts, the decrease in the maximum positive moment is 14% for a uniform ground movement, whereas the increase in maximum positive moment is 10% for the edge heave soil profile (Figure 7.45). For the same ground movement, the reduction in the maximum negative moment is 5% and 4% for uniform and edge heave ground movement, respectively.
Chapter 7

Behaviour of Piled Raft Systems in Swelling Soils

In the case of a flexible raft subjected to 80 mm ground movement, the decrease in the maximum positive moment is 3% for an uniform ground movement, whereas there is an increase in maximum positive moment of 7% for edge heave soil profile (Figure 7.46). For the same ground movement condition, the reductions in the maximum negative moment are 0% and 10% for uniform and edge heave soil profiles, respectively.

In the case of the piled raft foundation system, due to the upward ground movement towards the edge of the raft, the positive moment increases and the negative moment decreases with increasing in the ground movement. For the rigid raft ($K_r = 1$), the increase in the maximum positive moment with respect to the flat ground condition is 200% and 10% for a piled raft system and free standing pile group, both subjected to 80 mm edge heave ground movement, respectively (Figure 7.45). For the same ground movement, the reduction in the maximum negative moment is 103% and 7% for the piled raft system and free standing pile group, both subjected to an edge heave soil profile, respectively (Figure 7.45). The increase in the maximum positive moment is very high in piled raft systems compared to the free standing pile group, because of the upward thrust exerted by the swelling soil.

7.3 PRACTICAL IMPLICATIONS

It is important to understand the influence of swelling ground movements on the foundation behaviour at various stages of construction and performance. The potential problems are discussed below:

- The analysis indicates that, in the case of an unloaded or lightly loaded raft, due to the swelling movements, a tensile stress may develop at the pile raft interface, because the zero or small imposed load on the raft is unable to counteract the swelling force.

The condition of a structure, which is only partly constructed and is left for the full cycle of seasonal wetting and drying process, may be critical in a swelling soil condition. The small magnitude of imposed load may not be able to counteract the thrust underneath the raft caused by the swelling soils, and as a result, tensile stress may develop at some of the pile heads, depending on the raft thickness. These tensile stresses may cause cracking at the pile-raft interface, which may remain undetected. This may have an adverse effect on the overall integrity of the structure.

If a structure is only partly constructed and then left for a full cycle of seasonal moisture variation, the swelling soil will cause an upward movement of the edge of the raft and the corner piles and the raft will be subject to a movement profile prior to the
superstructure loading on the foundation. Further construction of the structure will impose additional compressive load on the foundation and will counteract the swelling force. As a result of this, the foundation movement profile developed by the swelling soil will subside, but the superstructure components may experience a change in the relative support condition. This may induce undesirable stresses in the superstructure.

- The swelling soil tends to cause an increase in the differential settlement, and therefore an allowance should be made in designing the structure and the foundation to cater for these differential settlements.

- The ground movement causes a dramatic change in the moments in the raft, depending on the raft thickness. Therefore, the moment capacity of the foundation system should be thoroughly investigated to cater for these moment variations. The transfer of these moments to the superstructure are usually be neglected in design and this may affect the superstructure adversely.

Therefore, the following aspects should be taken into account for piled raft systems in a highly reactive soil condition:

- The foundation should not be left unloaded for a full seasonal change
- The pile raft connection should be designed to account for the worst situation i.e. the foundation without any imposed load on the raft.
- A gap between the raft and the soil may be provided in order to reduce the impact of the upward ground movement, if the ultimate capacity of the foundation is not a problem.
- The beam - column connection should be designed for the induced moment caused by the changed settlement profile of the foundation raft.
- The length of the pile towards the edge of the raft should be designed to withstand the induced load by the swelling soils under lower imposed load conditions.
- Proper drainage should be provided in order to eliminate any possibility of potential water stagnation in the proximity of the structure.

The provision of a low friction coating on the pile shaft in the reactive zone, in order to reduce the upward force along the shaft, may not be very effective, as the majority of the upward force in the piled raft foundation is caused by the thrust underneath the raft.

7.4 SUMMARY

The movement of the ground due to the swelling soils has a significant impact on the performance of the piled raft foundation systems. The relative movement of the ground underneath the raft and along the pile cause a change in the state of contact stress as well as
the distribution of stress along the pile shaft. This phenomenon influences the load sharing between the raft and the pile very significantly. The difference in behaviour is dependent on the geometrical property of the foundation, and the number and location of the piles underneath the raft. The size of the raft influences the shape of the edge heave formation under the raft and the location of the pile influences the magnitude of the upward force along the pile shaft. A pile towards the outer periphery of the raft experiences higher upward force in comparison to an inner pile.

These upward forces influence the load distribution pattern between various piles, depending on the raft stiffness. In the case of a loaded raft, due to the upward ground movement, the load on the pile generally decreases with the increase in the ground movement due to the upward thrust exerted by the swelling soils underneath the raft. The influence of the ground movement on the inner piles is dependent on the location of piles as well as on the raft stiffness. In case of a rigid raft, due to the uniform movement of the raft, the effect of redistribution is very significant and higher load is transferred towards the central piles. In contrast, a flexible raft, the effect of redistribution is much less, and the effect is localised towards the edge of the raft. As a result of this, the swelling force is transferred to the outer piles, resulting in higher upward displacement of the corner piles.

The load on piles prior to the ground movement also plays a very important role in the load distribution. A highly loaded pile counteracts the swelling force, whereas, a lightly loaded pile may subject to tensile stress at the “pile-raft” interface due to the upward thrust underneath the raft. In the case of a flexible raft, the corner piles are worst affected by the swelling soils, as the raft is unable to redistribute the effect on the other foundation components and causes a localised effect.

The settlement behaviour of the foundation is dependent on the location and the number of piles underneath the raft and the raft stiffness. In the case of a loaded raft, the settlement of the piles reduces due to the upward movement of the soil. The corner piles experience higher force due to the greater upward thrust caused by the ground movement towards the edge of the raft compared to the central piles, and therefore, the reduction in settlement is higher in the corner piles. The difference in displacement between the central pile and the corner pile increases with the increase in the raft flexibility. In a rigid raft, due to the uniform movement, the difference in displacement is less and the force exerted by the swelling soils is redistributed to various foundation components. In contrast, in the case of a flexible raft, the effect of redistribution is less, and as a result, the piles experience a localised effect. Due to this, the piles towards the corner are subject to maximum upward displacement and the central piles experience less displacement, due to the lack of redistribution caused by the raft flexibility. The piles located between the central and the
corner piles initially experience less or no ground movement depending on their location, but the further increase in the ground movement influences these piles also. This process influences the pile and its effect is distributed to various foundation components, depending on the raft stiffness. The accurate assessment of this feature is quite complex and requires a thorough analysis by using a computer model (such as PRAE,5). Based on this non-linear process, the contact stresses underneath the raft are established.

The contact stress along the edge of the raft increases with an increase in the ground movement and the soil yields if the contact stress exceeds the limiting bearing stress value. Further increase in ground movement causes a redistribution and also increases the contact stress towards the centre of the raft as well as causing a further reduction in the pile load. The number and location of piles also influence the contact stress distribution beneath the raft, as they tend to anchor the upward force at various locations of the raft.

The upward ground movement along the raft edge causes an increase in the positive moment. The positive moment along a typical section increases with the increase in the ground movement. The increase in moment is higher in the case of a rigid raft, compared to a flexible raft. It is very important to note that, in the case of a loaded raft (Figures 7.23, 7.35, 7.43 and 7.44), the moment at the corner pile head is negative in a flat ground condition, but, with the increase in the ground movement, it reduces and beyond a certain point becomes positive. Therefore, it important to address this aspect in the design in order to cater for the change in the sign of the moment. In the case of a lightly loaded or unloaded raft, the increase in the positive moment is very rapid with the increase in the ground movement.

In the case of a free standing pile group, the upward ground movement causes an upward force along the pile shaft only, whereas for piled raft systems, the ground movement also causes an upward thrust on the base of the raft. Therefore, in free standing pile groups, after full slip has developed at the soil-pile interface, no further transfer of load takes place with an increase in the ground movement. As a result of this, the load distribution between various piles stabilises. The piles in the piled raft system still experience a reduction in load due to the upward thrust exerted by the swelling soils underneath the raft. The upward force is transferred to the raft base until the contact stress under the whole raft bottom exceeds the limiting bearing stress value. Therefore, the free standing pile groups are less influenced by the swelling soils, because of the gap between the raft base and the soil surface. Conversely, the piled raft systems are significantly influenced due to the contact with the soil surface.
The settlement behaviour of the free standing pile group in swelling soils is significantly different in comparison to the piled raft foundation system. In the case of a piled raft system, the upward soil movement induces an upward force along the pile shaft and also exerts a thrust under the raft. Therefore, even after the slip at the "soil-pile" interface, the upward thrust causes a reduction in the settlement. In the case of free standing pile groups, due to the gap between the raft bottom and the soil surface, the reduction in settlement stabilises beyond a point, whereas, in the case of piled raft systems, the reduction in settlement still continues with an increase in the ground movement.

In the case of uniform ground movement, the central piles experience similar ground movement as the corner piles, and therefore, it can be seen from Figures (7.41a) and (7.42a) that the central piles (pile 1) experience a higher reduction in settlement in comparison to the edge heave ground movement.

In the case of the free standing pile group, the stress along the pile shaft is less influenced by the ground movement than for a piled raft system (Figures 7.41 and 7.42), because the thrust exerted by the swelling soils underneath the raft causes a tensile stress at the "pile raft" interface, which influences the stress along the pile shaft very significantly.

