HYDRAULIC GRADIENT SIMILITUDE METHOD FOR GEOTECHNICAL MODELLING OF PILE GROUP SUBJECTED TO STATIC LATERAL LOAD

by

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ABSTRACT

The thesis considers the problem of vertical pile group response to lateral static loads. There are various solutions available for single pile response to lateral loads. These solutions have been verified against a large database obtained from field experiments and model experiments. For pile groups very few theories have been proposed and due to the comparatively smaller database available it is not possible to develop and test a sound theory for predicting the pile group response to lateral loads.

This thesis is aimed at obtaining a database for response of pile groups comprising of two piles subjected to lateral static loads. Tests were carried out in the Hydraulic Gradient Similitude Device in order to bring the stress state in the soil to the field stress level. For testing purposes three cases were considered, single pile, single pile adjacent to a loaded pile, and a pile group of two piles.

The single pile test results showed that the test results were repeatable and reliable. The tests on a single pile adjacent to a loaded pile showed that the position of the pile with respect to the loaded pile has a strong influence on the response of the pile. The unloaded pile in the direction of the loading and in front of the loaded pile is most effected. At a spacing of 2 diameters bending moment developed is up to a

maximum of 20 percent of bending moment developed in single pile. This percentage decreases rapidly with increasing spacing. If the unloaded pile is located behind the loaded pile or is at 90° to the loading direction, it essentially picks up very little load from the loaded pile. The installation of two piles densifies the soil in between.

In case of pile groups, the load sharing among the piles is based on the pile location and the interaction effect is not reciprocal. The lead pile, i.e. the pile in the direction of load, shares maximum load with trail pile sharing smaller load.

TABLE OF CONTENTS

ABSTRACT ii

AUTHORIZATION iv

TABLE OF CONTENTS v

LIST OF TABLES ix

LIST OF FIGURES x

ACKNOWLEDGEMENT xiv

CHAPTER 1 : INTRODUCTION 1

1.1 INTRODUCTION 1

1.2 SCOPE OF STUDY 3

1.3 ORGANIZATION OF THESIS 4

CHAPTER 2 : LITERATURE REVIEW 6

2.1 INTRODUCTION 6

2.2 REVIEW OF THE SINGLE PILE RESPONSE TO LATERAL LOADS 6

2.2.1 THEORETICAL STUDIES 6

2.2.1.1 The Elastic Boundary Element Approach 8

2.2.1.2 The Modulus Of Subgrade Reaction Approach 9

2.2.1.3 FINITE ELEMENT APPROACH 16

2.2.2 FIELD TESTING 16

2.2.3 MODEL STUDIES 19

2.3 REVIEW OF THE PILE GROUP RESPONSE TO THE LATERAL LOADS 21

2.3.1 FIELD TESTING 22

2.3.2 MODEL TESTING 23

2.3.3 ANALYTICAL STUDY 25

2.3.3.1 FINITE ELEMENT ANALYSIS 29

2.4 SUMMARY 32

CHAPTER 3 : HYDRAULIC GRADIENT SIMILITUDE PRINCIPLE . . . 35

3.1 INTRODUCTION 35

3.2 Hydraulic Gradient Similitude Principle 36

CHAPTER 4 : MODEL SOIL AND PILE PROPERTIES 40

4.1 MODEL SOIL 40

4.2 MODEL PILE PROPERTIES 41

CHAPTER 5 : HYDRAULIC GRADIENT SIMILITUDE TESTING DEVICE 47

5.1 INTRODUCTION 47

5.2 Hydraulic Gradient Similitude Testing Device . . 47

5.2.1 SAND CONTAINER AND AIR PRESSURE CHAMBER . 50

5.2.2 WATER SUPPLY AND CIRCULATION SYSTEM . . 52

5.2.3 PILE HEAD LOADING AND MEASURING SYSTEM . 52

5.2.4 DATA ACQUISITION SYSTEM 55

5.2.5 DATA REDUCTION 57

CHAPTER 6 : TEST PROCEDURE 61

6.1 Test Procedure 61

6.2 RECONSTITUTION OF SAND DEPOSIT 61

6.3 PILE INSTALLATION 62
6.4 SOIL LOADING AND PILE LOADING 63
6.5 PILE HEAD LOADING 64

CHAPTER 7 : RESULTS AND DISCUSSION 66
7.1 INTRODUCTION 66
7.2 Testing Series 66
7.2.1 Repeatability of the test results 67
7.2.2 Series I (S1)
Single Pile testing results 68
7.2.2.1 LOAD DISPLACEMENT RESPONSE 71
7.2.2.2 P-$(y-y_0)$ CURVE 71
7.2.2.3 BENDING MOMENT AND SHEAR FORCE PROFILE 73
7.2.3 Series II (S2)
Pile Group Of Two Piles (One Pile Loaded) 74
7.2.3.1 Load Displacement Response 74
7.2.3.2 Bending moment and shear force distribution 79
7.2.3.3 P-$(y-y_0)$ CURVE 91
7.2.3.4 CASE 2 (ϕ = 90°) 94
7.2.3.5 CASE 3 (ϕ = 180°) 94
7.2.4 SERIES III (S3) 97
7.2.4.1 Load Displacement 97
7.2.4.2 P-y CURVES 101
7.2.4.3 Bending moment Profile 103

CHAPTER 8 : PREDICTION OF THE PILE GROUP RESPONSE 114

8.1 INTRODUCTION 114

8.2 THE FREE FIELD CONCEPT AND ITS APPLICATION . . . 114

8.3 PREDICTION OF PILE RESPONSE 116

8.3.1 COMPARISON OF BENDING MOMENT 117

8.3.2 LOAD DISPLACEMENT RESPONSE 119

8.4 SUMMARY 122

CHAPTER 9 : SUMMARY AND CONCLUSION 125

REFERENCES 128

LIST OF TABLES

TABLE 3.1 SCALING RELATIONS FOR CENTRIFUGE AND HYDRAULIC GRADIENT TESTS 39

TABLE 4.1 HYPERBOLIC SOIL PARAMETERS FROM DRAINED COMPRESSION TRIAXIAL TESTS 42

TABLE 4.2 PHYSICAL PROPERTIES OF MODEL PILES 43

LIST OF FIGURES

FIGURE 2.1	CONCEPT OF WINKLER SPRINGS	11
FIGURE 2.2	n_{hi} VS RELATIVE DENSITY, AFTER MURCHISON AND O'NEILL(1984)	15
FIGURE 2.3	FACTORS FOR P_u, AFTER MURCHISON AND O'NEILL (1984)	15
FIGURE 2.4	CONCEPT OF WINKLER SPRINGS FOR TWO PILES (TROCHANIS ET AL, 1991)	34
FIGURE 3.1	HYDRAULIC GRADIENT SIMILITUDE PRINCIPLE	39
FIGURE 4.1	GRAIN SIZE DISTRIBUTION OF THE FINE OTTAWA SAND (YAN LI,1991)	45
FIGURE 4.2	VARIATION OF PERMEABILITY VS VOID RATIO (YAN LI, 1991)	45
FIGURE 4.3	PILE INSTRUMENTATION	46
FIGURE 5.1	HYDRAULIC GRADIENT SIMILITUDE DEVICE	49
FIGURE 5.2	CONTAINER LID - PLAN AND VIEW	53
FIGURE 5.3	WATER FLOW SYSTEM IN UBC-HGT DEVICE	54
FIGURE 5.4	LOAD CELL	56
FIGURE 5.5	PILE HEAD DEFLECTION MEASUREMENT	58
FIGURE 7.1	LOAD DISPLACEMENT BEHAVIOUR - COMPARISON	69
FIGURE 7.2	$P-(y-y_o)$ CURVE - COMPARISON	69
FIGURE 7.3	BENDING MOMENT PROFILE - COMPARISON	70
FIGURE 7.4	LOAD-DISPLACEMENT CURVE - SINGLE PILE	72
FIGURE 7.5	$P-(y-y_o)$ CURVES - SINGLE PILE	72

FIGURE 7.6	BENDING MOMENT PROFILE - SINGLE PILE	75
FIGURE 7.7	DISPLACEMENT PROFILE - SINGLE PILE	75
	SERIES 2	
FIGURE 7.8	LOAD-DISPLACEMENT CURVES - ADJACENT PILE . .	77
FIGURE 7.9	LOAD DISPLACEMENT CURVES - LOADED PILE . . .	77
FIGURE 7.10	LOAD-DISPLACEMENT CURVES (S/D = 2)	78
FIGURE 7.11	LOAD-DISPLACEMENT CURVES (S/D = 4)	78
FIGURE 7.12	LOAD-DISPLACEMENT CURVES (S/D = 6)	80
FIGURE 7.13	LOAD-DISPLACEMENT CURVES (LOADED PILE) . .	80
FIGURE 7.14	COMPARISON OF INTERACTION COEFFICIENT . . .	81
FIGURE 7.15	BENDING MOMENT PROFILE- PILE 2 (S/D = 2) . .	83
FIGURE 7.16	BENDING MOMENT PROFILE- PILE 2 (S/D = 4) . .	83
FIGURE 7.17	BENDING MOMENT PROFILE- PILE 2 (S/D = 6) . .	84
FIGURE 7.18	BENDING MOMENT PROFILE- PILE 1 (S/D = 2) . .	84
FIGURE 7.19	BENDING MOMENT PROFILE- PILE 1 (S/D = 4) . .	86
FIGURE 7.20	BENDING MOMENT PROFILE- PILE 1 (S/D = 6) . .	86
FIGURE 7.21	COMPARISON OF BENDING MOMENT PROFILES(S/D=2)	87
FIGURE 7.22	COMPARISON OF BENDING MOMENT PROFILES(S/D=4)	87
FIGURE 7.23	COMPARISON OF BENDING MOMENT PROFILES(S/D=6)	89
FIGURE 7.24	DEFLECTION PROFILE- PILE 2 (S/D=2)	89
FIGURE 7.25	DEFLECTION PROFILE- PILE 2 (S/D=4)	90
FIGURE 7.26	DEFLECTION PROFILE- PILE 2 (S/D=6)	90
FIGURE 7.27	DEFLECTION PROFILE- PILE 1 (S/D=2)	92
FIGURE 7.28	DEFLECTION PROFILE- PILE 1 (S/D=4)	92
FIGURE 7.29	DEFLECTION PROFILE- PILE 1 (S/D=6)	93
FIGURE 7.30	P - y CURVES- PILE 1 (S/D=2)	95

FIGURE 7.31 P - y CURVES- PILE 1 (S/D=4) 95

FIGURE 7.32 P - y CURVES- PILE 1 (S/D=6) 96

FIGURE 7.33 LOAD-DEFLECTION CURVE-PILE 1 : LOADING ANGLE 90° 96

FIGURE 7.34 BENDING MOMENT PROFILE-PILE 1 : LOADING ANGLE 90° 98

FIGURE 7.35 BENDING MOMENT COMPARISON : LOADING ANGLE 180° 98

SERIES 3

FIGURE 7.36 LOAD-DISPLACEMENT CURVE (S/D = 2) 99

FIGURE 7.37 LOAD-DISPLACEMENT CURVE (S/D = 4) 99

FIGURE 7.38 P-y CURVE (S/D = 2) 102

FIGURE 7.39 P-y CURVE (S/D = 4) 102

FIGURE 7.40 BENDING MOMENT PROFILE (S/D = 2) 104

FIGURE 7.41 COMPARISON OF BENDING MOMENT PROFILE 104

FIGURE 7.42 DEFLECTION PROFILE (S/D = 2) 106

FIGURE 7.43 BENDING MOMENT PROFILE (S/D = 4) 106

FIGURE 7.44 COMPARISON OF BENDING MOMENT PROFILE 108

FIGURE 7.45 DEFLECTION PROFILE (S/D = 4) 109

FIGURE 7.46 COMPARISON OF BENDING MOMENT (S/D = 2) . . . 109

FIGURE 7.47 COMPARISON OF BENDING MOMENT (S/D = 4) . . . 111

FIGURE 8.1 COMPARISON OF BENDING MOMENT FROM EXPERIMENT AND LATPILE FOR SINGLE PILE 118

FIGURE 8.2 COMPARISON OF BENDING MOMENT FROM EXPERIMENT AND LATPILE- PILE 2 (S/D = 2) 120

FIGURE 8.3 COMPARISON OF BENDING MOMENT FROM EXPERIMENT

AND LATPILE- PILE 2 (S/D = 2) 120

FIGURE 8.4 COMPARISON OF BENDING MOMENT FROM EXPERIMENT AND LATPILE- PILE 2 (S/D = 4) 121

FIGURE 8.5 COMPARISON OF BENDING MOMENT FROM EXPERIMENT AND LATPILE- PILE 2 (S/D = 4) 121

FIGURE 8.6 COMPARISON OF BENDING MOMENT FROM EXPERIMENT AND LATPILE- PILE 2 (S/D = 6) 123

FIGURE 8.7 COMPARISON OF BENDING MOMENT FROM EXPERIMENT AND LATPILE- TRAILING PILE (S/D = 2) 124

FIGURE 8.8 COMPARISON OF BENDING MOMENT FROM EXPERIMENT AND LATPILE- TRAILING PILE (S/D = 4) 124

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CHAPTER 1 : INTRODUCTION

1.1 INTRODUCTION

In foundation engineering practice, piles are frequently used to resist large horizontal loads from the superstructure. In the past, pile foundations were designed and constructed based on experience. In recent years, due to research conducted in this area, a better understanding of pile response to the loads has been achieved. This has led to more efficient, economical designs and safety.

Through field and model studies of piles, sound theories have been developed for single piles loaded vertically and horizontally. In addition, reliable solutions have also been developed for vertically loaded pile groups. However, there is a lack of reliable theory and solution to predict the response of pile groups to lateral loads and this thesis is directed towards this problem.

To develop a reliable theory which can predict the response of pile groups to lateral loads, a large data base of pile group response is required. So far, various researchers have proposed empirical solutions based on a very limited field database.

The database can be generated by conducting either field tests or laboratory model tests. Field tests give the actual performance of the pile group, but are very costly. In model tests, the size of the sample and the test piles has a considerable effect on the response of the pile group. The model pile group can give similar response as the field test only if the sample is at the same stress level as the field soil. In recent years innovative techniques to conduct small model tests at field stress level have been introduced.

One of the methods to increase the sample stress level is the Centrifuge model test. In this procedure the required stress level is obtained by rotating the sample at a given centripetal acceleration to achieve the field stress level in the model.

Another method used to increase the sample stress level is the Hydraulic Gradient method. The technique was first developed by Zelikson(1969) to increase the stress level of samples before conducting Centrifuge model test, but later was developed into a separate testing method. The Hydraulic Gradient Similitude method, as it is known, uses a principle similar to the Centrifuge Model test, i.e. it increases the stress level of the sample by increasing the body forces on the soil particles. The difference is in the method used to increase the body forces. Whereas the Centrifuge model test uses a centripetal acceleration to increase the body forces, the Hydraulic Gradient Similitude Method (HGSM) uses hydraulic gradient to increase the body forces. Using this technique Yan and Byrne (1991a) developed a Hydraulic Gradient Similitude Testing Device (HGST). In this thesis, the HGS method is used to study the interaction effect between the laterally loaded piles in a group of vertical piles. The HGS model tests were conducted to study the response of a pile group of two piles subjected to lateral loads. One of the piles was instrumented to measure the bending moments along the pile length. The load on the piles and pile head deflections were measured for each pile during the test. A computer software program was used to record and store the data. The measured results from the tests are compared with the predictions from analyses.

1.2 SCOPE OF STUDY

The major concerns for the laterally loaded pile are the bending moment, shear force and deflection of the pile. In recent years, the patterns of the bending moment and shear force developed in the single pile and the deflections of the single pile have been observed by various researchers. In this thesis a study of a pile group comprised of two piles subjected to lateral loads is conducted. The scope of the study of the thesis is as follows

- Study the bending moment, shear force and deflection profiles of the single pile subjected to lateral load.

- Study the effect of the presence of an adjacent unloaded pile on the lateral response of a pile.
- Study the bending moment, shear force and deflection profiles of piles in a pile group comprised of two piles subjected to lateral loading.

1.3 ORGANIZATION OF THESIS

The thesis is divided in to seven chapters as follows

- CHAPTER 1 :- INTRODUCTION. In this chapter, the thesis objective and the testing principle is given.
- CHAPTER 2 :- LITERATURE REVIEW. In this chapter a literature review of the single pile subjected to lateral static loads, and pile groups subjected to lateral and vertical loads is given. The current theoretical methods and previous field and model tests are critically reviewed for this purpose.
- CHAPTER 3 :- HYDRAULIC GRADIENT SIMILITUDE PRINCIPLE. The hydraulic gradient similitude (HGS) testing principle used in this thesis is explained in this chapter.
- CHAPTER 4 :- MODEL SOIL AND PILE PROPERTIES. Properties of the soil and pile used in the model are described in this chapter. These properties can be used to predict the pile response based on the existing solutions, e.g. elastic solution by Poulos

(1971).

- CHAPTER 5 :- HYDRAULIC GRADIENT SIMILITUDE TESTING DEVICE. The hydraulic gradient similitude testing device based on the HGS principle explained in the chapter 3 is described in detail in this chapter.
- CHAPTER 6 :- HYDRAULIC GRADIENT SIMILITUDE TESTING PROCEDURE. The hydraulic gradient similitude testing procedure for testing laterally loaded piles is described in detail in this chapter.
- CHAPTER 7 :- TEST RESULTS AND DISCUSSION. The results of the lateral load tests on the single pile and pile group are reported in this chapter. The results are discussed as to the effect of various parameters on the behaviour of the pile group and the differences in the behaviour of pile group and single pile.
- CHAPTER 8 :- PREDICTION OF RESULTS USING LATPILE PROGRAM. The response of single piles and pile groups are predicted using the LATPILE program. The theory used for the pile group analysis is explained before the presentation of the results.
- CHAPTER 9 :- SUMMARY AND CONCLUSIONS. In this chapter, the test results and conclusions are summarized.

CHAPTER 2 : LITERATURE REVIEW

2.1 INTRODUCTION

Pile supported foundations are used for a large number of structures. Although they have been used for a long time, they are generally analyzed and designed using empirical methods. With an increase in the use of pile foundations, more and more researchers are aiming their research to find a more reliable and economical design method.

Although there are some empirical solutions available which can be used to predict static response of laterally loaded piles, it is very difficult to predict dynamic response of the pile foundations due to the complexity of the problem. Furthermore, prediction of pile group response is more difficult due to various factors involved, like soil-pile interaction, pile-cap-pile interaction. In this chapter, the methods used to analyze laterally loaded single pile are reviewed, followed by a review of methods used for the analysis of pile group. The review is limited to the response of vertical piles subjected to lateral loads.

2.2 REVIEW OF THE SINGLE PILE RESPONSE TO LATERAL LOADS

2.2.1 THEORETICAL STUDIES

When a single pile is loaded laterally, the load is resisted by the soil surrounding the pile as well as by the pile itself in bending and shear. In the case of pile groups, load transfer by pile-soil-pile interaction also occurs. The main concerns for laterally loaded piles are

- Pile deflection
- Maximum Bending moment
- Maximum Shear force

These can be computed by either a field test, model test or by using available solutions. The lateral response of the pile foundation can be computed using one of the following methods

1. Elastic boundary element approach
2. Winkler spring approach or modulus of subgrade reaction approach
3. Finite element approach

The elastic boundary element approach uses an elastic continuous soil model and elastic pile model, whereas the subgrade reaction approach considers that the soil response can be simulated by compliance springs. These springs can be modelled as linear or nonlinear to simulate nonlinear response. In the finite element approach, the soil and pile are divided into small elements (soil elements and beam elements) and the behaviour of these elements can be linear or nonlinear. The only disadvantage of the finite element approach is that it is time consuming and costly.

These methods are discussed in detail in the following paragraphs.

2.2.1.1 The Elastic Boundary Element Approach

The solution based on elastic boundary element approach was developed by Poulos (1971). The elastic boundary element approach is based on linear elastic theory for the soil medium and uses Mindlin's solutions for the soil displacements due to a point load within an elastic isotropic homogeneous halfspace. The pile is simulated by using a vertical column with the equivalent stiffness, EI, of the pile. Compatibility of soil and pile displacements is forced at discrete points along the pile length. The main parameters used for this analysis are the elastic parameters for soil, the Young's modulus, the Poisson's ratio, and the stiffness of the pile given by the term EI. The results are available in the form of Design charts and have been widely used by researchers and practising engineers.

The advantage of this method is that it considers the soil as a continuum, which makes it easy to analyze the pile group behaviour. But, at the same time, it should be noted that the elastic continuum solution is strictly applicable only to small strain levels. Various other factors affecting the linear behaviour of the soil, such as soil yielding, finite depth of soil layer,non-homogeneity, etc can be taken into account by introducing a correction factor to the elastic solution.

Since, the pile-soil behaviour is non-linear, it is difficult to select the appropriate Young's modulus and Poisson's ratio (Poulos 1980,1987; Poulos et al 1992).

2.2.1.2 The Modulus Of Subgrade Reaction Approach

In this approach, the pile is treated as a linear elastic beam-column and the surrounding soil is replaced by a bed of uncoupled Winkler springs. The model is illustrated in figure 2.1 and represents load-deflection properties of soil-pile system under lateral loadings. The governing equation for this type of model is based on the classical Hetneyi's solution for a beam column on an elastic foundation. The resulting governing equation is given as

$$E\,I\frac{d^4y}{dz^4} + P_z\frac{d^2y}{dz^2} - P = 0 \qquad \text{....... eq. (2.1)}$$

P_z = axial load on the pile

y = lateral deflection of the pile at depth z along the pile length

z = depth below the ground surface of the point under consideration

P = soil reaction per unit length and

EI = the flexural rigidity of the pile

In the above model, the Winkler's springs can be either linear or nonlinear. Their force deflection response is usually termed as P-y curves and is specified at points along the pile length. This method provides a versatile analytical tool to incorporate both the soil non-homogeneity and nonlinear response. However, this approach does not take into account the soil continuity and hence can not be readily applicable to pile group analysis.

The soil reaction, P, in equation 2.1 is related to the lateral deflection, y, linearly as follows,

$P = K_h\, y$ .... eq. (2.2)

Where K_h is the horizontal subgrade reaction modulus

The coefficient of horizontal subgrade reaction, $\mathbf{k_h}$, used in soil mechanics is defined as

$p = \mathbf{k_h}\, y$ .... eq. (2.3)

where p is the soil pressure,

$\mathbf{k_h}$ is related to the horizontal subgrade modulus as follows

$\mathbf{k_h} = K_h / D$ .... eq. (2.4)

where D is the pile diameter.

The coefficient of horizontal subgrade reaction given by Terzaghi(1955) varies linearly with depth. In practice, the variation of $\mathbf{k_h}$ with depth can be nonlinear. Various closed form solutions for linear as well as parabolic distributions are available (Scott 1981; Poulos 1982; Franklin and Scott 1979;).

Various factors affecting the coefficient of horizontal subgrade reaction have been reported. These factors arise because the Winkler spring system is uncoupled and ignores the soil continuity and hence is not a fundamental approach and needs calibration with more fundamental analysis and field

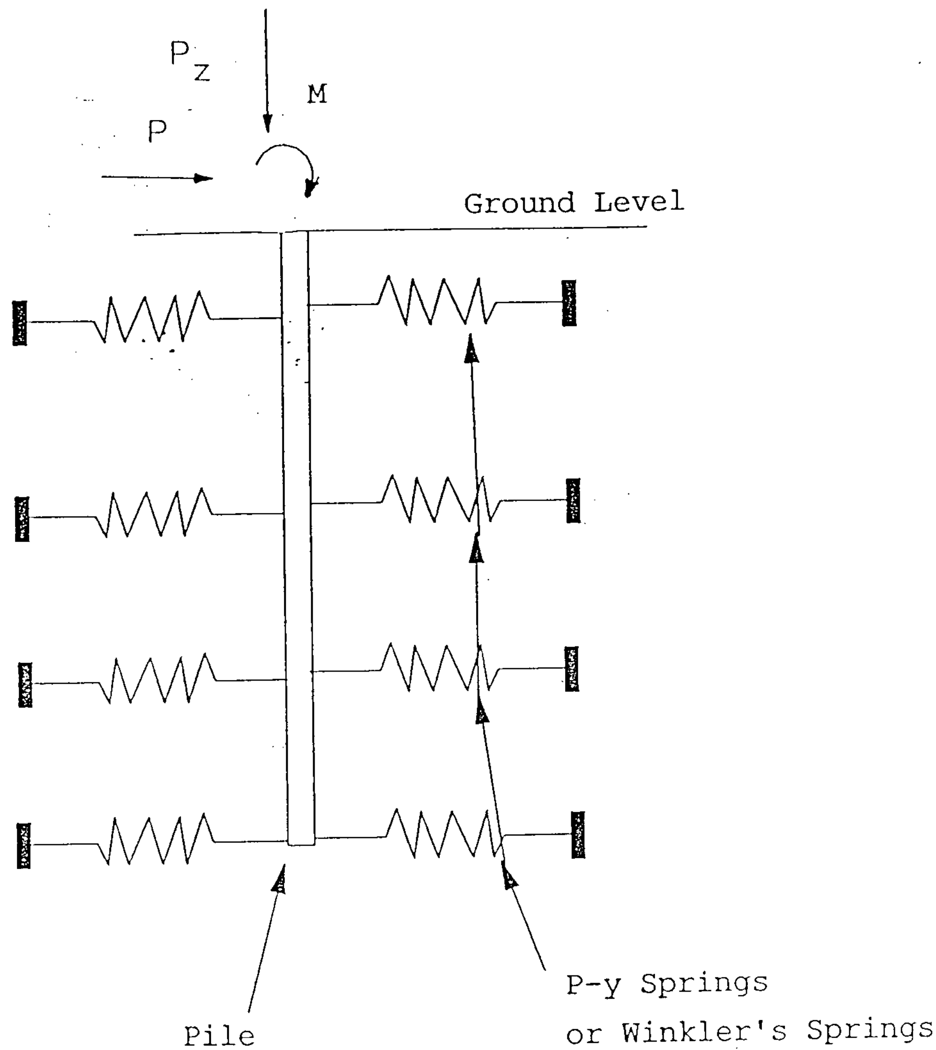


Fig. 2.1 Concept Of Winkler's Springs

experience. However, it has the advantage that both non-homogeneous and nonlinear soil effects can be simply incorporated.

Various methods to determine the P-y curves have been proposed. The method proposed by Reese et al(1974) is empirical and has been adopted by the American Petroleum Institute (API) design code. This procedure was based on the back-analysis of the full scale instrumented pile load tests on Mustang Island, Texas (Cox et al 1974). The P-y curves are constructed at desired depths with the initial slope of curves defined as

$$K_{hi} = n_{hi} \cdot z \qquad \ldots\ldots\ \text{eq. (2.5)}$$

where z is the depth of P-y curve,

K_{hi} is the subgrade reaction modulus, and

n_{hi} is the coefficient of subgrade reaction modulus.

Reese (1974) suggested that the values of n_{hi} to be used should be 2.5 to 4 times larger than those suggested by Terzaghi(1955). It should be taken into consideration that the Terzaghi values are at the working load values while the Reese et al values are the initial values. Jamolkowski and Garassino (1977) suggested the following expression for the coefficient of subgrade reaction modulus

$$n_{hi} = 19\ \gamma_w \cdot (\ D_r\ )^{1.19} \qquad \ldots\ldots\ \text{eq. (2.6)}$$

where γ_w is the unit weight of water,

D_r is the relative density of submerged soils.

Murchison and O'Neill(1983) gave the n_{hi} values for the dry sands in terms of relative density and friction angle (Figure 2.2). Yan and Byrne(1991a) have shown that the initial slope of the P-y curves can be represented by the maximum Young's modulus of the soil, E_{max}, obtained from downhole or crosshole seismic tests.

The ultimate soil resistance, P_u, in the Reese et al P-y curve was determined from the lesser of the following,

$$P_u = \gamma z \, [\, D(K_p - K_a) + z\, K_p \tan\phi \tan\beta \,] \quad \ldots\ldots\ \text{eq. } (2.7)$$

$$P_u = \gamma D z \, [\, K_p^3 + 2\, K_0\, K_p^2 \tan\phi + \tan\phi - K_a.] \quad \ldots\ldots\ \text{eq. } (2.8)$$

where P_u is the ultimate soil resistance force per unit depth,

z is the depth,

γ is the effective unit weight of soil (submerged or total),

K_a, K_p are the Rankine active and passive coefficients respectively,

K_0 is the at rest earth pressure coefficient,

ϕ is the angle of internal friction, and

$$\beta = 45^\circ + \phi/2 \,. \quad \ldots\ldots\ \text{eq. } (2.9)$$

A number of studies indicate that P_u for cohesionless soil is not well defined (Kubo, 1966; Yoshida and Yoshinaka 1972; Scott 1981; Ting et al 1987). Despite these findings, the concept of P_u is still used to define the P-y curves. Bogard and Matlock (1980) proposed the following equations for the ultimate soil pressures in sand

$$P_u = (\, C_1\, z + C_2\, D\,)\, \gamma\, z \quad \ldots\ldots\ \text{eq. } (2.10)$$

$$P_u = C_3\, D\, \gamma\, z \quad \ldots\ldots\ \text{eq. } (2.11)$$

The parameters C_1, C_2, C_3 are given in Figure 2.3.