In the case of free standing pile group, the load distribution along the shaft of the central pile (pile 1) is greatly influenced by the uniform soil movement, compared to edge heave ground movement. The uniform ground movement causes a greater influence on the load distribution along the shaft of the central pile (pile 1), compared to an edge heave ground movement. The difference in load is less for corner piles (pile 3), because they experience the same ground movement under both conditions.

The maximum positive moment increases and the maximum negative moment decreases with increasing ground movements. In the case of free standing pile groups, the change in moment stabilises, after slip takes place at the soil-pile interface. For the piled raft system, even after the slip, an increase in the positive moment takes place due to the thrust exerted by the swelling soils.
**Piled Raft Analysis**

$2^2$ Pile Group Analysis
Soil parameters:

- Soil Poisson's ratio = 0.3
- Soil modulus = 20 MPa

Pile:

- $L_p = \text{Length of pile} = 20 \text{ m}$
- $D = \text{Pile diameter} = 1 \text{ m}$
- $K_p = \text{Pile compression} = 10^3$
- $E_p = \text{Pile modulus} = 2 \times 10^4 \text{ MPa}$

Raft:

- $L_r = \text{Length of raft} = 7 \text{ m}$
- $B_r = \text{Width of raft} = 7 \text{ m}$
- $E_r = \text{Raft modulus} = 2 \times 10^4 \text{ MPa}$

Figure 7.1a: Plan (Quarter portion only)

Figure 7.1b: Section XX (Quarter portion only)

Figure 7.1: Piled Raft Arrangement (2^2 pile group), Quarter portion only
2\textsuperscript{nd} Pile Group Analysis

Swelling Soil Analysis
Elastic and Non Linear Comparison
$L_R/D = 15$

- $S_0 = 0$
- $S_0 = 40\text{mm}$
- $S_0 = 80\text{mm}$
- $S_0 = 120\text{mm}$

Settlement in mm

Figure 7.2: Elastic and non linear analysis ($L_R/D = 15$)

2\textsuperscript{nd} Pile Group Analysis

Swelling Soil Analysis
Elastic and Non Linear Comparison
$L_R/D = 50$

- $S_0 = 0$
- $S_0 = 40\text{mm}$
- $S_0 = 80\text{mm}$
- $S_0 = 120\text{mm}$

Settlement in mm

Figure 7.3: Elastic and non linear analysis ($L_R/D = 50$)
Figure 7.4: Load sharing characteristics of pile

Figure 7.5: Settlement analysis of $2^2$ pile group (Applied Load = 0, $K_R = 1$)
2° Pile Group Analysis

Swelling Soil Analysis
Settlement Analysis
Applied Load = 0.25 \( P_{ult} \)

- Settlement at raft centre
- Settlement at pile head

\[ K_R = 1 \]
\[ K_R = 0.01 \]

Figure 7.6: Settlement analysis of 2° pile group (Applied Load = 0.25 \( P_{ult} \))

2° Pile Group Analysis

Swelling Soil Analysis
Applied Load = 0

\[ K_R = 1 \]
\[ K_R = 0.01 \]

Figure 7.7(a): Distribution of load (Applied Load = 0)

Figure 7.7(b): Distribution of load (Applied Load = 0.25 \( P_{ult} \))

Figure 7.7: Distribution of load along pile shaft
**Chapter 7**

*Behaviour of Piled Raft Systems in Swelling Soils*

### 2^2 Pile Group Analysis

#### Swelling Soil Analysis

**Contact Pressure Distribution Along AA**

**Applied Load = 0**

- : $K_x = 1$
- : $K_x = 0.01$

- $So = 40\text{mm}$
- $So = 80\text{mm}$
- $So = 120\text{mm}$

**Figure 7.8:** Distribution of contact stress along AA (Applied Load = 0)

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### 2^2 Pile Group Analysis

#### Swelling Soil Analysis

**Contact Pressure Distribution Along AA**

**Applied Load = 0.25 P_{nh}**

- : $K_x = 1$
- : $K_x = 0.01$

- $So = 0$
- $So = 40\text{mm}$
- $So = 80\text{mm}$
- $So = 120\text{mm}$

**Figure 7.9:** Distribution of contact stress along AA (Applied Load = 0.25 P_{nh})
Chapter 7
Behaviour of Piled Raft Systems in Swelling Soils

2^2 Pile Group Analysis

Swelling Soil Analysis
Moment Along BB
Applied Load = 0

- $K_R = 1$
- $K_R = 0.01$

Figure 7.10: Moment analysis along typical section BB (Applied Load = 0)

2^2 Pile Group Analysis

Swelling Soil Analysis
Moment Along BB
Applied Load = 0.25 $P_{ult}$

- $K_R = 1$
- $K_R = 0.01$

Figure 7.11: Moment analysis along typical section BB (Applied Load = 0.25 $P_{ult}$)
Figure 7.12: Maximum moment analysis (Applied Load = 0)

Figure 7.13: Maximum moment analysis (Applied Load = 0.25 \( P_{ub} \))
Plate 7.1: Stress distribution (in kPa) between piles and raft ($P = 0.25 P_{ab} S_o = 0$)

Plate 7.2: Stress distribution (in kPa) between piles and raft ($P = 0.25 P_{ab} S_o = 80$ mm)
Plate 7.3: Moment (kN-m/m) in X-direction \( (P = 0.25 \, P_{ub}, S_o = 0) \)

Plate 7.4: Moment (kN-m/m) in X-direction \( (P = 0.25 \, P_{ub}, S_o = 80 \, mm) \)
Plate 7.5: Displacements in metres \((P = 0.25 \, P_{sb}, \, S_o = 0)\)

Plate 7.6: Displacements in metres \((P = 0.25 \, P_{sb}, \, S_o = 80 \, mm)\)
Piled Raft Analysis

4^2 Pile Group Analysis
Soil parameters:

- Poisson’s ratio = 0.3
- Soil modulus = 20 MPa

Pile:
- \( L_p = \text{Length of pile} = 20 \text{ m} \)
- \( D = \text{Pile diameter} = 1 \text{ m} \)
- \( K_p = \text{Pile compression} = 10^3 \)
- \( E_p = \text{Pile modulus} = 2 \times 10^4 \text{ MPa} \)

Raft:
- \( L_R = \text{Length of raft} = 20 \text{ m} \)
- \( B_R = \text{Width of raft} = 20 \text{ m} \)
- \( E_R = \text{Raft modulus} = 2 \times 10^4 \text{ MPa} \)

Figure 7.14a: Plan (Quarter portion only)

Figure 7.14b: Section XX (Quarter portion only)

Figure 7.14: Piled Raft Arrangement (4² pile group), Quarter portion only
Figure 7.15: Load sharing characteristics of pile (Applied Load = 0.25 P_{ub})

Figure 7.16: Settlement analysis (Applied Load = 0)
4th Pile Group Analysis

Figure 7.17: Settlement behaviour (Applied Load = 0.25 P_{ult})
Figure 7.18: Stress distribution along pile shaft (Applied load = 0)

4th Pile Group Analysis

Figure 7.19: Load distribution along pile shaft (Applied Load = 0.25 P_uk)
4th Pile Group Analysis

Swelling Soil Analysis
Contact Pressure Distribution Along AA
Applied Load = 0

- : $K_R = 1$
- : $K_R = 0.01$

- $So = 40\text{mm}$
- $So = 80\text{mm}$
- $So = 120\text{mm}$

Figure 7.20: Contact pressure distribution along typical section AA (Applied Load = 0)

4th Pile Group Analysis

Swelling Soil Analysis
Contact Pressure Distribution Along AA
Applied Load = 0.25 $P_{ui}$

- : $K_R = 1$
- : $K_R = 0.01$

- $So = 0$
- $So = 40\text{mm}$
- $So = 80\text{mm}$
- $So = 120\text{mm}$

Figure 7.21: Contact pressure distribution along typical section AA (Applied Load = 0.25 $P_{ui}$)
4^2 Pile Group Analysis

Figure 7.22: Moment distribution along typical section BB (Applied Load = 0)

Figure 7.23: Moment distribution along typical section BB (Applied Load = 0.25 P\text{ub})
4² Pile Group Analysis

**Figure 7.24:** Maximum moment analysis (Applied Load = 0)

4² Pile Group Analysis

**Figure 7.25:** Maximum moment analysis (Applied Load = 0.25 $P_{uh}$)
Plate 7.7: Stress distribution (in kPa) between piles and raft \((P = 0.25 P_{ub} \ S_o = 0)\)

Plate 7.8: Stress distribution (in kPa) between piles and raft \((P = 0.25 P_{ub} \ S_o = 80 \text{ mm})\)

Plate 7.10: Moment (kN-m/m) in X-direction \((P = 0.25 P_{ub} \ S_o = 80 \text{ mm})\)
Plate 7.9: Moment (kN-m/m) in X-direction \( (P = 0.25 \, P_{ult} \, S_o = 0) \)

Plate 7.10: Moment (kN-m/m) in X-direction \( (P = 0.25 \, P_{ult} \, S_o = 80 \text{ mm}) \)
Plate 7.11: Displacements in metres ($P = 0.25 P_{alb} S_o = 0$)

Plate 7.12: Displacements in metres ($P = 0.25 P_{alb} S_o = 80$ mm)
Piled Raft Analysis

$6^2$ Pile Group Analysis
**Soil parameters:**

- Soil Poisson's ratio = 0.3
- Soil modulus = 20 MPa

**Pile:**

- \( L_p = \) Length of pile = 25 m
- \( D = \) Pile diameter = 1 m
- \( K_p = \) Pile compression = \(10^3\)
- \( E_p = \) Pile modulus = \(2 \times 10^4\) MPa

**Raft:**

- \( L_R = \) Length of raft = 45 m
- \( B_R = \) Width of raft = 45 m
- \( E_R = \) Raft modulus = \(2 \times 10^4\) MPa

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**Figure 7.26 a:** Plan (Quarter portion only)

**Figure 7.26 b:** Section XX (Quarter portion only)

**Figure 7.26:** Piled Raft Arrangement (6² pile group), Quarter portion only
Figure 7.27: Load sharing characteristics of piles (Applied Load = 0.25 \( P_{ab} \))

Figure 7.28: Settlement analysis (Applied Load = 0)
6. Indicates Pile 6

Central of the raft

6^2 Pile Group Analysis

Figure 7.29: Settlement analysis (Applied Load = 0.25 P_{ub})
6. Pile Group Analysis

Fig. 7.30a: Load distribution along pile 1
Fig. 7.30b: Load distribution along pile 2
Fig. 7.30c: Load distribution along pile 3

Fig. 7.30d: Load distribution along pile 4
Fig. 7.30e: Load distribution along pile 5
Fig. 7.30f: Load distribution along pile 6

Figure 7.30: Load distribution along pile shaft (Applied Load = 0)
6. Pile Group Analysis

Fig. 7.31a: Load distribution along pile 1
Fig. 7.31b: Load distribution along pile 2
Fig. 7.31c: Load distribution along pile 3
Fig. 7.31d: Load distribution along pile 4
Fig. 7.31e: Load distribution along pile 5
Fig. 7.31f: Load distribution along pile 6

Figure 7.31: Load distribution along pile shaft (Applied Load = 0.25 Pnk)
6^2 Pile Group Analysis

Swelling Soil Analysis
Contact Pressure Distribution Along AA
Applied Load = 0

- : $K_R = 1$
- : $K_R = 0.01$

- $S_0 = 40\, \text{mm}$
- $S_0 = 80\, \text{mm}$
- $S_0 = 120\, \text{mm}$

Figure 7.32: Contact pressure distribution along typical section AA (Applied Load = 0)

6^2 Pile Group Analysis

Swelling Soil Analysis
Contact Pressure Distribution Along AA
Applied Load = 0.25 $P_{ult}$

- : $K_R = 1$
- : $K_R = 0.01$

- $S_0 = 0$
- $S_0 = 40\, \text{mm}$
- $S_0 = 80\, \text{mm}$
- $S_0 = 120\, \text{mm}$

Figure 7.33: Contact pressure distribution along typical section AA (Applied Load = 0.25 $P_{ult}$)
Figure 7.34: Moment distribution along typical section BB (Applied Load = 0)

Figure 7.35: Moment distribution along typical section BB (Applied Load = 0.25 \( P_{w} \))
Figure 7.36: Maximum moment analysis (Applied Load = 0)

Figure 7.37: Maximum moment analysis (Applied Load = 0.25 P_{uk})
**Plate 7.13**: Stress distribution (in kPa) between piles and raft ($P = 0.25\ P_{ult} S_o = 0$)

**Plate 7.14**: Stress distribution (in kPa) between piles and raft ($P = 0.25\ P_{ult} S_o = 80\ mm$)
Plate 7.15: Moment (kN-m/m) in X-direction \((P = 0.25 P_{ub} S_o = 0)\)

Plate 7.16: Moment (kN-m/m) in X-direction \((P = 0.25 P_{ub} S_o = 80 \text{ mm})\)
Plate 7.17: Displacements in metres ($P = 0.25 P_{ub}, S_o = 0$)

Plate 7.18: Displacements in metres ($P = 0.25 P_{ub}, S_o = 80$ mm)
Piled Raft Foundation System

and

Free Standing Pile Group

A Comparative Study
Figure 7.38 (a) : Piled raft system subjected to edge heave  

Figure 7.38 (b) : Free Standing group subjected to edge heave  

Figure 7.38(c) : Free Standing group subjected to uniform ground movement  

Figure 7.38 : Piled Raft vs Free Standing pile group - Pile and Raft arrangement
Figure 7.38 (a): Piled raft system subjected to soil mound
Figure 7.38 (b): Free Standing group subjected to soil mound
Figure 7.38 (c): Free Standing group subjected to uniform ground movement

Figure 7.38: Piled Raft vs Free Standing pile group - Pile and Raft arrangement
Piled Raft Foundation System vs Free Standing Pile Group

Figure 7.39: Settlement analysis ($K_R = 1$)

Figure 7.40: Settlement analysis ($K_R = 0.01$)
Piled Raft Foundation System vs Free Standing Pile Group

Figure 7.41: Load distribution along pile shaft ($K_R = 1$)

Legend:

- **Piled Raft Systems**
- **Free Standing Group (Uniform Soil Movement)**
- **Free Standing Group (Edge Heave Movement)**

Figure 7.42: Load distribution along pile shaft ($K_R = 0.01$)

Piled Raft Foundation Systems in Expansive Soils
Chapter 7

Behaviour of Piled Raft Systems in Swelling Soils

Piled Raft Foundation System vs Free Standing Pile Group

Figure 7.43: Moment distribution along BB ($K_R = 1$)

Figure 7.44: Moment distribution along BB ($K_R = 0.01$)
Figure 7.45: Maximum moment analysis ($K_R = 1$)

Figure 7.46: Maximum moment analysis ($K_R = 0.01$)
CHAPTER - 8

COMPARISON OF RESULTS WITH FIELD AND
LABORATORY MEASUREMENTS

8.1. INTRODUCTION

In this chapter, a number of field and laboratory results are compared with the computed results in order to demonstrate the ability of the computer analysis to model the actual behaviour of the foundation systems. The site and the laboratory conditions are modelled and analysed by using the computer programs described in the previous chapters. Then the computed results are presented and compared with the actual measured data. The movement of the ground due to the swelling or shrinking soils is also incorporated in the analysis, where appropriate.

The case studies on the foundation systems have been obtained from various published sources, which provided the information on the foundations, which were instrumented, and their performances, which were generally monitored for a significant period of time. Other related aspects were also monitored and their influences on the foundation behaviour were observed. The effects of these aspects are also included in the analysis and presented for comparison purposes.

The analyses have been carried out by incorporating non-linear conditions. The limitation on stresses at the "soil - pile - raft" interfaces is incorporated, based on the soil parameters assessed to be relevant for at the site.

8.2. RAFT FOUNDATIONS

In this section, a full scale raft foundation system was modelled and analysed by using the computer program RAE, S (Raft Analysis in Expansive Soils). The computed results are compared with the field data, which were obtained by monitoring the slab foundation for a significant period of time. The information on the foundation details and the monitoring procedures have been obtained from the relevant publications.
The raft slab was extensively instrumented and the behaviour was monitored for several cycles of wetting and drying. The corresponding ground movements due to the wetting and drying process was also monitored. The settlements at the centre as well as at the corner of the raft were monitored for various heave conditions. The behaviour of the raft slab was monitored and presented in conjunction with the free ground heave formation.

8.2.1. RAFT BEHAVIOUR IN MELBOURNE

In the eastern states of Australia, the expansive nature of the clay soils and the prevailing climatic conditions lead to the development of significant seasonal soil surface movement in the top few metres. In the event of seasonal moisture variation, clay soils change in volume and are subject to vertical as well as horizontal movements. The horizontal movement causes opening of surface cracks in the drier periods, whereas the vertical movements lead to cyclic changes in the soil surface level. The magnitude of these movements decreases with depth down to a level below which no volume change occurs. The presence of an impermeable surface or a slab on the expansive soil will alter the pattern of seasonal moisture change. The slab acts as a barrier to the process of direct drying and wetting.

A general soil surface profile underneath the raft slab, which is influenced by the wetting and drying cycles, is indicated in Figure (8.1). It can be seen that the ground surface, which was flat at the time of laying the foundation slab, develops vertical surface movements due to the change in the moisture content. As a result of this, lightly loaded residential slabs experience a relative movement of the ground support. The slabs, without adequate stiffening beams, may be subject to excessive differential settlement and may experience structural failure.

In order to understand the actual field behaviour of raft slabs on expansive soils, a series of experimental slabs were constructed some years ago in the city of Melbourne, Victoria. These slabs were extensively instrumented and their deformations were monitored for a significant period of time. The observed behaviour of these foundations is presented in the following publications:

- "The design, performance and repair of housing foundations" by J.E.Holland (1981)

In this section, "Slab S3" is adopted for the comparisons. Further details on the foundation system and its related aspects can be obtained from the above sources.
8.2.1.1. DESCRIPTION OF THE PROBLEM

The size of the slab "S3" was 7.4 m × 7.4 m and it was loaded to simulate the condition of a single storey brick house construction. The slab was laid when the site was very dry, heavily fissured, and in its seasonally driest state. The details of the slab are shown in Figure (8.2). The behaviour of the raft slab and the respective ground heave were monitored simultaneously. The settlements at the centre as well as at the corner of the raft were measured for various heave conditions. The findings of the tests are presented in Figure (8.3).

The raft was analysed by using RAE5 (Raft Analysis in Expansive Soils) and the values obtained from the ground measurements and the results are compared with the field values. The ground movement was incorporated as described in section 3.2.4.1. The procedure for the analysis and the comparison are presented in the following sections.

8.2.1.2. ANALYSIS AND COMPARISON

In order to undertake the analysis, the quarter portion of the raft was analysed by dividing it into 121 elements and was loaded to simulate a single storey brick house construction. The thicknesses of the beam and raft elements were adopted as 300 mm and 100 mm, as constructed at site. To incorporate the effect of the ground movement at various stages of the wetting and drying process, a free field soil movement profile was adopted underneath the raft on the basis of the measured ground heave and the provided information in the paper. The initial and the intermediate ground movements, as indicated in Figure (8.3), were analysed on the basis of the approximate soil profile derived from the measured heave values, whereas the long term mound analysis was accurate as it was based on the actually measured soil profile (Figure 8.1e). The analysis was carried out with these soil profiles and the differential settlements are compared with the measured values. The comparison of results is presented in Table 8.1. The interaction between the raft and the soil was considered in the analysis, as described in Chapter 3. The influence of lift-off and the redistribution of stresses was also included in the analysis. Based on the information provided in the papers, the soil modulus (Et) and the Poisson's ratio (vt) were adopted as 8 MPa and 0.4, respectively. The analyses were undertaken for uncracked section.