Murchison and O'Neill(1984) gave the following equation for determining the soil resistance P at any deflection value, y,

$$P = \eta A P_u \tanh\left[\left(\frac{n_h z}{A \eta P_u}\right) y\right] \qquad \text{....... eq. (2.12)}$$

in which P_u is taken as lesser value of eqn.s 2.10 and 2.11.

The empirical factor A is given as

A = 0.9 for cyclic loading and .... eq. (2.13)

= 3 - 0.8 z/D ≥ 0.9 for static loading .... eq. (2.14)

The 1987 API code has adopted this equation to describe the P-y curves and η is a factor used to describe pile shape effect.

During their testing, Yan and Byrne (1990) found that the pile head response and bending moment are significantly affected by the relative soil-pile stiffness, pile diameter, pile head eccentricity and pile head fixity. It was also found that the P-y curves are not affected by the pile diameter, pile head eccentricity and pile head fixity but significantly affected by the relative soil-pile stiffness due to the soil stress levels. It was found that for the monotonic loading the stress level dependency of the P-y curves can be reasonably normalized by the Young's moduli of the soils and the pile diameter for P-y curves at depths below 1 pile diameter, using the hyperbolic stress-strain relationship for soils.

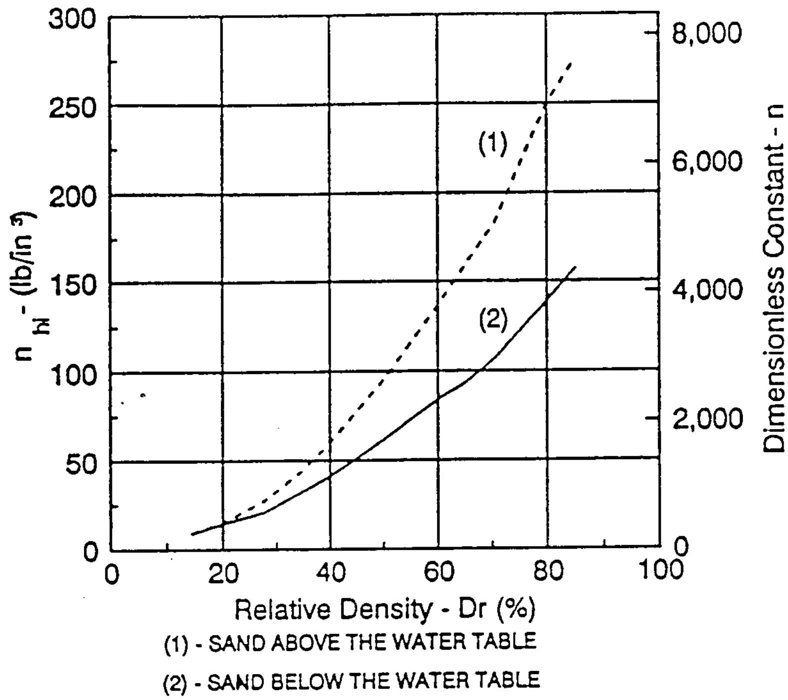


Fig. 2.2 n_{hi} vs. relative density, after Murchison and O'Neill (1984)

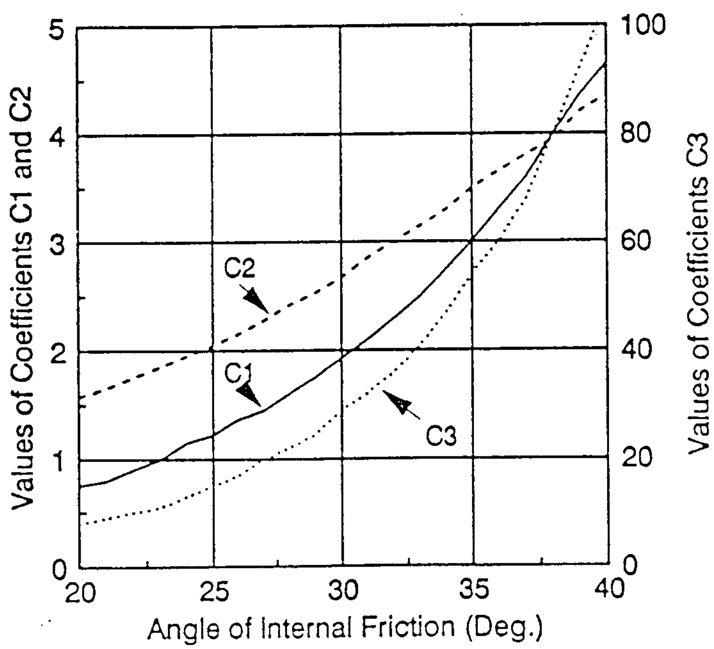


Fig. 2.3 Factors for P_u, after Murchison and O'Neill (1984)

2.2.1.3 FINITE ELEMENT APPROACH

Various studies have been conducted on the behaviour of the piles using the finite element methods. Due to its versatility this method is most suitable for studying effects of various parameters such as soil nonlinearity, soil nonhomogeneity, etc. on the pile response. Nair et al(1969) conducted a detail study of single piles and pile groups using finite element methods. Butterfield and Banerjee(1971) used finite element method to study the effects of various materials on the pile foundation, i.e. concrete, steel and wood. The soil-pile separation is stress dependent and affects the pile capacity significantly. To model this stress dependency a detailed three dimensional finite element program is required.

2.2.2 FIELD TESTING

In the field testing of single piles, a hydraulic actuator and a reaction pile are used to apply a horizontal force on the pile. The loading connection can be made as required. Generally a free head connection is made. In this type of connection the pile head is allowed to rotate with the application of load. The pile is instrumented to measure and record the applied loads and displacements. The bending moment along the pile length can also be computed by attaching strain gauges at various depths. Also the rotation of the pile at the pile head or ground level can be calculated or observed. Reese and Cox (1986) have proposed a method to develop the P-y curves along the length of the pile using the pile head deflections and rotations, along with the corresponding loads.

In practice, field tests are very costly. A large number of the case histories reported have been performed on the piles with pile response measured only at the pile head. Very few tests are performed on fully instrumented piles. Results of the few fully instrumented full scale pile tests are used along with the other tests to evaluate the pile-soil interaction behaviour along the pile length.

Fully instrumented test piles were used by Alizadeh and Davisson (1970) in the Arkansas River Project. In this project, the piles were steel H shaped piles in medium dense sand subjected to lateral static and cyclic loads. The loads were applied horizontally at the ground level. In the analysis of the cyclic behaviour of the piles, the unloading behaviour of the piles is very important. The residual moments left in the piles are also very important. Unfortunately these were not reported. The load-deflection behaviour of the pile head and the bending moments along the piles at various load levels were reported. Using the Matlock and Reese (1960) method, where the observed pile head response is modelled elastically, it was found that the pile model parameter, n_h, depends on the displacement or the load level. The pile head deflection was found to significantly increase with number of cycles under one-way cyclic loading. However, soil-pile interaction in terms of P-y curves was not evaluated in these studies.

The major breakthrough in the analysis of single piles subjected to lateral loads came in 1974, when Cox et al (1974) reported test results of the single fully instrumented pile embedded in sand subjected to lateral monotonic and cyclic loads. In these tests, the pile head response was measured along with the bending moments along the length of the pile. This assisted in determining the pile-soil interaction not only at the ground level but along the length of the pile as well. Based on these results Reese et al (1974)proposed P-y construction method for the vertical piles in cohesionless materials subjected to lateral loads.

Brown et al(1987) of the University of Houston conducted displacement controlled two-way cyclic loading on single piles. The piles were embedded in sand overlying stiff clay deposits up to about 10 pile diameters deep. It was found that response of the piles in sand was not affected significantly by the number of two-way loading cycles. It was also found that the Reese et al (1974) P-y curve procedure underestimates the field measurements.

Most of the field studies were not comprehensive and hence do not allow for a fundamental study. It is desirable to perform fully instrumented full scale lateral pile load tests. However, such pile tests are expensive and time consuming.

2.2.3 MODEL STUDIES

Model tests are often used due to the low cost involved and the convenience. Previous to the development of the Centrifuge model testing, all the model tests were performed in 1g gravity condition and the pile responses were extrapolated to field condition. But due to the different stress conditions, the model tests had severe limitations over the full scale field testing.

With the development of the Centrifuge testing method, the number of model tests conducted at the field stress levels is increasing, but is still very small due to the large costs involved.

Kubo(1963) conducted 1g model pile tests in sand. The pile section was either rectangular or circular and the pile head conditions used were either fixed or free head restraint. Based on the results of these experiments an equation for the soil pressure along the pile length was proposed as follows:

$$p = k \cdot z \cdot y^{0.5} \qquad \ldots\ldots\ \text{eq. (2.15)}$$

where k is the fitting parameter,

z is the depth along the pile,

y is the pile deflection.

Based on the analysis of some lateral load tests on sandy and clayey soils using the subgrade reaction method, Yoshida and Yoshinaka (1972) indicated that for a circular plate the horizontal soil reaction modulus is a function of the diameter of the plate. These results contradict the results reported by

Terzaghi (1955), Reese et al (1974).

Poulos reported some model tests at 1g condition for studying the single pile and pile group responses in clay (Mattes and Poulos,1971) and sand (Selby and Poulos,1983). Using these test results Poulos calibrated the elastic boundary element solution for use in practice. Scott (1976) performed a series of model pile tests in the centrifuge. The testing program was designed to simulate the full scale field testing at the Mustang Island performed by Cox et al (1974). The full scale testing condition was simulated in 100 g with both dry and saturated soil conditions. The model test results from these tests underestimated the field pile head response. This shows the difficulties in the model testing even at the field stress levels.

Barton(1982) conducted a more comprehensive study of piles subjected to static and cyclic lateral loads in the centrifuge testing machine. The comparison between the experimental P-y curves and those proposed by Reese et al (1974) showed that Reese et al curves underestimate the soil resistance near the ground surface and overestimate it at greater depth.

Zelikson(1978) first employed the hydraulic gradient similitude method to study model piles under inclined loads. Yan(1991) conducted a thorough study of model piles subjected to static and cyclic lateral loads using the same principle. In his study, the model pile consisted of steel pipe piles either fixed or pinned at the top. The experimental P-y curves were compared with the theoretical P-y curves obtained by using API method. He found that the same set of P-y curves can be used regardless of pile head eccentricity and fixity. Based on test results, he found that the API method tends to overpredict the bending moment and shear force at large loads and underestimates displacement at smaller loads. Also on the basis of the finite element studies, he found out that the hyperbolic soil parameters give overall better prediction in all aspects of pile response for plane strain analysis.

2.3 REVIEW OF THE PILE GROUP RESPONSE TO THE LATERAL LOADS

Although there is a good understanding of the single pile response to lateral loads, the response of the pile group and the load transfer and the pile-soil-pile interaction is still not completely understood. Very few field pile group tests are conducted due to large costs involved. Because it is very difficult to obtain the same soil stress in the laboratory as in the field the number of model tests conducted are also very few. The theories that have been proposed are based on a small database and cannot be validated correctly due to lack of a large database. In the past few decades some research has been directed towards this problem and as result there is some understanding of the pile group response to lateral loads. The following paragraphs present a detailed review of the testing and the theoretical solutions developed during past few years.

2.3.1 FIELD TESTING

All the lateral pile load tests were conducted by applying the load using a reaction pile at some distance away. The loads on the piles in the pile group and the displacement of the piles at the top in the pile group were measured. The bending moment was obtained at specified points along the pile length and integrated numerically to obtain the bending moment profile. This bending moment is then integrated to get slopes and deflections, and differentiated to get force and soil pressures on the pile. As the installation of the strain gages and measuring the strains produced is costly, many researchers still measure only the pile head load and deflections.

Schmidt (1981,1985) conducted an extensive field testing program on a group of vertical piles. He subjected the pile group to one cycle of loading. Cyclic loading was followed by loading one pile in the pile group and measuring the displacements of the adjacent pile. The results of the experiment showed that the induced displacements have no relationship with the efficiency of the pile group. The efficiency of the pile group is defined as, the ratio of the total load taken by the pile group divided by the product of the load taken by single pile for same displacement and the number of piles in the group. It should be noted here that, during his testing he first subjected the pile to one cycle of loading and then used the same set up to study the effect of the induced then used the same set up to study the effect of the induced displacement on the pile group efficiency. Also, during the experiments he noticed that the depth of the maximum bending moment in the rear piles in pile groups is more than that in the single pile. Similar results were obtained from the tests conducted by Sharnouby and Novak(1985). In their testing they used a pile group of six piles subjected to lateral load. A pile group of eight piles was tested by Holloway(1981).

Reese et al(1987) conducted lateral load tests on pile group. Based on the results of these tests, they suggested that the effect of pile group can best be achieved by not increasing y in the typical P-y curves for the pile-soil system but by reducing P. Ochoa and O'neil (1989) conducted full scale lateral load pile group testing in submerged sand from medium to high relative density. Their results show that the interaction between the piles in the laterally loaded pile group is very much dependent on the pile positions and the load applied. This suggests it is unwise to assume the reciprocity of the interaction factors.

2.3.2 MODEL TESTING

Although the field testing is very costly, not many researchers have conducted model studies of groups of piles. One of the reason for this is that until recently model studies involved only reducing the size of the prototype and testing it under low stress conditions. Since the behaviour of the soil is stress dependent the response of the small soil sample at low stress level is very different than the field response. A second reason was that, due to boundary effects, there was a limit to the size of the model that could reasonably represent the field prototype.

In 70's and 80's, with the development of the centrifuge modelling technique, model testing was given an altogether different perspective. Using this technique, it became possible to conduct tests on small models at stress levels equivalent to that in the field. But the centrifuge model testing is very costly. Also it requires highly skilled technical staff to maintain and operate the machine. Therefore, many researchers still prefer to conduct the tests in normal stress conditions.

Meyerhof et al (1988) reported group tests on model piles of various materials. Davisson and Sally (1970) report the model testing of a large group of piles for the Arkansas River Navigation Project. These tests were conducted under a stress condition of 1g. Aurora (1983) reports centrifuge testing used for the analysis of behaviour of group of piles of large diameter. The tests conducted by Kulkarni et al (1985) included groups of piles having two or three piles, and the tests were conducted in the centrifuge machine. The piles were fixed at the top and connected with a pile cap. During their tests they noticed that the non-linearity of the soil and the plastic flow of the soil around the pile are very important in the laterally loaded pile group analysis. Also it was seen that the front piles share a much larger load and flexural stresses.

The tests conducted by Shibata et al (1989)on laterally loaded pile group were under normal stress conditions. The efficiency of the pile group from experiment was compared with the theoretical efficiency. The solution given by Randolph(1981) was used for this purpose. A discrepancy of 30% was observed in the results.

2.3.3 ANALYTICAL STUDY

Analytical methods for pile groups include the theoretical solutions and the solutions based on numerical methods. Theoretical solutions of a single laterally loaded pile have been explained in previous paragraphs. The elastic approach can be readily extended to analyze the pile groups, while it is difficult to analyze pile groups by the P-y curve approach. In case of single piles the P-y curve approach is the most widely used method of analysis. The current practice for analysing the pile groups is to use softened P-y curves for the group analysis. The modulus of subgrade reaction approach for single piles was described above.

Broms (1964) first proposed a theory to analyze the group of piles loaded laterally. The main concept of his theory was that the piles can be treated as beams and long piles develop plastic hinges at a certain depth below the ground level. If this depth can be determined and the pile response compared with that of the beam, then the beam can be used to predict the pile response.

Randolph (1981) developed an expression to calculate the interaction factors based on the finite element analysis of pile groups. He solved the differential equations using the Fourier technique instead of the boundary element method used by Poulos(1971). The expression for a fixed head pile given by Randolph is as follows,

$$\alpha_{\rho F} = 0.6\,\rho_c \left(\frac{E_p}{G_c}\right)^{1/7} \frac{r_0}{S}\,(1 + \cos^2\psi) \qquad \text{....... eq. (2.16)}$$

Where α_{pF} is the interaction coefficient for a pile group with fixed head,

S is the pile spacing,

r_0 is the pile radius,

ψ is the angle between pile centres and the direction of the load,

ρ_c factor to take into account the variation of soil stiffness with depth,

0.5 for stiffness proportional to the depth,

1.0 for homogeneous soils,

G_c the average value of G^* over the active length of the pile,

G^* $= G\,(1 + 3\mu/4)$, ... eq. (2.17)

μ Poison's ratio,

G shear modulus of the soil,

If $\alpha_{pF} > 0.5$ then $\alpha_{pF} = 1 - (4\,\alpha_{pF})^{-1}$ .... eq. (2.18)

Similarly for free headed piles the interaction factor is given by

$$\alpha_{\rho h} = 0.5\,\rho_c \left(\frac{E_p}{G_c}\right)^{1/7} \frac{r_0}{S}\,(1 + \cos^2\psi) \qquad \text{....... eq. (2.19)}$$

If $\alpha_{ph} > 0.5$, then $\alpha_{ph} = 1 - (4\,\alpha_{ph})^{-1}$ .... eq. (2.20)

Focht and Koch (1973) proposed a theory, called y-modifier approach, which is most widely used in practice. They suggested use of elastic interaction factors given by Poulos(1971) for interaction of piles and use of P-y curve method for obtaining deflection of single pile. According to their method the deflection of a pile group, ρ_k, is given by

$$\rho_k = \bar{\rho} \sum_{(j=1, j \neq k)}^{m} \rho_j\,\alpha_{kj} + y_t \qquad \text{....... eq. (2.21)}$$

Where P Unit deflection at the mudline,

ρ_j Displacement of j^{th} pile,

α_{ij} Interaction effect of pile k on pile j,

y_t Deflection of the single pile.

The limitations of the theory come from the differences in the assumptions made in the subgrade modulus theory and the elastic theory by Poulos. The subgrade modulus theory is based on the assumption that the soil response can be modelled by well defined springs which are not connected to each other whereas the elastic model assumes the soil to be an elastic continuum. The y modifier approach suggested above softens the P-y curve not only at mudline but at all depths. Hence the resulting moment and deflection curve overestimate the moments and deflections.

The above equation can be rewritten as

$$\rho_k = \overline{\rho_F} \left(\sum_{(j=1, j \neq k)}^{m} H_j \alpha_{jFkj} + R H_k \right) \qquad \ldots\ldots \text{ eq. } (2.22)$$

Where ρ_k is the deflection of the k^{th} pile,

ρ_F Unit reference displacement of a single pile under a unit horizontal load, from elastic theory,

H_j Lateral load on pile j,

α_{jFkj} Coefficient to get influence of pile j on pile k,

R Relative stiffness factor, where R is the ratio of mudline deflection of a single pile from P-y method to the mudline deflection from Poulos's method,

H_k Lateral load on pile k,

m number of piles

Reese et al (1984) compared the results from this method with the results from the field tests. They also compared the results by analysing the pile group as a large diameter imaginary pile. They found that the y modifier approach by Focht and Koch(1973) is very sensitive to the R value used.

Sharnouby and Novak (1986) proposed a new method based on Mindlin's displacement field in the elastic half space. The main concept is to view the whole pile group with the soil as one compressible continuum whose conditions of equilibrium are specified at a number of discrete points. The stiffness of this composite continuum is obtained by combining the pile stiffness with the soil stiffness. The piles are assumed vertical and of constant circular cross section. The stiffness of soil is derived using Mindlin's solution for displacement field generated in the interior of the elastic half space by a horizontal point load. The displacements in the vertical direction and the direction perpendicular to the direction of loading are considered to be zero. To get the stiffness of the continuum, the stiffness of the pile and soil are added together. This theory is basically a linear theory, but the non-linearity is approximated by adjusting the soil stiffness and material damping to the level of strains expected and by incorporating a weakened zone around the pile.

Brown et al (1988) proposed a concept of P multiplier based on the analysis of field experiments. This approach amounts to reducing the soil pressure for the given displacement rather than increasing the displacements for given soil pressures. It is argued that the effect of the overlapping shear zones in reducing the soil resistance is more dominant than the superposition of strains. Thus if P-y curves for single pile are available along with the P-multiplier factor (f_m) then one can easily obtain the P-y curves for the piles in the pile group.

2.3.3.1 FINITE ELEMENT ANALYSIS

One of the main advantages of the finite element analysis is that it is very easy to incorporate a soil model that is more appropriate, i.e. elastic, incremental elastic, elasto-plastic, etc. Other advantage is that it is easier to study the effects of various factors on the pile group response. Also the variations and restrictions of three and two dimensional analyses can be compared. The main drawback of the finite element analysis is that the solution is vary costly compared to the solutions in the form of charts and figures.

A number of researchers have carried out finite element analysis of pile groups subjected to lateral loads. Nair et al (1969) conducted a three dimensional analysis to study the pile group response. They used the concept of equivalent cantilever which was a modification of the model proposed by Broms (1964). In this method each pile is replaced with an equivalent cantilever which has

1. structural section identical to the original pile
2. equivalent axial length (L_c) for resisting direct loads
3. equivalent bending length (L_b) or resisting lateral loads

and moments.

The basis for determining L_c and L_b is that the behaviour of the cantilever and the actual pile be equivalent under direct loads.

The hyperbolic model developed by Duncan and Chang (1970) was used by Tamura et al (1982) and later by Muqtadir et al (1985) in their finite element analysis. In their analysis Muqtadir et al developed a special element called thin layer element to model interaction behaviour between the soil and the pile.

In their analysis, Kay et al(1983) compared the finite element response with the P-y curve approach for single piles, while the group effect was accomplished by applying the free field displacements to the pile, instead of calculating the interaction factors by elastic method. This approach has previously been used for piles near slopes and in offshore piles in mud slide areas. The main advantage of the method is that, both the single as well as group pile behaviour can be obtained from one finite element program. Also, the error that occurs in the Focht-Koch method due to the non compatibility of the two methods, P-y curve approach and elastic continuum approach, used is eliminated.

Chow et al(1987) used a new approach by which they divided the pile soil system into two systems. One system consisted of the group piles acted upon by an external applied loads and pile-soil interaction forces and second system consisted of a layered soil continuum acted upon by a system of pile-soil interaction forces at the imaginary positions of the piles.

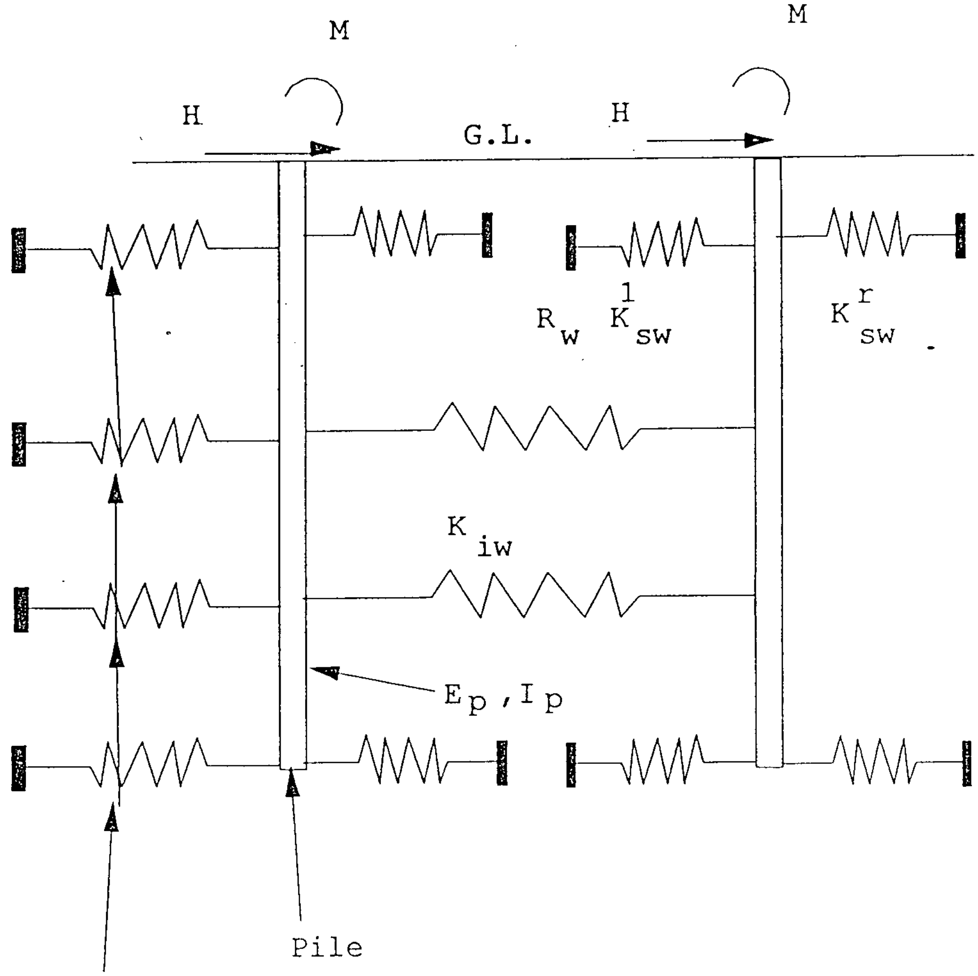
Najjar and Zaman(1988) incorporated a plasticity model developed by Desai and co-workers to model the soil non-linearity in their study while piles and pile cap were assumed linear elastic. Trochanis et al(1991) used the method proposed by Kay et al(1983) and conducted a three dimensional analysis. During their study they changed various variables like load eccentricity, pile spacings,etc. Based on these studies they developed a simple one dimensional finite element program and calibrated it with the results of the three dimensional analysis. In this program the piles were connected with each other to take into consideration the pile interaction effect and at the same time the stiffness of the springs between piles and between pile and soil was adjusted so as to obtain the non-linearity of system. The schematic representation of the model is shown in Figure 2.4.

2.4 SUMMARY

From the above review it appears that the non-linearity of the soil, soil-pile system, and the non-homogeneity of the system is critical in the analysis of the laterally loaded pile groups.

At present there is no cost-effective sound theoretical solution which can be used for analysing pile group behaviour under lateral load. Although some of the solutions discussed above are currently used in practice they are mostly based on experience. At the same time these methods do not take into account all the factors that significantly influence the pile group response. The analytical method can be developed and evaluated by comparing it with the experimental results.

The most reliable data can be obtained by conducting field tests. Due to the high costs involved very few field tests are done. The centrifuge testing machine or the hydraulic gradient similitude test can be effectively used to obtain model test results that are similar to the field response. The hydraulic gradient similitude test as used for this thesis is a cost effective method of conducting a model scale test to represent the field prototype.



P-y Springs or Winkler's Springs

K_{iw} is the P-y spring connecting the pile to the pile.

K^r_{sw} is the P-y spring connecting the pile to the soil.

K^1_{sw} is the P-y spring connecting the pile to the soil between the piles

Fig. 2.4 Concept Of Winkler's Springs For Two-Piles (Trochanis et al, 1991)

CHAPTER 3 : HYDRAULIC GRADIENT SIMILITUDE PRINCIPLE

3.1 INTRODUCTION

In this chapter the testing principle and scaling laws of the hydraulic gradient similitude method (HGST) are described. This testing principle was first introduced by Zelikson in 1969. Since then it has been used in model tests of anchor and pile problems (Zelikson 1978,1988a, Yan 1990, Dau 1991). Zelikson et al (1982) and Zelikson and Laguay (1981) have compared HGST with centrifuge model and good results were observed where comparison was possible. This technique was originally developed to consolidate the soil sample before conducting the centrifuge model test. Later it was modified to conduct model testing of foundations on level surfaces.

The first HGST model testing equipment in the North America was developed at University of British Columbia by Yan and Byrne(1991a). Various tests were conducted including the model study of shallow footings, single pile response to static and dynamic lateral load, downhole and crosshole seismic tests. These tests were used to study the various factors affecting the test results and the testing device was upgraded continuously.

For this study the equipment was modified to conduct pile group testing. The principle used in HGST is similar to that used in centrifuge testing. The body forces acting on the soil particles are increased to simulate the field stress conditions. But where the centrifuge test uses centripetal acceleration to increase the body forces, the HGST technique involves increasing body forces by increasing the seepage force through the porous material.

3.2 Hydraulic Gradient Similitude Principle

Figure 3.1 shows a sample of soil subjected to a controlled downward hydraulic gradient, i. The downward hydraulic gradient will increase the body force on a unit volume of the sample by an amount $i\gamma_w$. This is equivalent to increasing the unit weight of the material by $i\gamma_w$. The effective unit weight of the model soil γ_m can be given by

$$\gamma_m = i\gamma_w + \gamma' \qquad \text{....... eq. (3.1)}$$

Where i is the applied downward hydraulic gradient,

γ_w is the unit weight of the water, and

γ' is the submerged unit weight of the soil

Thus the sample soil or model can be considered to have a unit weight, γ_m. If the soil in the field (prototype) has an effective unit weight, γ_p, then the scale factor, N, is given by

$$N = \gamma_m / \gamma p$$

$$= (i\gamma_w + \gamma') / \gamma_p$$

If $\gamma_p = \gamma'$, then

$$N = (i\gamma_w + \gamma') / \gamma'$$

$N \approx i\gamma_w / \gamma'$ (Since $i\gamma_w >> \gamma'$)

and since $\gamma_w \approx \gamma'$,

$N \approx i$ (approximately) .... eq. (3.2)

The factor N is called the Hydraulic gradient scale factor.

For a hydraulic gradient test with gradient N=n, where n is the scale of the model used, the stresses due to the self weight in the model and prototype at the homologous points will be equal as shown below.