As discussed previously, the computer program RAE5 incorporates a soil profile beneath the raft to include the effect of the ground movement underneath it. In carrying out the analysis, the soil profiles at various stages of the wetting and drying cycles were derived, based on the information provided in the paper. The following stages were considered to undertake the analysis:
• **Initial heave analysis**: The slab "S3" was constructed in the driest period of time, and therefore, during the first wet period, it was subjected to an upward ground movement. As discussed earlier, the ground movement towards the edge of the raft is higher, compared to the central raft and in this situation, the soil profile may be regarded as an "edge heave", as indicated in Figure (8.1b). To undertake the analysis in this state, a free field soil movement profile, based on 10 mm ground heave measurement, was derived and incorporated in the analysis. The results of the analysis are presented in Table 8.1.

• **Transition phase**: The following dry period caused drying of the soil towards the edge of the raft, but part of the moisture penetrated towards the central portion of the raft. This caused the central soil to swell and exert an upward thrust underneath the raft. During this phase, the central raft remained on a swollen soil surface, because the penetrated moisture remained entrapped under the central raft and was unable to evaporate due to the presence of the raft. The soil surface along the raft edge may however, be directly exposed to the seasonal moisture fluctuation. This phenomenon lifted the centre of the raft upwards depending on the imposed loading condition. The profile is indicated in Figures (8.1c & 8.1d). This may be regarded as a "transition phase" because the edge heave changed to a central heave formation underneath the raft. During this process, the differential settlement reduced and became zero and again increased in a reverse fashion. The phenomenon can be seen from Figure (8.3). This feature of the raft behaviour has also been analysed and discussed in Figure (3.28a) of Chapter 3.

• **Intermediate and long term mound analysis**: The analysis for the intermediate mound formation was based on the soil profile in between the "Transition Stage" and the long term mound formation underneath the raft slab. The paper indicated that the long term mound formation was a profile with 40 mm (y_m) differential movement. Based on this, the intermediate mound formation was considered as a soil profile falling in between the "transition phase" and the long term mound formation. The analysis of this stage was carried out with y_m = 20 mm. The computed values are presented in Table 8.1 and compared with the field measured values.

The above stages were analysed separately and discussed. The long term mound underneath a flexible cover was observed as indicated in Figure (8.1e). The observation indicated that these covers were subject to a differential mound of about 40 mm (y_m) with an edge distance of about 2.1 metres (e). The final mound analysis was carried out with these values and are presented in Table (8.1). The analysis of this
stage of the mound formation was accurate as the soil profile was accurately incorporated in the analysis. The computed differential settlement is compared with the field value and presented in Table 8.1. The maximum curvature (\(\Delta/L\)) is also presented for comparison purposes.

<table>
<thead>
<tr>
<th>Stage of Analysis</th>
<th>Computed Differential Settlement</th>
<th>Measured Differential Settlement</th>
<th>Computed Maximum Curvature ((\Delta/L))</th>
<th>Measured Maximum Curvature ((\Delta/L))</th>
<th>Heave Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial edge heave analysis</td>
<td>8 mm</td>
<td>6 mm</td>
<td>1/925</td>
<td>Not Available</td>
<td>Edge</td>
</tr>
<tr>
<td>Intermediate Mound Formation</td>
<td>12 mm</td>
<td>11 mm</td>
<td>1/616</td>
<td>Not Available</td>
<td>Central</td>
</tr>
<tr>
<td>Final Mound Formation</td>
<td>26 mm</td>
<td>24 mm</td>
<td>1 / 290</td>
<td>1/310</td>
<td>Central</td>
</tr>
</tbody>
</table>

# : Results based on approximate analysis  
* : Results based on actual observation

It can be seen from Table 8.1 that the computed results agree quite well with the field values.

8.2.1.3. DISCUSSION

The good agreement in between the computed values and the field data indicates the validity of the method of analysis. The analysis was carried out incorporating a "soil-structure interactive" approach. Therefore, it is necessary to incorporate an accurate soil movement profile in the analysis to achieve the accuracy of results. Due to the interaction and the redistribution of stresses, the loaded raft distorts the soil profile quite significantly. The distortion of the soil profile is dependent on the raft stiffness (\(K_R\)) or in the case of stiffened raft condition and also on the slab-beam configuration. The general behaviour of the uniform and the stiffened rafts is presented in Chapter 3.

8.3. PILE FOUNDATIONS

In this section, analyses were undertaken for two full scale pile foundation systems which were tested in expansive soils. The settlements and the stresses of these foundations were monitored for various heave conditions. These foundation systems were analysed by using the computer program PAES (Pile Analysis in Expansive Soils), as described in previous chapters, and the results are compared with the measured data. The field information and the test results were obtained from the relevant technical papers. A brief description of
these problems is also presented to explain the overall aspect of the foundation system and also the related testing strategies.

8.3.1. Pile Behaviour near the Town of Vereeniging in South Africa

A thermal power station was to be built on a site underlain by deep expansive soil. The climate of the site was generally temperate and semi-arid. The maximum surface heave was observed to be up to 120 mm. In order to restrict the differential movement, all structures were to be supported by bored cast insitu piles. Design for heavy loads was not a problem as the piles could be socketed into the carbonaceous shale at the base. However, there was a concern in the design of lightly loaded piles, which were to be installed two to three years prior to receiving their full dead load. The piles were supposed to experience a full cycle of drying and wetting every year. As a result of this, the upward force along the pile shaft due to the swelling effect was expected to be a potential problem. To cater for this problem, the piles were designed as anchor piles, including the effect of potential upward ground movement. Further details are contained in the following technical paper:


8.3.1.1. Description of the Problem

In order to observe this behaviour, a group of seven piles were installed, three of which were instrumented. The pile arrangement is shown in Figure (8.4). The instrumented piles 1, 2 and 3 were located to represent a corner pile, a side pile and an interior pile of a typical pile group. The main objective of adopting a group of piles was to observe the influence of one pile on another in a swelling soil situation. All these piles were straight-shafted, 1050 mm diameter and 33 metres long. All were reinforced with 12 mild steel bars of 25 mm diameter having a yield stress of 280 Mpa. The relative pile stiffness ($K_P$) was approximately 1100.

In order to measure the tensile stress along the pile shaft, the piles were instrumented with vibrating wire and electric resistance gauges. A vertical tube was also installed to water the soil and allow swelling to occur. The heave formation was also measured at various time intervals. The details of the installation procedure and their respective locations can be seen from the above paper. The stress distribution along the pile shaft was measured and
presented for piles 1, 2 and 3, as indicated in Figure (8.6). These stresses were caused due to the upward force induced by the expansive soil.

8.3.1.2. ANALYSIS AND COMPARISON

In order to carry out the analysis, each pile was divided into 11 elements and the measured upward ground movement was distributed along the pile shaft. The procedure of analysis was similar to as described in Chapter 4. The analysis was undertaken for a maximum heave of 88 mm extending up to a depth as indicated in Figure (8.5).

A large scale insitu tests, carried out at the site, indicated that the soil strength increased linearly with depth, with the minimum value as 30 kPa at the top. Based on this, an average soil strength 150 kPa was adopted for the analysis purpose. The $E_i/c_u$ and $c_d/c_u$ ratios were considered as 200 and 0.4 respectively. The limiting stresses were derived on the basis of the above assessment and incorporated in the analysis.

The analysis was carried out for the piles 1, 2 and 3 including the effect of non-linear conditions. The computed results are presented with the field measured data in Figure (8.6). It can be seen from Figure (8.6) that the load along the pile shaft and the location of the neutral plane obtained from the computer analysis agree quite well with the field values for the outer piles (piles 1&2). The uplift force along the inner pile shaft (pile 3) obtained from the analysis is found to be approximately 15% higher than the field value. The reason for this feature is discussed in the following section.

8.3.1.3. DISCUSSION

As discussed in previous chapters, the piles tend to restrict the upward movement of the ground due to its anchorage in the stable zone. In a group situation, the number, spacing and length of piles may have some effect on the ground movement formation within the pile group area due to the restraining effect created by the piles. Due to this, there may be a reduction in the ground movement towards the centre of the group. As a result of this, the inner piles may experience lesser ground movement compared to the outer piles. Therefore, it can be seen from Figure (8.6) that in case of inner pile (pile 3), the analysis overestimates the uplift force by approximately 15%. In this case, the spacing between the piles is less ($S/D = 2.5$), which might have caused larger reduction in the ground movement. It is obvious that smaller spacing between piles will have greater influence on the ground movement. In normal cases, where $S/D$ varies from 5 to 7.5, this restraining effect is expected to be less. It should be noted that it is necessary to undertake further research work to establish the restraining effect created by the group of piles in the process.
of ground movement. This will improve the accuracy of results. This characteristic has also been discussed in section 9.5 to outline the scope of future research work.

Generally, the results obtained from the computer analysis agree quite well with the field data. The proposed method of analysis is found satisfactory to represent the actual behaviour of the pile foundation systems in a reactive soils, provided the soil parameters are adopted, adequately. Nevertheless, there is still scope for further work to improve the analytical method and to improve predictive capability for pile groups. These aspects are discussed in section 9.5 in detail.

8.3.2. PILE BEHAVIOUR AT INDORE

A full scale pile was constructed and monitored for a significant period of time. The pile displacement and the stresses along the pile shaft were measured and presented. The work was carried out by Bhandari et al and the findings were presented in the Sixth International Conference on Expansive Soils, 1987. The details of the tests and the observations are described in the following sections.