Model	Prototype
$\gamma_m = N \cdot \gamma_p$	γ_p = Prototype soil density
$Z_m = Z_p / N$	Z_p = Depth at a point in prototype
$(\sigma_v)_m = \gamma_m \cdot Z_m$	and $(\sigma_v)_p = \gamma_p \cdot Z_p$
$(\sigma_v)_m = (N.\gamma_p) \cdot (Z_p/N)$	$= \gamma_p \cdot Z_p$

where Z_m and Z_p are the model and prototype depths and $(\sigma_v)_m$ and $(\sigma_v)_p$ are the effective vertical stresses at the homologous points of model and prototype soil elements, respectively.

This shows that the scale factor for stresses will be unity. Thus if same soil is tested in the model, as in prototype, and the same stress path is followed then the strains in the model and the prototype will be same at homologous points while the displacement of the prototype will be 'N' times that of the model. Therefore the Hydraulic gradient similitude tests are expected to follow the same scaling laws as the centrifuge modelling tests.

This scaling laws, for the hydraulic gradient similitude testing method, were examined by Yan (1990) and Dou (1991). A summary of these laws is given in table 3.1.

Since it is difficult to simulate both the specific geological settings of the prototype soil conditions and the exact stress path followed in the prototype loading, direct comparisons between model and prototype is not always possible. Therefore, another experimental technique known as modelling of models was used by Yan to verify the results obtained from the HGST. In this method models of different scales are tested at various hydraulic gradients such that they will represent the same prototype. The results are then compared to verify the similitude laws.

The HGST device used in testing was fitted with three pore water pressure transducers and the hydraulic gradient was monitored continuously throughout the test.

The piles were tested in the HGS device under a constant hydraulic gradient. The hydraulic gradient throughout the sample was monitored continuously to maintain the constant gradient. The major objective of using the modelling tests in this study was to generate a data base on Pile group response to lateral loads from which methods of analysis can be tested. Modelling tests can be used to analyze and investigate prototype behaviour directly.

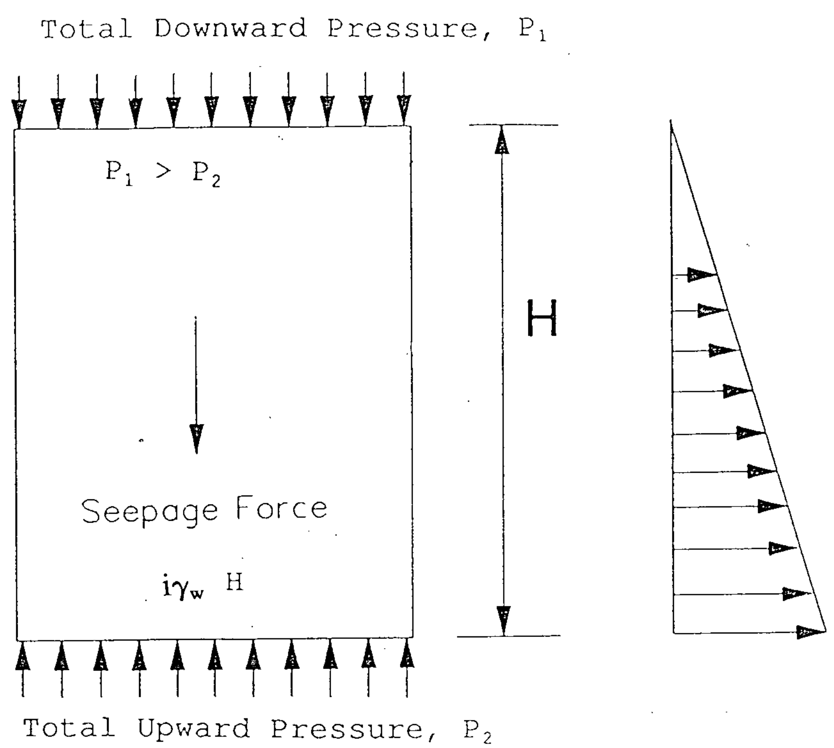


FIG. 3.1 Hydraulic Gradient Similitude Principle

TABLE 3.1 SCALING RELATIONS FOR CENTRIFUGE AND HYDRAULIC GRADIENT TESTS

QUANTITY	FULL SCALE	MODEL AT N g'S
LINEAR DIMENSION	1	1/N
AREA	1	1/N^2
VOLUME	1	1/N^3
STRESS	1	1
STRAIN	1	1
FORCE	1	1/N^2
ACCELERATION	1	N
VELOCITY	1	1
TIME- IN Dynamic TERMS	1	1/N
TIME - IN DIFFUSION CASES	1	1/N^2
FREQUENCY IN DYNAMIC PROBLEMS	1	N

CHAPTER 4 : MODEL SOIL AND PILE PROPERTIES

4.1 MODEL SOIL

The sand used in the tests is uniform rounded Ottawa sand. The mineral composition of this sand is primarily quartz. It has a specific gravity of 2.67 and a constant volume friction angle ϕ_{cv} of 31°. The grain size distribution curve for this sand is shown in Figure 4.1. Only sand retained on #140 sieve was used for testing purpose. Reference minimum and maximum void ratio of the sand are 0.58 and 0.88 respectively. Figure 4.2 shows the variation of permeability with void ratio. The hyperbolic stress strain parameters of the sand for relative densities of 30% and 75% are given in the table 4.1. Using these properties and the hyperbolic model proposed by Duncan and Chang (1980) the P-y curves for the given pile-soil system can be developed. For finite element analysis the following formulae were used to calculate the stresses in the soil sample, the initial Young's modulus, and the Young's modulus at various stages of loading. A constant relative density of 75 was used for all the tests.

$$\Delta \varepsilon_1 = \frac{\Delta \sigma_d}{E_t} \qquad \text{....... eq. (4.1)}$$

$$\Delta \varepsilon_v = \frac{\Delta \sigma_m}{B_t} \qquad \text{....... eq. (4.2)}$$

where ε_v is the volumetric strain,

ε_1 is the major principal strain,

σ_d is the deviator stress,

σ_m is the mean normal stress,

E_t is the tangent Young's modulus,

B_t is the tangent bulk modulus

$$E_t = E_i \left(1 - \frac{\sigma_d}{\sigma_{df}} R_f \right)^2 \qquad \text{....... eq. (4.3)}$$

$$E_i = k_E P_A \left(\frac{\sigma_3}{P_A} \right)^n \qquad \text{....... eq. (4.4)}$$

$$B_t = K_E P_A \left(\frac{\sigma_3}{P_A} \right)^m \qquad \text{....... eq. (4.5)}$$

Where E_i is the initial Young's modulus,

P_A is the atmospheric pressure,

σ_{df} is the ultimate deviator stress,

4.2 MODEL PILE PROPERTIES

Two piles were used in the experiment. Both the piles were

TABLE 4.1 Hyperbolic Soil Parameters From Drained Compression Triaxial Tests

Sands	K_E	n	K_b	m	R_f	ϕ_1	$\delta\phi$	ϕ_{cv}	K_0
D_r = 30%	600	0.88	470	0.25	0.95	32	0.0	31	0.5
D_r = 75%	1600	0.67	600	0.05	0.70	39	4.0	31	0.4

Where

K_E The Young's Modulus Number

n The Young's Modulus Exponent

K_b The bulk modulus number

m The bulk modulus number

R_f The failure stress ratio

ϕ_1 The mobilized friction angle at a confining stress of 1 atm.

$\delta\phi$ The decrease in the mobilized friction angle for a tenfold increase in the confining stress

ϕ_{cv} The constant volume friction angle

K_0 The at-rest pressure coefficient (1 - sin ϕ)

TABLE 4.2 Physical Properties Of Model Piles

Outer Diameter, inches	1/4"
Thickness,inches	0.032"
Length, mm.	424.0
Weight,gm.	20.3
m (gm/mm)	0.0479
EI ($N.mm^2$)	4.03×10^6

NOTE : - Two identical piles were used for the tests. One of the piles was instrumented with eight pairs of strain gauges while the other was uninstrumented.

of diameter 6.75mm. (1/4 inch) and 424mm. long. The piles were made of 6061-T6 aluminium tubing. One of the piles was instrumented with 8 pairs of strain gauges for measurement of the bending moment. This arrangement in particular allows measurement of the bending moment variation along the pile length and thus gives a deflection profile along the length of the pile. The connections for these strain gauges were provided on the cylinder lid. The flexural rigidity of the pile was measured by fixing one end of the pile in a clamp and applying a known load at the free end. From the deflection and load measurements the pile flexural rigidity value was determined. The value is given in TABLE 4.2. Eight pairs of 120 Ω foil type strain gauges mounted on the outside of the pile were used. The position of the strain gauges is shown in Figure 4.3. The advantage of using a pair of strain gauges mounted on opposite sides is that the effect of tension and compression, on the opposite faces of the pile, is compensated. The piles were installed in the sample after the preparation of the soil sample. For this purpose various guiding blocks were designed to push the piles into the soil. The pile guides were made from plexiglass and designed in such a way as to provide the required spacing in between the piles.

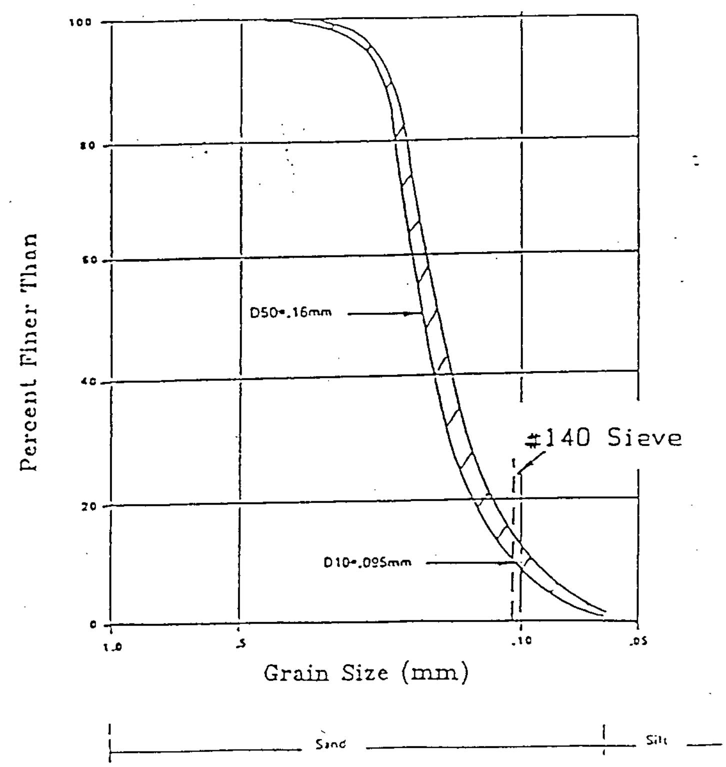


Figure 4.1 Grain Size Distribution of Fine Ottawa Sand (Yan Li, 1991)

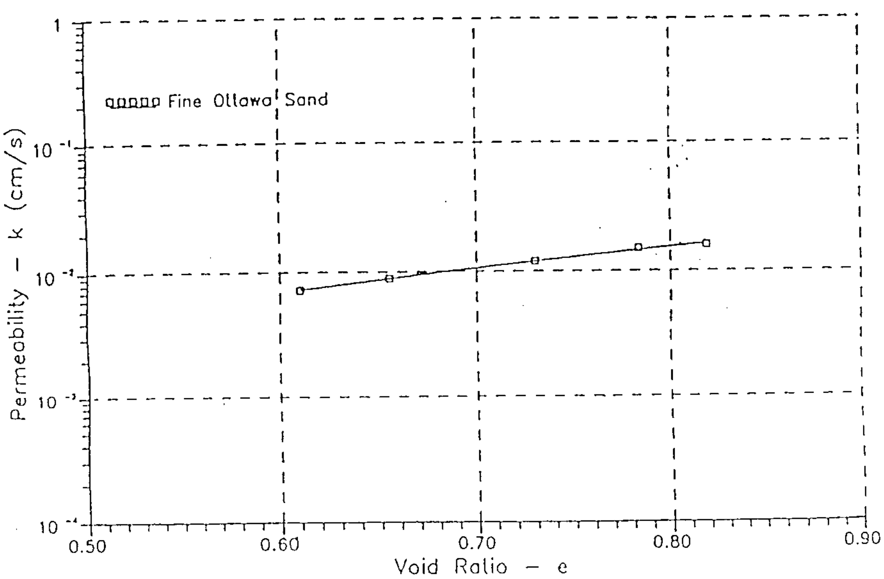


Figure 4.2 Variation of Permeability Vs Void Ratio (Yan Li, 1991)

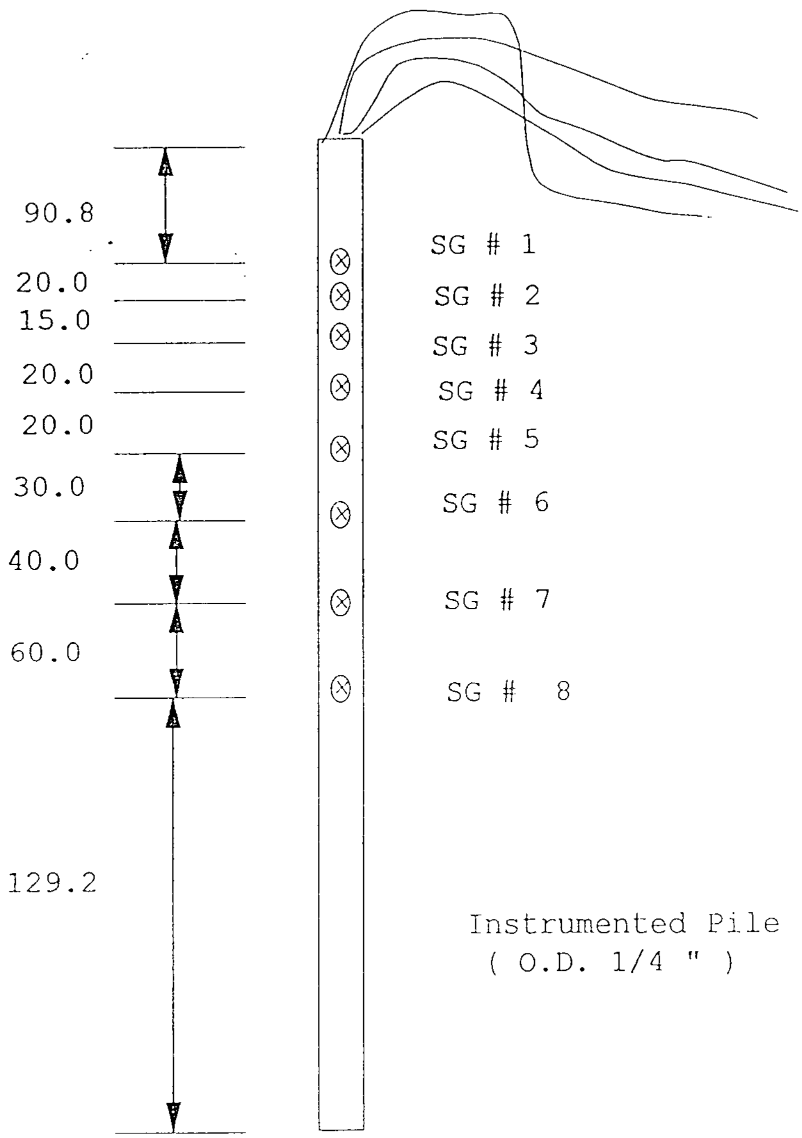


Figure 4.3 Pile Instrumentation
All Dimensions in mm.

CHAPTER 5 : HYDRAULIC GRADIENT SIMILITUDE TESTING DEVICE

5.1 INTRODUCTION

In this chapter the hydraulic gradient similitude testing device developed at University of British of Columbia is described in detail. The design and fabrication of this device was started in December,87. The device was designed in such a way that the construction and the operation of the device is simple and at the same time facilitating versatility in application with reliability of results. The device has been under continuous modification and improvement to incorporate various applications.

At present the device is mainly designed to perform load controlled lateral load tests on vertical piles. It can also be used to conduct displacement controlled test, axial load test on piles or other types of foundations resting on level ground with some minor modifications. In addition to the static testing the dynamic loads or seismic loads can also be applied to the piles.

5.2 Hydraulic Gradient Similitude Testing Device

The UBC-HGST device is shown in figure 5.1. The device consists of

1. a large soil container and air pressure chamber

2. water supply and circulation system.
3. Air pressure system
4. Loading system
5. Data acquisition and control system.

During a test, the water is continuously supplied by a high power centrifugal water pump. The hydraulic gradient across the soil deposit is obtained by applying an air pressure in the air chamber with water drainage provided at the base of the soil container. The water level is maintained about one inch above the sand surface by balancing the air pressure and water flow for a given hydraulic gradient. The pore pressure development in the soil sample due to the increased hydraulic gradient is measured by three pore pressure transducers mounted on the walls of the soil container. The average hydraulic gradient within the sand deposit is obtained from the pore pressure measurements and sample height as follows :

- When the bottom drainage valve is closed and there is no water flow , i.e. i=0
- When the bottom drainage valve is open and water is flowing under gravity effect, i.e. i = 1.
- When the bottom drainage valve is open and water is flowing with controlled air pressure applied at top ,

$$i = \frac{H_w}{H_s} + \frac{(P_1 - P_3)}{\gamma_w H_s} \qquad \text{....... eq. (4.1)}$$

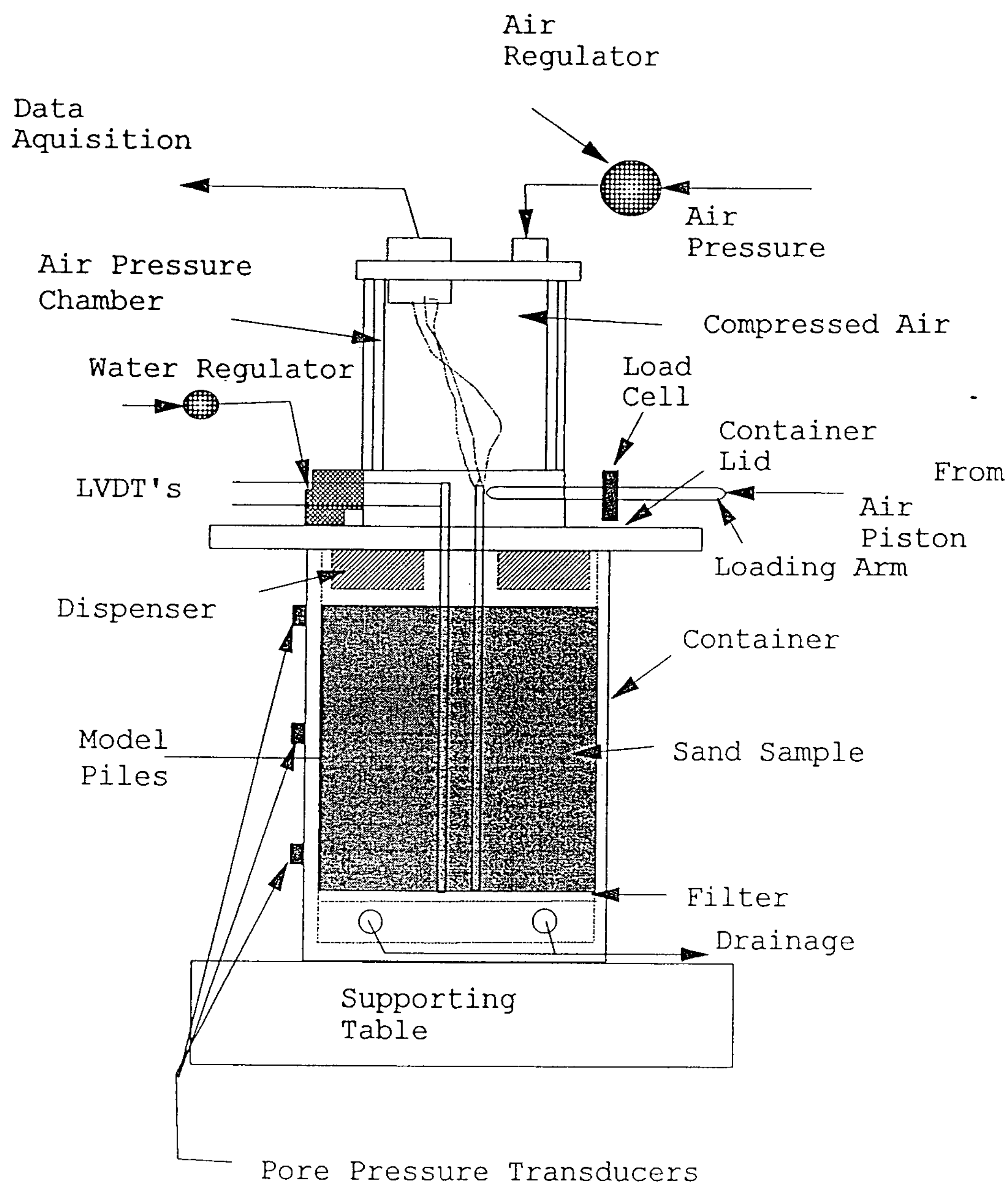


Figure 5.1 Hydraulic Gradient Similitude Device

Where P_1, P_3 are water pressures at top and base of the soil sample, respectively, H_s is the sample height and H_w is the height of water above the sample base.

5.2.1 SAND CONTAINER AND AIR PRESSURE CHAMBER

The soil container in which the model pile groups are tested is a rectangular box with 404 x 190mm inside dimensions and a depth of 400mm. The thickness of the wall is 19.05mm. The box is made of thick welded aluminium plates anodized with hard coatings to prevent water corrosion. The rectangular shape of the box helps in reducing the boundary effects and at the same time reducing the box area size and the flow quantity. The maximum hydraulic gradient that can be applied to the soil using the existing device is 100. The corresponding maximum air pressure required is about 350 kPa.

The soil is retained in the box with the help of a filter supported on a grid of perforated aluminium strips. The strips are about 25.4mm. or 1 in. thick. This filter helps in two ways, it prevents the soil from being washed away with the water and at the same time the space provided below the filter allows the water to flow freely before draining out of the soil container. The filter is prepared by putting a series of stainless steel sieves including #10, #140, #200 mesh sieves resting on top of a 6.35mm. thick perforated aluminium plate. As the sand used has been re-sieved and only the portion retained on sieve #140 is used in the tests, the soil will be retained on the filter with very little head loss across it. The filter and its support strips are designed as a grid system under a uniform distributed load. The spacing of the cellular support is chosen so that the vertical deflection of the filter at each cell centre would be less than 0.1mm.

The soil container lid is 19.1mm. thick plate and is bolted on to the container with 14 Hex Head cap stainless steel bolts. A rubber gasket is used to seal the water pressure between the lid and container wall. The side view and plan of the container lid are shown in Figure 5.2. The lid has a 5"(127mm.) open hole at its centre to allow for pile and soil loading and instrumentation. An annular block 2.5"(63.5mm.) high is permanently bolted on the lid to provide vertical space for mounting loading and deflection measurements units.

A plexiglass cylinder sitting on top of the annular block forms the air chamber. The cap on top of the cylinder is fitted with special pressure tight electrical plugs for instrumentation. The air pressure connection is also provided in the cap. The plexiglass cylinder allows a visual observation during the test. The connection between the annular block and lid and those at top and bottom of the plexiglass cylinder are sealed using O-ring. On the inside of the lid a filter is provided to dissipate the flow of the incoming water so as to minimise the disturbance to the sample. The lid is provided with a 1" I.D. hole for water inlet.

5.2.2 WATER SUPPLY AND CIRCULATION SYSTEM

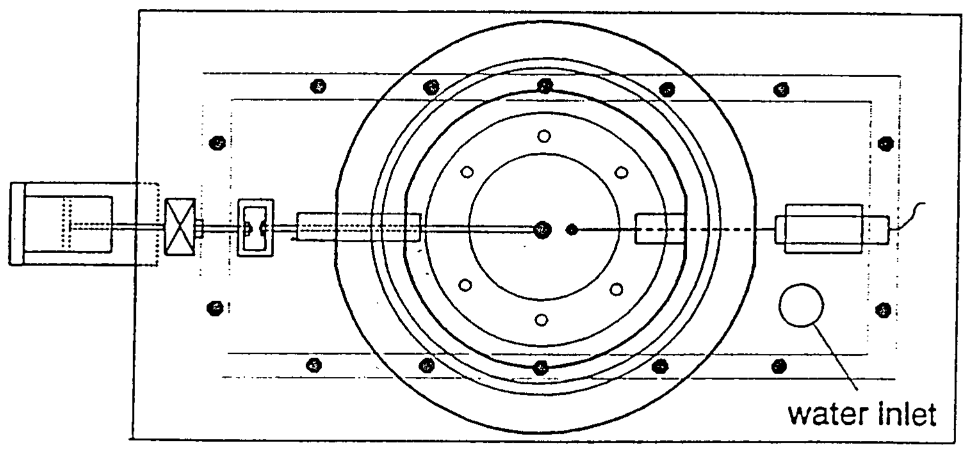
Figure 5.3 shows the water flow chart in UBC-HGST device. The water pump used is a centrifugal type with a capacity of 24 US GPM at a total pressure head of 100 ft. manufactured by Monarch Industries Ltd. The pump has a 1.5hp built in motor and 1.4" and 1" ID suction and outlet pipes, respectively. The pump is connected to the water tank and the container by plastic hoses. Before raining the sand into the container the whole system is connected and saturated.

The water flow is controlled by valves at top and bottom of the soil container. During the test valve #1 and #3 are opened to create a downward gradient and an upward gradient is created by opening valve #2 and closing valves #1 and #3. Although there is an air-water interface above the sand surface considering the short duration and dynamic nature of the test the effect of air diffusion can be neglected.

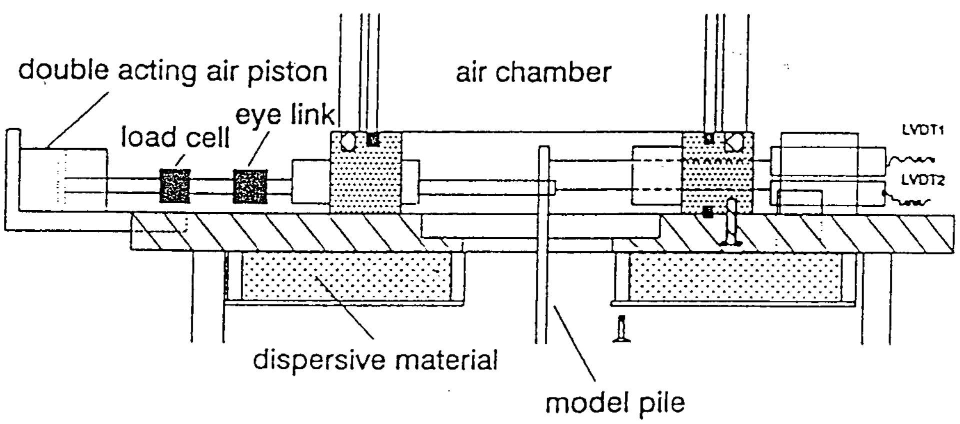
5.2.3 PILE HEAD LOADING AND MEASURING SYSTEM

The loading system included a two way piston to provide two way loads. Both chambers of the piston were connected to regulated air supply systems. Load was applied by controlling the air pressure in both the chambers. The load cell was connected between the piston and the rigid loading ram to measure the applied load.

A low friction loading bushing without O-ring was used to



(a). Plan View of Soil Container Lid



(b). Side View of Soil Container Lid

Figure 5.2 Container Lid – Plan and Side View

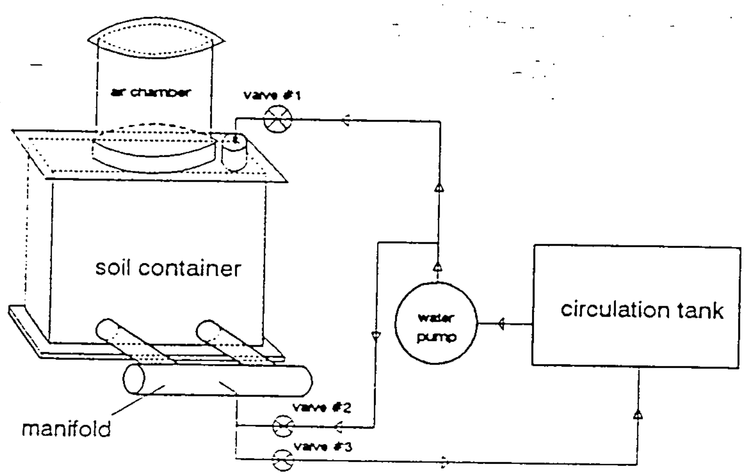


Figure 5.3 Water Flow System in UBC–HGT Device

transfer the load to the pile head. The two way air piston was mounted on the soil container lid. The load cell was calibrated using known weights for both tension as well compression and was found to be linear within the practical range.

For testing the pile group in which both the piles were loaded simultaneously a new load cell was designed to measure the load applied to each of the piles. This load cell is shown in Figure 5.4. This load cell was used only to measure tensile forces. A frictionless air leaking bearing system is used for the LVDT cores. These LVDT's are placed on the soil container lid as shown in Figure 5.5. The LVDT's were so situated that the displacements of the pile opposite the loading ram were measured at the loading point and at a distance above the loading point (20.0mm.). The displacement of the other pile was measured by attaching an LVDT to the loading ram .