8.3.2.1. DESCRIPTION OF THE PROBLEM

In the field test, two sets of three different types of concrete piles comprising uniform diameter, single and double underreamed piles, were constructed to observe their behaviour in expansive soils. In this section, the uniform diameter pile was considered for analysis and comparison purposes. The diameter and the length of these piles were 300 mm and 3.5 m respectively. There were two uniform diameter piles constructed, out of which one was instrumented with the base load cell and two shaft load cells and the other was provided with only the base load cell. The load cells were designed and constructed by using vibrating wire type load sensing units, which were capable of monitoring up to 100 kN axial loads.

The test was carried out in black cotton soil (also known as black soil). The ground is subject to shrinkage between March and mid June and tends to swell during the rainy season that usually occurs between June and September. The site observations during summer indicated cracks of a polygonal pattern along the depth. The width of the cracks varied between 3 mm and 80 mm, with the large one extending to a depth of about 2 m.

The sub-soil consisted of blackish silty clay up to 2.7 m to 2.9 m depth followed by yellowish clay with nodules up to 5 m depth. No water table was observed up to this depth. The soil type falls under the category of CH (medium to high plastic clay). The variation
of moisture content extended to a depth of 4.5 m, indicating the depth to be the swelling zone of about 4.5 m. The surface heave due to swelling soil was observed as 60 mm.

Cone penetration resistance indicated that the site stratigraphy was almost uniform. The penetration resistances and the shear strength tests showed that the strength increased with depth. The values of undrained shear strength \((C_u)\) obtained in the laboratory agreed well with those obtained from static penetration tests, using the relationship \(C_u = q_c/20\) \((q_c =\) static cone penetration test resistance), suggested by various workers (Sanglerat, 1972). Based on static cone resistance, the values of the soil modulus \((E_s)\), using correlations summarised by Sanglerat, were found as 3.5-5 MPa for the top 2 m, 10-15 MPa for the next metre and 7.5-10 MPa for 3m to 4.5 m depth.

The readings of the load cells were taken towards the end of the summer peak and were considered as the control readings for the subsequent changes. During the rainy season, as the sub-soil expanded, the observations were taken at monthly intervals. At every stage of measurement, the observations were continued for a sufficient length of time, at least 3 days in each case.

The field observations and the analysed results are presented and compared in following section.

8.3.2.2. ANALYSIS AND COMPARISON

In order to carry out the analysis, the pile was divided in 7 elements and 8 nodes and the movement of the ground was distributed along the pile shaft, linearly. The analysis was carried out, incorporating the effects of non-linear conditions at the soil-pile interface. Based on the field tests, the soil modulus was recommended as 5000 kPa in the paper. A limiting shear stress of 37.5 kPa was assumed along the pile shaft in the swelling zone as well as in the stable zone. The analysis was undertaken for the various stages of the ground movement increase. The maximum movement of the ground was adopted as 60 mm, based on the field observation. The ground movement was distributed along the pile shaft linearly up to a depth of 3.5 metres with the maximum value at the soil surface.

The computed upward force along the pile shaft with the pile displacement is presented in Figure (8.8 i). The maximum uplift force due to 60 mm ground movement was computed as 53 kN in comparison to the field value of 46 kN. The difference in values is approximately 15%. The upward displacement of the pile for a ground movement of 60 mm was computed as 23 mm in comparison to the field value as 18 mm. The stress along
the pile shaft was also analysed and presented in Figure (8.8 ii). The computed stress distribution along the pile shaft indicates good agreement with the field data.

8.3.2.3. **DISCUSSION**

The comparison indicates fairly reasonable agreement between the computed values and the field measurements.

In the analysis, the extent of the ground movement and its distribution along the pile shaft have very significant impact on the final values. It is generally recommended to measure not only the heave, but also its distribution along the depth, which helps to incorporate the actual induced upward force at different depths of the pile. In the analysis, the 60 mm ground movement was linearly distributed along the full length of the pile. It is quite interesting to note that two piles in the same test site (3.5 metres apart) experienced 16 and 18 mm upward displacements, which is a difference of approximately 13%. It is obvious that the swelling force along the pile which experienced higher displacement was higher than the other pile. In this situation, if the ground movement distribution along the shaft is known, then the computer analysis can achieve a higher degree of accuracy.

The swelling soil induced an upward force along the pile shaft and caused an upward displacement under a zero imposed load condition. These upward stresses were counteracted by the downward stress in the lower portion of the pile shaft in the stable zone, as indicated in Figure (8.8 ii). The plane at which the stress along the pile shaft changes its sign is defined as the “neutral plane”, and is shown in Figure (8.8 ii). The location of the neutral plane is in good agreement with the field measurement. This is a very important aspect of the comparison, as it indicates the capability of the method of analysis to assess the length of the pile in the stable zone. The accurate determination of the neutral plane at a certain ground movement provides value information on the load which the pile can carry without excessive movement. These aspects of the pile behaviour are discussed in Chapter 4 in detail.

Based on these comparisons, the method of analysis adopted to formulate PARES is assessed adequate for its purpose.

8.4. **PILED RAFT FOUNDATIONS**

In this section, the measured values from two instrumented piled raft foundation systems are compared with the analysed results. The analysis was carried out by using PARES (Piled Raft Analysis in Expansive Soils), as described in previous chapters. The settlement
behaviour and the load distribution feature in between the pile and the raft were analysed and compared with the actual measured values. The distribution of load along the pile shaft is also discussed. These analysed results are compared with the data obtained from the field or the laboratory measurements.

In a laboratory, tests are carried out in a controlled condition to represent an ideal foundation situation, whereas in the field, other parameters may also be present which influence the foundation behaviour quite significantly. For example, the location of the water table and its fluctuation may induce additional forces along the pile shaft. Similarly, the sequence of construction may also influence the load distribution pattern in the foundation. Therefore, in the following sections, two cases have been considered for comparison purposes, one representing laboratory testing and the second one for a full scale instrumented piled raft system.

8.4.1. LABORATORY TEST : PILED RAFT MODEL TESTING BY THE BUILDING RESEARCH ESTABLISHMENT, LONDON

A series of tests with models of piled raft foundations of various sizes was undertaken by the Building Research Establishment in London. These tests were carried out in a laboratory and the behaviour of these foundations was examined and presented in the paper by Cooke (1986).

8.4.1.1. DESCRIPTION OF THE PROBLEM

The configuration adopted for the analysis is shown in Figure (8.9). It is a $7^2$ piled raft group with a spacing / diameter ratio of 3. The piles were formed of brass rods 3.2 mm in diameter and were 150 mm long. The piles were connected to a thick plate, representing a rigid raft condition. The load was imposed on the piles through the raft by means of a loading ball. The detailed arrangement of the set up can be seen from the above mentioned paper.

The tests were carried out in London Clay remoulded with additional water to achieve an undrained shear strength of between 5 kPa and 15 kPa. The $E/c_u$ value was quoted as 400 in the original paper.

8.4.1.2. ANALYSIS AND COMPARISON

The analysis was carried out for the piled raft system indicated in Figure (8.10). The raft was considered as rigid plate and the load was applied at the centre. In order to carry out
the analysis, symmetry was used, and the quarter raft was discretised into 64 elements and 81 nodes, and each pile was divided into 15 elements.

The selection of soil parameters was based of the information provided in the paper. The literature indicated that although $E/c_u$ may vary widely for London clay, it is frequently considered to be of the order of 400. In the analysis, the soil parameters were based on an undrained soil shear strength of 15 kPa.

The limiting shear stress along the pile shaft was obtained by adopting an adhesion ($\alpha$) factor ($c_d/c_u$) of one. The limiting stress at the pile base was taken to be $9$ $c_u$. These conditions were incorporated to carry out the analysis and the results are presented in the Figure (8.10).

The settlement of the piled raft system was analysed for various increments of loading. These results are presented in Figure (8.10) and compared with the actual observed value. It can be seen from the figure that the comparison indicates a very good agreement with the values obtained in the laboratory test.

8.4.1.3. Discussion

The good agreement indicates the validity of the method adopted to formulate the computer program. It is understood that the testing of the foundation systems in laboratories is carried out under highly idealised conditions. Therefore, the foundation performance is unaffected by various parameters which are generally present in full-scale foundation systems. In such ideal situations, provided that appropriate parameters are employed, the agreement with the actual values is generally accurate and the analytical results represent the actual behaviour of the foundations quite well.

8.4.2. Full Scale Foundation Analysis: Piled Raft Behaviour in Frankfurt

Messe Turm is the tallest building in Frankfurt and the in Europe and by far the tallest building in Frankfurt. The structure is 256 m high, with 60 stories. Therefore, the tilt of the overall structure was a serious point of concern in the design stage. The initial analysis indicated that in the case of a mat foundation, the expected settlements would be in the order of 350 mm to 400 mm. Therefore, to reduce the settlement and tilt of the structure, a series of large diameter piles were introduced strategically underneath the raft. The length of these piles varied from 27 m to 35 m into the clay below the mat. Longer piles were concentrically placed towards the central portion of the raft, in order to reduce the
settlement. As discussed previously, the strategic placement of piles underneath the raft can reduce the potential differential settlement in the raft (Burland et al. 1977). The foundation system was fully instrumented and monitored during various stages of loading and construction. The measured data and their findings are published in the following papers:

- Sommer et al. in the Fourth International DFI Conference 1991 Balkema, Rotterdam, pp. 139-145.

An overview of the foundation system and its related aspects are presented in the following section. The information was extracted from the above published papers. The foundation has been analysed by using the computer program PRAE5S and the results are compared with the field measured data. A brief discussion is also presented on the influence of the construction sequence on the foundation performance.

8.4.2.1 DESCRIPTION OF THE PROBLEM

The size of the raft is 58.8 m × 58.8 m. The thicknesses towards the centre and the edge of the raft are 6 m and 3 m, respectively. The raft is supported by 64 piles and the diameter of the piles is 1.3 m. The pile lengths varied from 26.9 m to 34.9 m. There were 16 piles (34.9 m long) towards the central portion of the raft, 28 piles (26.9 m long) towards the outer side of the raft and 20 piles (30.9 m long) in between the central and the edge piles. The longer piles were configured towards the centre of the raft in order to reduce the settlement and also to achieve an even distribution of load on piles. The geometrical configuration of the foundation and the location of piles are shown in Figure (8.11).

8.4.2.2 ANALYSIS AND COMPARISON

In order to undertake the analysis, use was made of symmetry, and the quarter raft was discretised into 256 elements and 289 nodes and the longest and the shortest piles were divided into 18 and 14 elements respectively. The analysis included the various lengths of the piles and the variable raft thickness. It was assumed that due to the thickness of the raft, the imposed load was uniformly distributed over the raft. The intensity of load was adopted as 470 kPa, including the effect of buoyancy.

The literature (Franke, 1991) mentioned that the previous observation by Amann (1975) indicated widely deviating values of soil modulus of the Frankfurt clay. Therefore, for
analysis purposes, the soil modulus was obtained on the basis of the undrained shear strength \(c_u\) values. Based on various information, it was concluded that the overconsolidation ratio of the Frankfurt clay is not very high (Franke, 1991). Therefore, for the analysis purposes, the range of \(E/c_u\) was adopted as 750 to 1200 for lightly overconsolidated clay. The undrained shear strength of the soil system varied approximately from 100 kPa to 500 kPa from a depth of 14 m to 50 m (Figure 3 of Sommer et al, 1991). Based on these values, an average shear strength value was adopted as 300 kPa. Other soil parameters were derived on the basis of this information. The details of the soil properties such as liquid limit, plasticity index and others can be found from the above papers.

In order to determine the limiting shear stress \(\tau_{sud} = c_u\) value, the \(\alpha\) factor \((c_u/c_u)\) was taken to be as 0.4. The limiting stress at the pile base was adopted as 9 \(c_u\). The analysis was undertaken with above values and the results are discussed in the following sections.

As discussed above, the sequence of construction may also have a significant influence on the foundation behaviour. In this case, the construction of the foundation system was undertaken in various stages and the analysis was carried out incorporating these conditions. The various stages of construction were as follows:

**Stage - 1:** Initially the ground surface was excavated up to a depth of 7 metres

**Stage - 2:** The piles were constructed leaving the upper 7 metres empty

**Stage - 3:** The ground water table was lowered by 7 metres

**Stage - 4:** The pit was further excavated to the final depth of 14 metres.

The stages of the construction are shown in Figure (8.12). It is obvious that, due to the lowering of the water table in Stage -3, the ground must have experienced a downward movement, causing a drag force along the pile shaft. In contrast, the excavation of 7 metres in Stage - 4 would have caused an elastic rebound due to the release of surcharge up to the pile head level. Due to this, the ground must have experienced an upward movement. The forces developed in Stage 3 and 4 must have counteracted each other and the resultant of these two forces must have acted along the pile shaft. In other words, in a situation where both phenomena take place simultaneously, or one after another prior to the imposed loading, the resultant of these two processes will induce stresses along the pile shaft. These stresses and the ground movements were not measured, due to construction problems. However the observations indicated that there were some upward induced stresses locked along the pile shaft prior to imposing the superstructure loads on the foundation (Franke, 1991). With the construction of the superstructure, these stresses were counteracted by the imposed load on the piles.
In the analysis, the negative ($-S_o$) and positive ($+S_o$) ground movements indicate upward and downward ground movements, respectively. If the influence of lowering the ground water table is dominant, compared to the elastic rebound, the resultant stresses will be downward, whereas a higher elastic rebound will cause stresses in the upward direction. A balanced condition ($S_o = 0$) indicates that the forces, developed in stages 3 and 4, nullified each other and the pile shaft remains unaffected by any locked-in stresses prior to imposed loading. In order to indicate these potential ground movement conditions, the following three conditions were analysed and compared with the actual measurements.

- $S_o = 0$: The stresses induced in stage 3 and 4 have nullified each other
- $S_o = +30\ mm$: The presence of locked stresses due to an upward ground movement of 30 mm
- $S_o = -30\ mm$: The presence of locked stresses due to a downward ground movement of 30 mm

The results are presented in Figure (8.13) and Tables (8.2 & 8.3).

It can be seen from Figure (8.13) that in case of upward locked stress ($S_o = +30\ mm$), the piles take higher load, compared to the "flat ground" condition ($S_o = 0$). The upward locked stresses along the pile shaft cause the pile to be stiffer, and as a result of this, it attracts higher load on it. In the case of downward locked stresses, the piles share lesser load and transfer a higher load to the raft.

It can be seen from Figure (8.13) that the actual measurements of the stress distribution along pile shafts are very close to the computed results for the "flat ground condition" ($S_o = 0$). This indicates that there were negligible downward locked stress along the pile shaft prior to the imposed loading. It should also be noted that the downward stress along the pile shaft increases the settlement of the foundation, as discussed in previous chapters. The analysis indicates that the settlement of the foundation is in a good agreement with the $S_o = 0$ condition (Table 8.2). Therefore, it may be inferred from these findings that there was no downward stress locked along the pile shaft prior to the imposed loading.

The upward locked stress may not be evident due to high imposed load on the foundation, which counteracted these locked stresses. These piles were provided only to reduce the settlement of the foundation system, therefore, they were designed to develop their full capacity in frictional as well as in bearing before reaching their structural capacity (Sommer et al, 1991). In this situation, most of the pile capacity was utilised, and as a result of this, the influence of the locked-in stresses on the overall foundation performance
was negligible. In other words, the upward locked stresses will not be able to influence the foundation because of the high imposed loads, which counteract these stresses.

Based on this analysis, it may concluded that the upward locked stresses had negligible effect on the overall foundation performance. It should however be noted that during construction, due to the partial imposed load, these locked stresses must have influenced the load distribution pattern between the raft and the pile, which must have been subdued by the increase in the imposed load.

The settlements at the centre and at the corner of the raft have been analysed and are presented in Table 8.2 for comparison purposes. The predicted and measured load sharing between the piles and the raft is shown in Table 8.3.

### Table 8.2

<table>
<thead>
<tr>
<th>Feature</th>
<th>Settlement at the centre</th>
<th>Settlement at the corner</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Field value</td>
<td>Computed value</td>
</tr>
<tr>
<td>$S_x = 0$</td>
<td>70 mm</td>
<td>77 mm</td>
</tr>
</tbody>
</table>

### Table 8.3

<table>
<thead>
<tr>
<th>Feature</th>
<th>Load shared by piles</th>
<th>Load shared by raft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Field value</td>
<td>Computed value</td>
</tr>
<tr>
<td>$S_x = 0$</td>
<td>53 %</td>
<td>52 %</td>
</tr>
</tbody>
</table>

#### 8.4.2.3.Discussion

Piled raft foundations are quite complex systems due to the “pile-raft-soil” interface conditions, which are highly non-linear in nature. The accuracy of the result is dependent on these limiting values, adopted to incorporate the non-linear conditions. Therefore, it is necessary to pay proper attention to the selection of the correct parameters in carrying out the analysis. In a laboratory situation, as the test is carried out in a highly idealised condition, the soil parameters can be obtained with less uncertainty, whereas, in the field, it is difficult to obtain confident assessments of the various soil parameters. Therefore it is to be expected that the computed results will exhibit better compatibility with the laboratory results compared to the field condition. Engineering judgment is also necessary to incorporate various site conditions in the analysis. The stress history, the geotechnical
conditions, and their implications on the soil properties also need be investigated prior to undertaking the analysis.

In this case, lowering the water table (Stage 3) caused a downdrag force along the pile shaft, exerting a compressive force in the pile, whereas the excavation of the ground (Stage 4) induced an upward stress tending to generate tensile stress in the pile. The stress generated in stage 4 must have been counteracted by the previously induced compressive stress along the pile shaft. Therefore, it had little or no adverse influence on the pile.

In the author’s opinion, the excavation of ground prior to imposing load on the piles could have been a dangerous sequence of work. The excavation of ground after installing the piles may have caused the development of tensile stress along the pile shaft, which could induce cracking in the pile. This may have led to structural damage of the pile. To illustrate the seriousness of this problem, the author would like to draw the attention to a field test carried out by Blight (1984) in South Africa with a group of 33 metres long piles of 1.05 m diameter. These piles were tested for an upward ground movement of 88 mm. The ground movement extended up to a depth of 14 m. The piles were tested without any imposed load on them. The test indicated that the upward ground movement caused very high stress along the pile shaft. As a result of this, the piles cracked and the tensile stress was taken by the reinforcement. Therefore, careful attention should given to the construction sequence to minimise the possibility of inducing undesirable stresses prior to the superstructure loading.

8.5. **CONCLUSION**

The comparisons in this chapter indicate good agreement between the computed and measured values. The compatibility with the laboratory results is very good, whereas in the case of full scale foundation systems, the agreement of results is satisfactory. The tests in a laboratory are carried out in a highly idealised condition, and therefore some control on the soil properties can be maintained. In the case of a full scale foundation system, the soil properties are greatly influenced by various site conditions, which can have a great impact on the accuracy of the result. Nevertheless, provided parameters are appropriately selected, the analysis can still provide an acceptable solution to the foundation problem.

In the case of deep foundation systems, the soil history is very important. The state of consolidation and its related parameters can influence the foundation performance very significantly. The accuracy of foundation performance prediction is also dependent on the limiting condition of stresses at the “raft-soil-pile” interfaces. The selection of
inappropriate values leads to an inaccurate distribution of stresses to various foundation components. Therefore, the information on the load history of the soil system is desirable for a proper analysis.