5.2.4 DATA ACQUISITION SYSTEM

A micro-computer based data acquisition system was used in this research. This system comprised of three components: a multichannel signal amplifier, multichannel analog to digital converter DT2801A card, and a IBM-PC computer. All transducers were excited by a common supply of 6 Volts.

Total of 16 channels were monitored during the testing. Three channels were used for monitoring the pore pressures at the top, centre and bottom of the soil sample to calculate the hydraulic gradient. Eight channels were used to monitor the pile

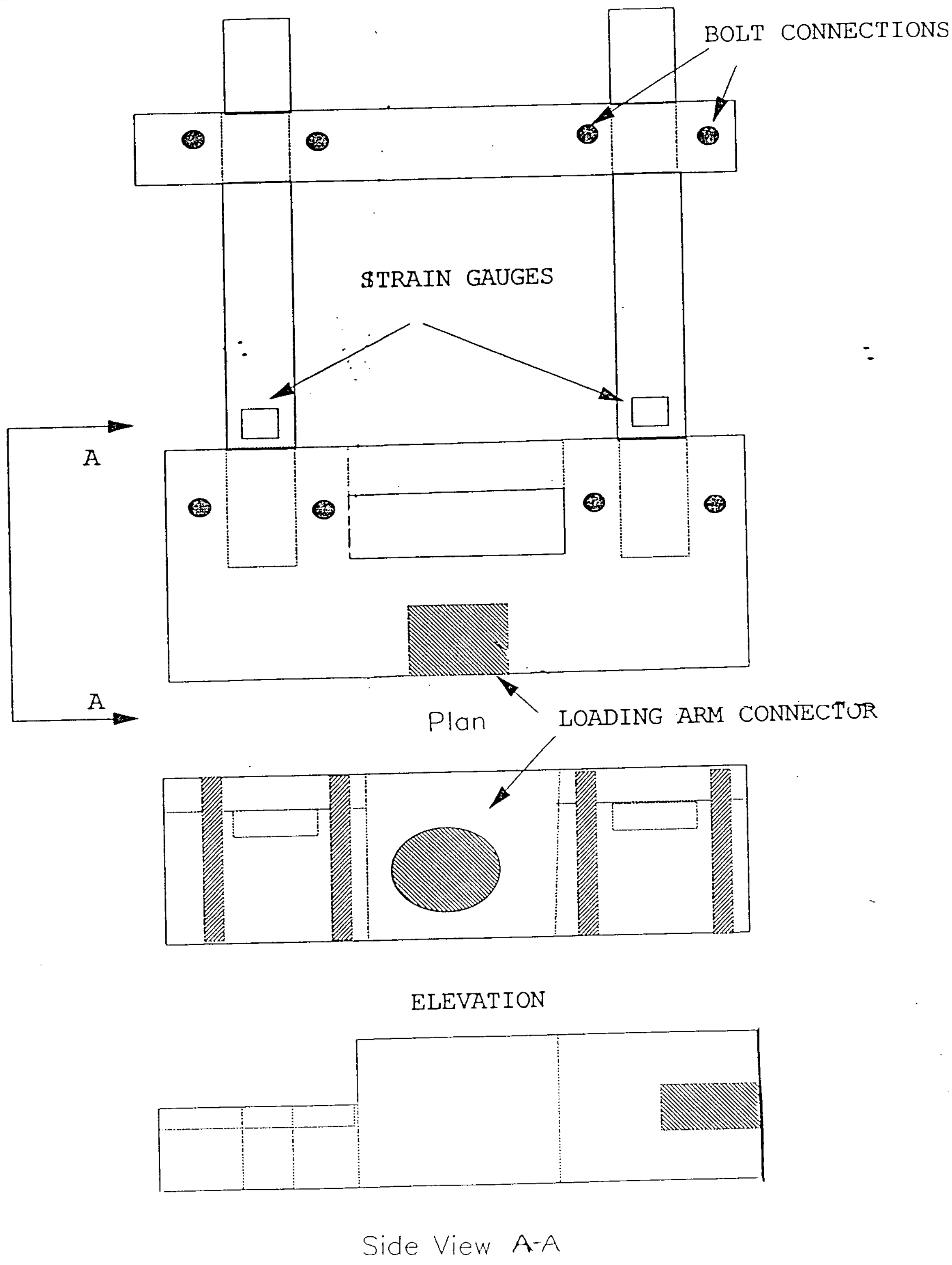


Figure 5.4 Load Cell

strain gauges, three channels were used to monitor the LVDT's and the remaining two channels were used to monitor the two load cells. The transducer signals except 2 LVDT's were all amplified at a gain of 1000 by the amplifier.

The DT2801A A/D converter has 12 bits in its accuracy which gives 4.88mV accuracy for a ±10V bipolar configuration. The noise level was monitored for each channel and found to be ± 5mV at the scanning frequency of 0.5Hz. This gave an accuracy of ± 0.2kPa for the pore water pressure, ± 0.02mm. for the LVDT's, and ± 0.014kg for the load cell.

A program written in Quick basic was used to monitor the hydraulic gradient during the gravitational process and LABTECH NOTEBOOK software was used to monitor channels during the test. The data obtained was then processed to obtain various results.

5.2.5 DATA REDUCTION

In the tests, the bending moment distribution along the pile is obtained from the strain gage readings. Based on the simple beam theory the bending moment can be integrated or differentiated to obtain the pile inclination,Θ, deflection, y, or shear force, Q, or soil resistance, P, as follows

$$\Theta = \int \frac{M}{EI} dz \qquad \text{....... eq. (4.2)}$$

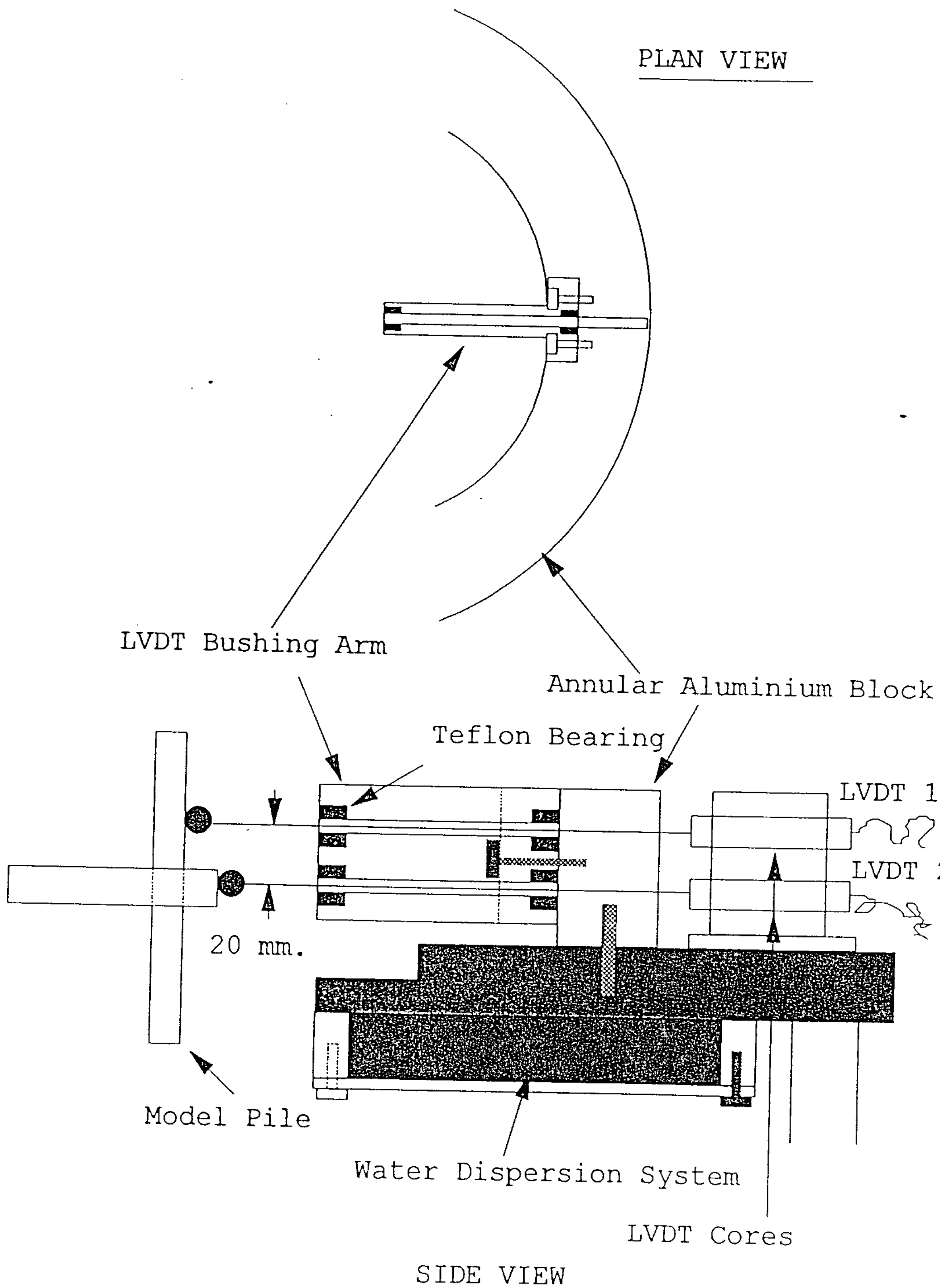


Figure 5.5 Pile Head Deflection Measurement

$$Q = \frac{dM}{dz} \qquad \text{....... eq. (4.3)}$$

$$y = \int\int \frac{M}{EI} dz \qquad \text{....... eq. (4.4)}$$

$$P = \frac{d^2M}{dz^2} \qquad \text{....... eq. (4.5)}$$

Where EI is the flexural rigidity of the model pile, z is the distance along the pile.

Since the bending moment is only known at some discrete locations along the pile, a numerical curve fitting scheme is necessary to obtain the needed soil resistances and pile deflections along the pile length at each loading stage. The desired P-y curve at a given depth can be obtained by repeating the curve fitting scheme at various loading stages. For deflections simple numerical integration is sufficient since any slight error in the bending moment data becomes smoothed in the integration process. However for soil resistance any slight errors or deviations in the bending moment data becomes greatly magnified during double differentiation. To alleviate this problem a cubic spline fitting is used. In this procedure a at each point. Then the spline is differentiated to give the distributed shear force and soil resistance along the pile and integrated to give the pile inclination and deflection. Boundary conditions used are

1. For free head single pile the bending moment at the loading point is set to be zero.
2. For two piles when only one pile is loaded the bending moment above the ground for the adjacent pile is zero. For the loaded pile bending moment is zero at the loading point.
3. For both piles loaded the bending moment at the loading point is zero.

CHAPTER 6 : TEST PROCEDURE

6.1 Test Procedure

The test procedure for the hydraulic gradient similitude test can be divided into three steps as

1. Reconstitution of soil deposit
2. Pile installation
3. Soil loading and subsequent pile loading

6.2 RECONSTITUTION OF SAND DEPOSIT

The technique used for the sample preparation involves upward seepage forces together with sedimentation and densification processes to reform soil deposits for each test as described below.

During the sample preparation, the top cap of the cell was removed and the drainage lines from the circulation chamber (figure 5.3) were closed. De-aired water was used to fill the cell and all the measurement lines. A fixed amount of oven dried sand was weighed in flasks. Water was added to each flask and the sand water mixture was boiled. After cooling to room temperature the boiled sand was transferred to the cell using a water pluviation technique, flask by flask. To remove the layering effect a controlled upward seepage gradient was applied. The slurry formed due to the upward gradient is then stirred to obtain a homogeneous state.

The upward gradient was then turned off and the required soil density was achieved by densifying the soil by tapping the side and base of the cell. After each test was completed, the sand was loosened by an upward gradient and reformed as discussed above.

6.3 PILE INSTALLATION

After preparation of the sand deposit was completed, the model piles were installed by pushing the piles into the sand deposit. Pile driving guides for different pile spacings were designed so that pile group can be tested with various spacing between the piles. The piles were aligned with the loading ram and LVDT measurement cores. All the model piles were closed ended at the tip and all were driven to the bottom of the sand deposit and rested on the base with an embedment length of 310mm. Thus the model represents full displacement end bearing hollow circular steel piles.

A study of model pile installation in sand at the 1 g. condition was conducted by Robinsky and Morrison(1964). The sand displacement and compaction around the piles were found to be dependent upon the pile property, soil stress level and soil density. It was found that the soil displacement envelope starts at 4.5 to 5.5 pile diameters below the ground level. For the laterally loaded pile tests, densification effects may be less significant since the pile behaviour is dominated by the soil reaction close to the soil surface where the disturbance is minimum.

The tests reported by Oldham(1984), Craig(1984,1985) show that the effects of stress level during the pile installation are important for axially loaded piles but much less important for laterally loaded piles. Therefore the procedure of installing piles at the normal stress condition and then performing test at a higher stress level was employed in this study.

After pile installation was completed, the loading system, including the double acting piston and load cell was mounted on the soil container lid. The appropriate loading connection was made depending on the test conditions. The LVDT cores were attached to the pile head to measure the corresponding deflections. The air pressure chamber was then installed and all the instruction wires were connected to the data acquisition systems.

6.4 SOIL LOADING AND PILE LOADING

After enclosing the whole testing device, the soil loading process was begun by applying air pressure in the air chamber and simultaneously increasing the water inflow to the sand container. The pore water pressure was continuously monitored and hydraulic gradient calculated. The soil loading was stopped on reaching the required gradient.

The air pressure inside the pressure chamber pushes back the loading ram increasing the lateral pulling force on the piles if the horizontal force is not balanced. A special air pressure control system was designed for this purpose.

The design of the pressure control system has to satisfy the following requirements

- During the soil loading process it should automatically balance the pressure acting on the loading ram so that the pile will stay in the same position.
- After the soil loading is completed the piles can be loaded very easily.

To satisfy the first requirement a double acting air-piston was employed whose piston area has exact same diameter as the loading ram connecting to the model pile. Thus when the air pressure, pressure supplied to the HGST chamber during the gravitation process, is connected to the air piston it balances out the force on the loading ram. When the air pressure in the chamber has reached the target air pressure, the air pressure in front of the double acting piston is reduced. The load is applied by increasing the air pressure in the back chamber of the two way piston.

6.5 PILE HEAD LOADING

The lateral load was applied to the model piles. The scanning rate of the data was 0.5Hz. All the tests in which load was applied to single pile were load controlled. The tests conducted with pile groups were displacement controlled. In the pile group the displacement of both the piles at the loading point was kept equal.

CHAPTER 7 : RESULTS AND DISCUSSION

7.1 INTRODUCTION

This chapter contains the test results of the lateral loading of model piles and pile groups under controlled laboratory environment using the HGST device. This study is aimed at evaluating the fundamentals of pile group response to static and cyclic lateral load under well controlled soil conditions.