The adopted methods of analysis appear to be capable of providing reasonable practical predictions of the behaviour of rafts, piles and piled rafts subjected to direct axial loading and ground movements. However, the analysis may further be refined to include other various conditions, the scope of which is discussed in Chapter 9.
(a) Slab Placed on Dry Site

(b) Middle of First Wet Season After Placement

(c) Middle of First Dry Season After Placement

(d) Middle of Second Wet Season After Placement

(e) Long Term Mound Condition

Figure 8.1: Idealised Mound Development
Figure 8.2: Details of Slab

Figure 8.3: Comparison with the Experimental Observation
Figure 8.4: Layout of test pile group

Notes:
- Piles 1, 2 and 3 were instrumented
- Piles 4, 5, 6 and 7 were installed to produce group effect

Figure 8.5: Recorded heave at various depths
Figure 8.6 (i): Load distribution along pile 1

Figure 8.6 (ii): Load distribution along pile 2

Figure 8.6 (iii): Load distribution along pile 3

Figure 8.6: Comparison with the field measurement
Flat Ground Surface, $S_p = 0$

(a) Pile Size
(b) Distribution of ground movement along pile shaft

Figure 8.7: Single Pile Test in Expansive Soils

![Graph showing load vs settlement](image)

Figure 8.8 (i): Load vs Settlements

![Graph showing stress along pile shaft](image)

Figure 8.8 (ii): Stress along pile shafts

Figure 8.8: Comparison of Results
Comparison of Results with Field and the Laboratory Measurements

Figure 8.9 (i): Plan

Figure 8.9 (ii): Section Details

Figure 8.9: Piled Raft Arrangement

Figure 8.10: Comparison of settlement
Chapter : 8

Comparison of Results with Field and the Laboratory Measurements

![Diagram of Soil Profile and Piled Raft Configuration]

**Figure 8.11 (i) : Section**

- **Soil Profile:**
  - **Quaternary sed.**
  - **Tertiary sed.**
  - **Sand, Gravel**
  - **Clay**
  - **Limestone**

- **Exterior Corner Column**
- **Piles:**
  - **Pile A**
  - **Pile B**
  - **Pile C**

**Figure 8.11 (ii) : Plan**

- **Pile diameter = 1.3 m**
- **28 Piles = 26.9 m**
- **20 Piles = 30.9 m**
- **16 Piles = 34.9 m**

**Figure 8.11 : Piled Raft Configuration (Messe Turm, Frankfurt)**
Stages of construction:

1: Pit excavation to 7 m depth
2: Pile construction (upper 7 m empty)
3: Ground water table (GWT) lowered by 7 m
4: Pit excavation to final depth of 14 m

Figure 8.12: Sequence of construction
Chapter: 8

Comparison of Results with Field and the Laboratory Measurements

Figure 8.13 (i): Stress along pile A

Figure 8.13 (ii): Stress along pile B

Figure 8.13 (iii): Stress along pile C

Figure 8.13: Comparison of behaviour
CHAPTER - 9

SUMMARY AND CONCLUSIONS

9.1 INTRODUCTION

In this chapter, the findings of this thesis are summarised in order to present an overview of the foundation behaviour in a reactive soil environment. The response of foundation systems subjected to various cycles the wetting and drying process has been discussed on the basis of the analyses described in previous chapters. The critical aspects of these features have also been discussed and their implications on the structural integrity of the foundation systems have addressed. Finally the scope of work for the further refinement of the method of analysis has been presented and discussed.

The objective of this work has mainly been focussed on developing a method of analysis, which can be used to analyse the behaviour of piled raft foundation systems in a reactive soil environment. The behaviour of the piled raft systems has been thoroughly investigated when subjected to swelling and shrinking soils caused by the seasonal moisture fluctuations. In order to carry out the analysis, a computer program \textit{PRAE,S} (Piled Raft Analysis in Expansive Soils) has been developed using FORTRAN 77. To provide a complete understanding of the foundation behaviour, brief analyses of piles and raft foundations, as independent systems, have also been carried out and presented. Methods have been developed to carry out the analysis of these foundation systems. Computer programs such as \textit{RAE,S} and \textit{PAE,S} have been developed to undertake the raft and the pile foundation analyses, respectively.

Some conclusions on raft and pile foundation systems are not totally original, as those aspects were discussed by other authors previously. Brief attention is drawn towards these conclusions, because they are useful in explaining the various aspects on the piled raft foundation behaviour. The conclusions on the piled raft foundation systems in expansive soils are considered to be totally original and are summarised in greater detail.
9.2 RAFT FOUNDATIONS

The raft was analysed as an elastic plate resting on a semi-infinite and homogeneous soil mass. The contact stresses between the raft and the soil were assumed to be uniform blocks of pressure acting in a vertical direction only. These contact pressures were determined and reapplied to the raft to obtain the moments and displacements at various locations in the raft. The influence of lift-off and yielding of soil was considered in the analysis to incorporate the effect of non-linear conditions.

The conclusions in relation to the raft foundations are summarised below:

- The mound index $m$ has a great influence on the raft behaviour. Higher $m$ value causes lesser bending moment and settlement, whereas a lesser $m$ value causes higher moment and settlement.
- Lifting-off causes a reduction in the differential settlement and an decrease in the maximum moment.
- Yielding of soil under the raft reduces the differential settlement in the raft and causes an increase in total settlement.
- Raft behaviour in swelling and shrinking soils can not be easily generalised because of its non-linear response, excepting a very flexible raft, which may not experience lift-off due to its flexibility.
- The initial state of the ground moisture condition dictates the soil profiles in the following cycles of the seasonal wetting and drying process.
- A raft slab, constructed during the driest period, experiences an upward ground movement due to the swelling of soils in the following wet season and subjects to an "edge heave formation" underneath the raft. In the longer term, the slab is subject to a "central heave formation" due to the intrusion of moisture towards the central portion of the raft.
- A raft slab, constructed in a wet ground condition, experiences a "central heave formation" in the following dry seasons.
- The influence of the edge effect towards the centre of the raft is dependent on the stiffness of the raft.
  - In the case of a rigid raft, the transfer of the edge effect towards the centre of the raft is high due to the rigidity.
  - In contrast, for a flexible raft, the centre remains nearly unaffected.
- Formation of edge heave causes an increase in the differential settlement, whereas the central heave initially causes a reduction in the differential settlement, and then increases
again, but in a reverse fashion i.e. the corner experiences higher settlement compared to the centre.

- Edge heave formation causes an increase in the positive moment, whereas central heave reduces the positive moment.
- A rigid raft experiences lesser differential settlement, compared to a flexible raft. Therefore, to reduce the differential settlement, a raft of higher stiffness may be adopted in the design.
- Stiffened rafts are the alternative solution to restrict the differential settlement. Placement of beams at strategic locations has a great influence in reducing the differential settlement in the raft.
- Further reduction in the differential settlement may be achieved by varying the depth and width of the beams. The number of beams and their respective configuration also have a great influence in reducing the differential settlement of the raft slab.
- Comparison of the computer results with field measurements indicates that the computer model $RAE_5$ may be regarded to be an useful and potentially accurate tool to analyse the behaviour of raft foundation in expansive soils.

### 9.3 PILE FOUNDATIONS

Piles were analysed as a series of uniformly loaded cylindrical elements together with uniformly loaded circular base. The effects of interaction between the shaft elements and the base were considered to formulate the equations. The analysis was undertaken including the effect of the non-linear conditions at the soil-pile interfaces.

The conclusions of the work in relation to piles are summarised below:

- Ground movement induces a force along the pile shaft:
  - a pile installed in a wet ground condition is subject to a downdrag force in the following dry season due to the shrinking effect.
  - conversely, a pile constructed in a dry soil condition experiences upward force in the next wet season due to the swelling movements.
- A purely elastic analysis predicts higher force along the pile shaft compared to a non-linear analysis.
- Slip at the soil-pile interface in the stable zone causes an increase in the displacement. In contrast, a slip in the reactive zone causes less transfer of force to the pile shaft.
A pile under the application of compressive load experiences higher settlement due to the shrinking of the soils. In contrast, the swelling soils cause a reduction in settlement. A lightly loaded pile may experience an upward displacement due to the swelling soils.

Upward displacement of an anchor pile (piles under the application of tensile force) increases due to the swelling soils, and decreases in shrinking soils.

Swelling soils may induce very high tensile stresses in an unloaded pile, which may lead to cracking of concrete piles. Therefore, piles constructed in a dry season should not be left unloaded when subjected to a full cycle of seasonal change.

Additional settlement of a pile due to the downward movement of the soil increases as the applied compressive load increases.

Pile under compressive load, subjected to soil shrinking, experience an increase in settlement and this causes an upward movement of the neutral plane.

Movement of the ground influences the displacement of a pile; however geotechnical failure still takes place at its original ultimate capacity.

Comparison of the theoretical and field results indicates that the computer model PAES may be regarded to be an useful and potentially accurate tool to analyse the behaviour of pile foundations in expansive soils.

### 9.4 PILED RAFT FOUNDATION SYSTEMS

The behaviour of raft and pile foundations have been discussed and presented in previous sections as independent components. In piled raft foundation systems, the raft and a series of piles act in relation to each other, depending on their relative stiffnesses.

In a reactive soil environment, the interactive feature between various foundation components and the soil system makes the whole mechanism more complex in nature. The movement of the ground influences the piles as well as the raft in a different manner. The raft experiences an uneven ground support condition underneath it, whereas the piles are subject to induced forces along the shaft. Due to the structural connection between the raft and the pile, their independent movements are restricted and the effect is distributed to various components of the foundation system, depending on their stiffnesses. The redistribution of stresses is also dependent on the limitations incorporated in the non-linear analysis for various stress values at the soil-pile-raft interfaces. These limiting values may be estimated on the basis of field investigations as well as from laboratory results.
A piled raft foundation constructed in the driest period experiences an upward ground movement due to the swelling of soils in the following wet season. The soil along the edge of the raft experiences a direct exposure of the seasonal variation of moisture and is subject to the swelling process. As a result of this, the following two phenomena take place in the foundation system:

- The piles in the close proximity of the edge experience an upward ground movement along the shaft and
- The edge of the raft is subject to an upward thrust due to the swelling pressure.

The upward thrust causes an increase in the contact stress along the edge of the raft. As a result of this, a reduction in stress takes place at the pile-raft interface. In other words, a reduction in the compressive stress takes place at the pile head.

The effect of upward thrust is dependent on the imposed load on the raft as set out below:

- In the case of a heavily loaded foundation, the contact stress is high under the flat ground condition, and therefore further increase in contact stress due to the ground movement causes the soil to yield and the effect is redistributed to the other raft and pile elements. The redistribution is dependent on the raft stiffness. In this process, the compressive stress at the pile-raft interface also reduces with a increase in the ground movement.
- In case of a lightly loaded or unloaded foundation, the thrust underneath the raft develops a very high tensile stress at the pile head, as there is little or no compressive force to counteract these swelling forces. The thrust causes an upward loading on the raft, and the piles act as support points. As a result of this, the moment pattern may become opposite to that for the normal loading condition.

A high tensile stress at the pile-raft interface may cause damage to the structural connection between the raft and the pile head. In practice, this condition may take place if a piled raft is left unloaded for a full cycle of wetting and drying process. The following wet period may cause the soil to swell and damage the structure.

The occurrence of the following seasonal wetting and drying processes causes further intrusion of moisture towards the centre of the raft. As a result of this the soil in the central part of the raft swells and pushes the raft upward. The upward thrust is dependent on the imposed load on the raft, as discussed above. This causes further reduction in the compressive
stress at the central pile heads. The long term seasonal moisture fluctuations may cause a central mound formation underneath the raft, but the soil along the edge of the raft will still be subjected directly to the seasonal wetting and the drying effects. These actions are highly non-linear in nature, and therefore the actual magnitude of various movements and stresses are dependent on both soil and foundation parameters.

The distortion caused by the mound formation is dependent on the imposed load on the raft as well as on the restriction offered by piles. Higher imposed load restricts the upward ground movement and as a result of this, an increase in the contact stress takes place underneath the raft. This may cause yielding of soil and redistribution of stresses to other foundation components. In a similar fashion, longer piles cause a higher anchoring effect compared to shorter piles and restrict the upward ground movement underneath the raft. This process causes a reduction in the compressive stress at the pile-raft interface and in extreme cases, a tensile stress may develop at the interface. In this situation, the whole pile may be subject to tension. This characteristic restricts the upward movement of the ground and causes higher distortion to the mound formation.

In a ground movement situation, the stiffness of the raft plays a very important role in redistributing the stress to the various foundation components. In the case of a rigid raft, the edge effect is transferred to the central portion of the raft, and the whole foundation system acts as a unit. This feature also restricts the movement of the soil surface and causes higher distortion of the mound. However, a flexible raft is unable to transfer the effect to other foundation components and the mound distortion is less. In this case, the influence of the ground movement is localised and the effect of the movement is transferred to the piles in close proximity to the edge of the raft. Therefore, these piles are most affected by ground movement.

In the case of a foundation system constructed in the wettest period, the soil along the edge of the raft experiences a reduction in volume in the following dry period. This forms a "central heave" underneath the raft and induces a downdrag force along the pile shaft as well as a reduction in the contact stress towards the edge of the raft. The reduction in the contact stress causes a transfer of load to the piles as well as to other portions of the raft. The transfer is dependent on the raft stiffness. The downdrag force along the pile shaft increases the settlement of the pile. The increase in the settlement tends to reduce the compressive stress at the pile-raft interface, but the reduction in the contact stress causes a transfer of load to the pile head. A rigid raft transfers more of the influence of the ground movement towards other
foundation components, whereas a flexible raft causes a localised effect and transfers the effect to the piles closer to the ground movements.

The redistribution of stress is not only dependent on the raft stiffness but also on the number and the location of piles. In the case of central heave formation, the piles towards the centre of the raft are less influenced by the ground movement, and therefore they are in effect stiffer than the edge piles. Therefore, in the case of a rigid raft, such piles attract more load in the process of redistribution.

The total and the differential settlements are significantly influenced by the movement of the ground. A piled raft foundation constructed in a flat ground condition usually experiences higher settlement at the centre in comparison to the corner or the edge of the raft. The central mound formation underneath the raft causes an increase in the corner settlement with respect to the central raft. This reduces the difference in settlement in between the corner and the centre of the raft. This reduction may equalise the central and the corner settlements, depending on the length of the piles. Shorter piles experience higher settlement under the effect of same ground movement, compared to a longer piles. Therefore, in case of shorter piles, the increase in the corner settlement is higher and the reduction in the difference in settlement is rapid. In this situation, there may a stage when the difference in settlements becomes zero. A further increase in the ground movement increases the differential settlement, but in the reverse fashion i.e. the corner experiences higher settlement compared to the centre. This condition should be addressed in the design phase as it may influence the foundation system as well as the superstructure. In the case of longer piles, the settlement due to the ground movement will cease beyond a certain ground movement, because of the longer portion of the shaft in the stable zone. If slip at the soil - pile interface takes place, any further increase in the ground movement will not cause increase in the settlement.

The magnitude and the distribution of moments are influenced very significantly by the ground movement. In the case of central heave formation, the maximum positive moment reduces and the maximum negative moment increases with an increase in the ground movement. In contrast for edge heave formation, the maximum positive moment increases and the maximum negative moment decreases with increasing ground movement.

Generally, in the case of a semi-rigid raft condition, the moment is negative at the corner pile locations in a flat ground condition. In swelling soils, due to the upward movement of the ground, this negative moment reduces and becomes positive with increasing ground
movement. In a shrinking soil condition, this moment increases with the increasing ground movement. These aspects should be considered carefully in the design phase.

In a shrinking soil condition, the downward moving soil surface induces downdrag force along the pile shaft until slip occurs at the soil - pile interface. Beyond this point, the pile shaft is no longer influenced by the ground movement, but the change in the soil surface profile underneath the raft due to the increase in the ground movement imposes additional load on the piles.

The above foundation behaviour is highly non-linear in nature, therefore it is difficult to obtain a generalised solution for various conditions. Although the overall behaviour and the understanding of the piled raft systems in expansive soils remain the same for all cases, but to estimate actual values it is imperative to undertake a thorough non-linear soil-structure interactive analysis, with the limiting values being assumed from a proper evaluation of the site conditions.

Good agreement was found between the computed results and the results obtained from tests in the field and the laboratory. Based on these comparisons, the adopted method of analysis is considered to be adequate to analyse these foundation systems. Some further refinements which may be incorporated in future research are discussed and presented in section 9.5.

The conclusions of the overall work on piled raft foundation systems may briefly be outlined as set out below:

- Swelling soils induce an upward force along the pile shaft as well as an upward thrust under the raft, which reduces the compressive stress at the pile-raft interface. Conversely, a shrinking soil imposes a downward force along the shaft and causes a reduction in the contact stress underneath the raft, which increases the compressive stress at the pile head.

- Change in the stress condition due to the ground movement causes a redistribution of stress between various foundation components, which influences the load distribution between the piles and the raft very significantly. Redistribution of stresses in various components is dependent on the following parameters:
  - Raft stiffness: A rigid raft causes a higher transfer of the ground movement effect to various raft locations as well as to different piles, whereas a flexible raft is less able to transfer the load due to its flexibility.
reduce the movement locally around it. Due to this effect the inner piles will be subject to a lesser ground movement, compared to the external piles. Greater numbers of piles will significantly influence the soil profile. Therefore, there is scope to investigate the movement of the soil surface underneath a piled raft system. A comparative field study of the ground movements under a “raft only” and a piled raft system may indicate the influence of piles in ground movement formation.

- Versatility of the method

The present method of analysis may be modified for the various imposed load, such as lateral loads and moments. These conditions shall make the method more versatile for various applications.

- Lateral load and soil separation at the upper segment of the pile shaft

The variation in the moisture content causes the soil to change in volume in vertical as well as horizontal direction. In the case of foundations constructed in a dry period, the volume of soil will increase with the increasing moisture content and as a result of this, the piles may experience lateral thrust in the upper segment of the shaft. In contrast, a transition from wet to a dry period will cause reduction in the soil volume, causing a separation at the soil-pile interface in the zone of maximum ground movement. The influence of these affects may also be taken into account in the analysis. The inclusion of lateral load, separation and moment loading on the piled raft is desirable for more general and versatile analysis method to be developed.

- Layered soil profile

The present method of analysis has been developed for homogeneous and infinite by deep soil mass and has been mainly focused on the movement of the ground, which takes place in the zone known as “depth of seasonal moisture change”. An equivalent approach can be adopted to analyse the foundation in layered soil profiles. However, future enhancements in the method of analysis should be undertaken to incorporate the influence of layered soil profiles.

- Laboratory and field tests

A series of laboratory and field tests is necessary to understand the piled raft behaviour in a reactive soil environment. The pattern of the ground movement underneath the raft and along the pile shaft should be investigated thoroughly, because these phenomena influence
the foundation behaviour very significantly. The movement of the ground at various pile locations underneath a raft, for various raft dimensions, may be observed. The settlements, contact stresses, stresses along the pile shaft and the load sharing between the pile and the raft may be studied under various ground movement conditions due to the swelling and shrinking of the soil. Centrifuge tests may also be useful in investigating these phenomena.
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