7.2 Testing Series

The testing was conducted in three series of tests. In the SERIES I the single pile was installed and loaded horizontally. The load was applied in increments and displacements and bending moments were measured. The results of these tests were studied to confirm the response of the single pile. They were also used in the analysis of the pile group. In the SERIES II the pile group of two piles was installed and one pile in the pile group was loaded. In this series there are various cases depending on the direction of loading. In CASE I the pile is pushed towards the adjacent pile. The distance between two piles is varied from 2 to 6 diameters. In the CASE II the pile is pulled away from the adjacent pile and again distance is varied from 2 diameters to 6 diameters. In CASE III the angle of loading is changed. In this case the load is applied at an angle of 90 to the center line of the piles.

During the SERIES III testing both the piles in the pile group were loaded and the bending moments for the trailing pile were recorded. The pile spacing used was 2d and 4d. The terminology used in the figures includes series no., then case no. followed by spacing, e.g. S2C2S4 denotes that the figure is for series 2 (S2), case II (C2) and pile spacing is 4 diameters (S4). Similarly S3C0S4 denotes series 3 pile spacing of 4 diameters and since there is only one case the center portion denotes C0.

7.2.1 Repeatability of the test results

To check the repeatability of the test, three identical tests on the single pile under static lateral load were conducted. The results from these tests are given in figures 7.1 to 7.3.

Figure 7.1 shows load-displacement response of the single pile under lateral load. The figure shows response at the point of load application and at the ground level. The horizonal load is acting at a distance of 64.0mm above the ground surface(eccentricity = 64.0 mm). The results presented are for a hydraulic gradient of 60. It can be seen that the load-displacement response of the three tests are quite similar indicating repeatability of the test results.

The P-y curves for these three tests for the depth of 2 diameters below ground level are given in figure 7.2. The P-y curves are developed from the Bending Moment profile. The method of calculating the P-y curves from the bending moment profile was explained in CHAPTER 4. The P-y curves show some variation in the results. This is because the soil pressure in P-y curves is obtained by differentiating the bending moment profile as opposed to the displacement which is obtained by integrating the moment profile.

The bending moment and shear force profile for two different load cases is given in figure 7.3. It may be seen that the results from the three tests are in reasonable agreement.

In general, the tests on single piles show results that are generally reliable and repeatable. It should be noted that the accuracy of the test data and the program is highly dependent on the hydraulic gradient as well as the eccentricity of the loading point above the ground surface. The load-displacement curves shown in figure 7.1 is the actual data obtained while the P-y curves shown in figure 7.2 are developed from the bending moment profile shown in figure 7.3.

7.2.2 Series I (S1)

Single Pile testing results

The sand sample was prepared at a relative density of 75% and a hydraulic gradient of 60. The sample preparation technique and the pile installation method are described in the CHAPTER 4. After installation the single pile was subjected to the lateral

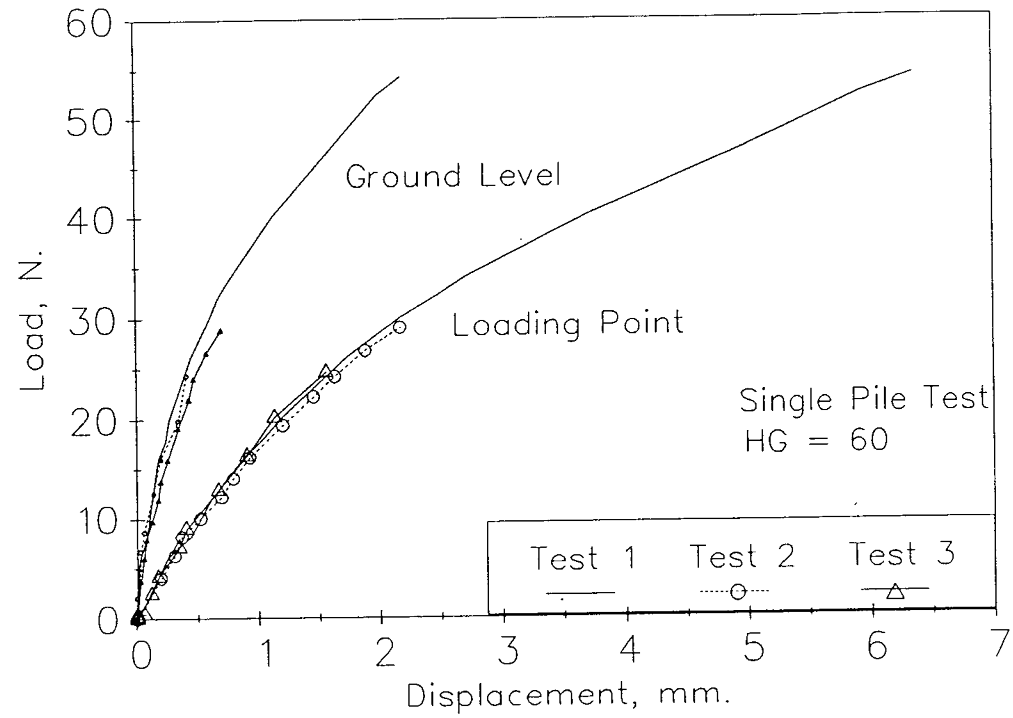


FIG. 7.1 Load Displacement Behaviour

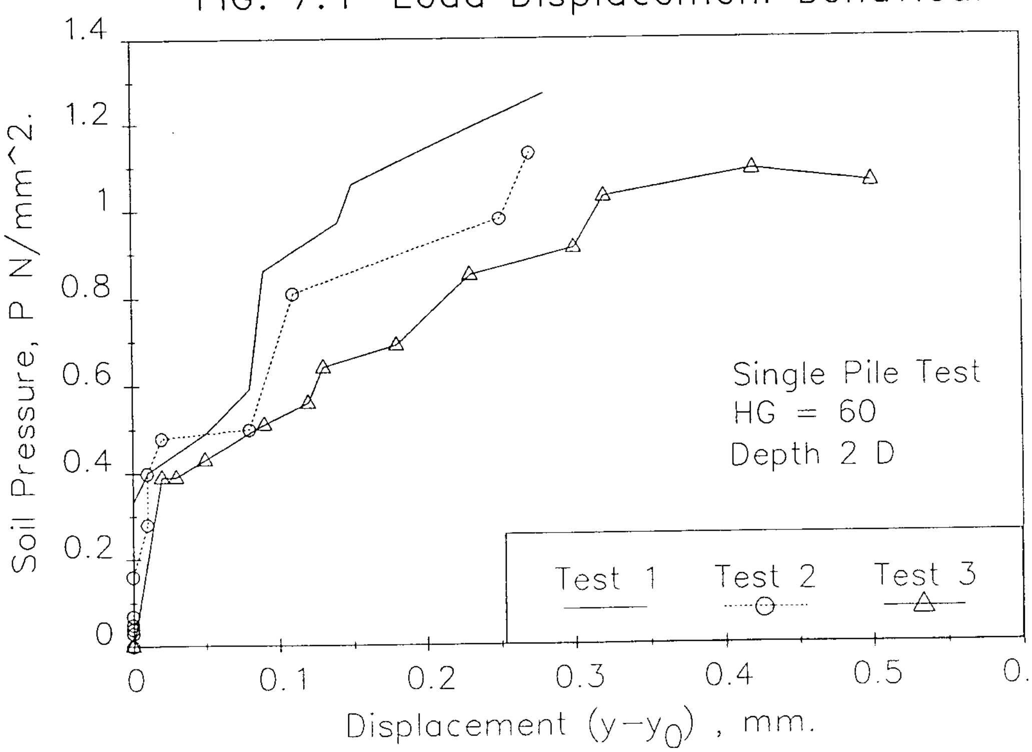


Figure 7.2 $P-(y-y_0)$ Curve

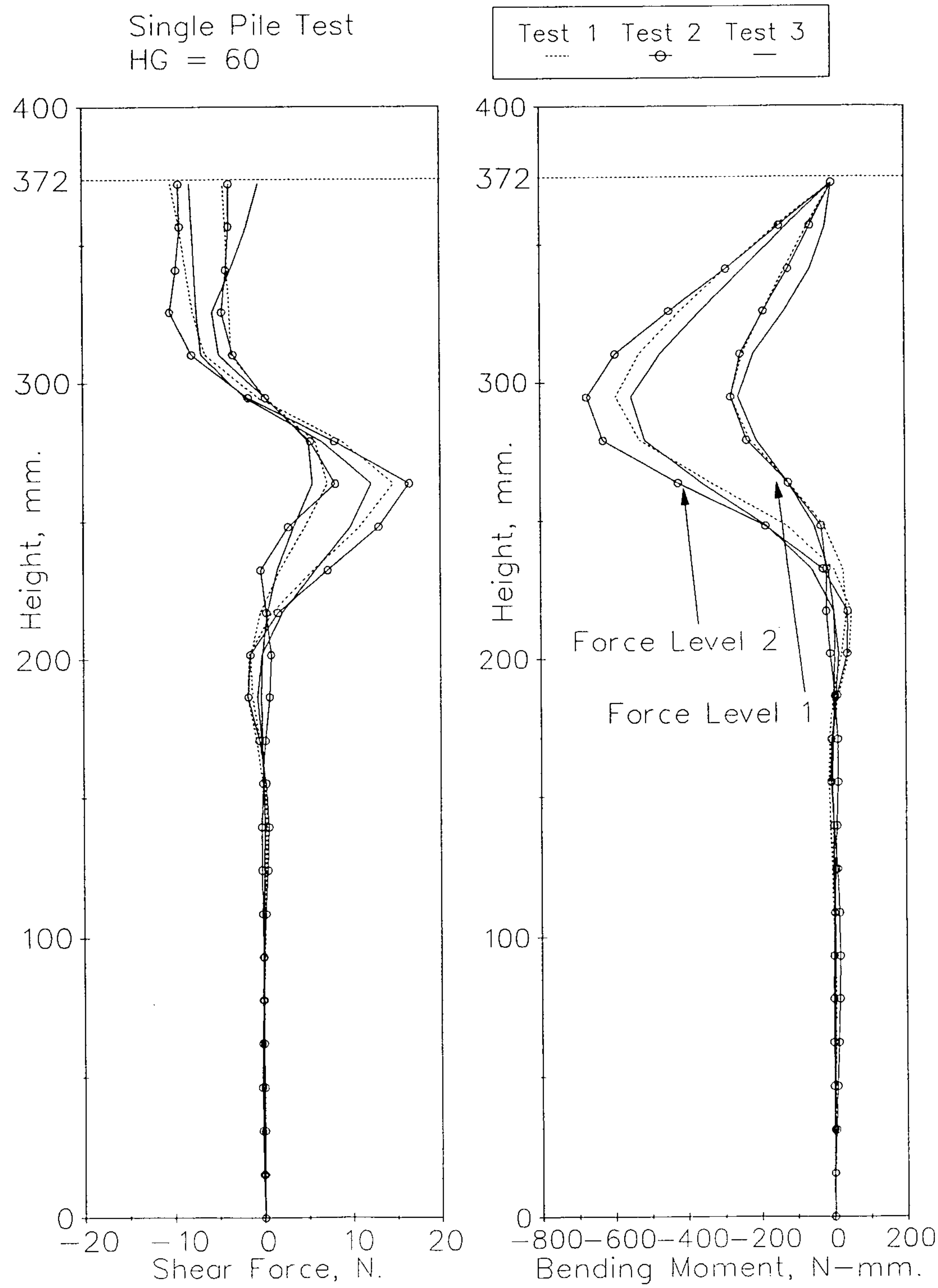


Figure 7.3 Bending Moment Profile - Single Pile

load, and the bending moment and displacements recorded.

7.2.2.1 LOAD DISPLACEMENT RESPONSE

The load-displacement curve of the single pile at the Ground level is given in figure 7.4. The characteristic load-displacement curve is non-linear with increasing deflections with the load. As the pile is loaded, with the increasing displacement a gap develops between the pile and the soil behind the pile. This gap increases as the load is increased. In the tests conducted, large displacements were observed under applied load. Due to the large displacements the test was stopped before the ultimate failure load was reached. This shows that in the tests conducted large displacements and soil failure occurred before pile failure, and thus is a major factor in deciding the pile capacity. For the structural capacity of the pile foundation soil failure is not a major factor. But large displacements play a major role due to accompanying high bending moments.

7.2.2.2 P-$(y-y_0)$ CURVE

Figure 7.5 shows the P vs $(y-y_0)$ curve for the lateral load test conducted on a single pile. The typical P vs $(y-y_0)$ curve is a nonlinear curve becoming asymptotic to the $(y-y_0)$ axis. The P vs $(y-y_0)$ curve is the characteristic curve for the given pile and the given soil. The figure shows P vs $(y-y_0)$ curves at

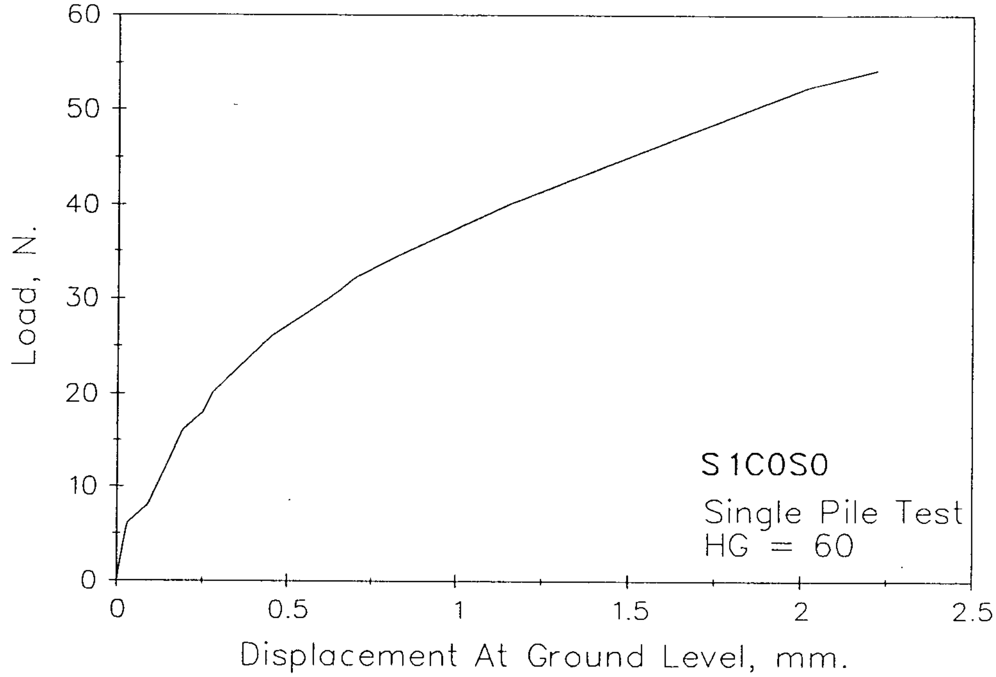


Figure 7.4 Load–Displacemnt Curve

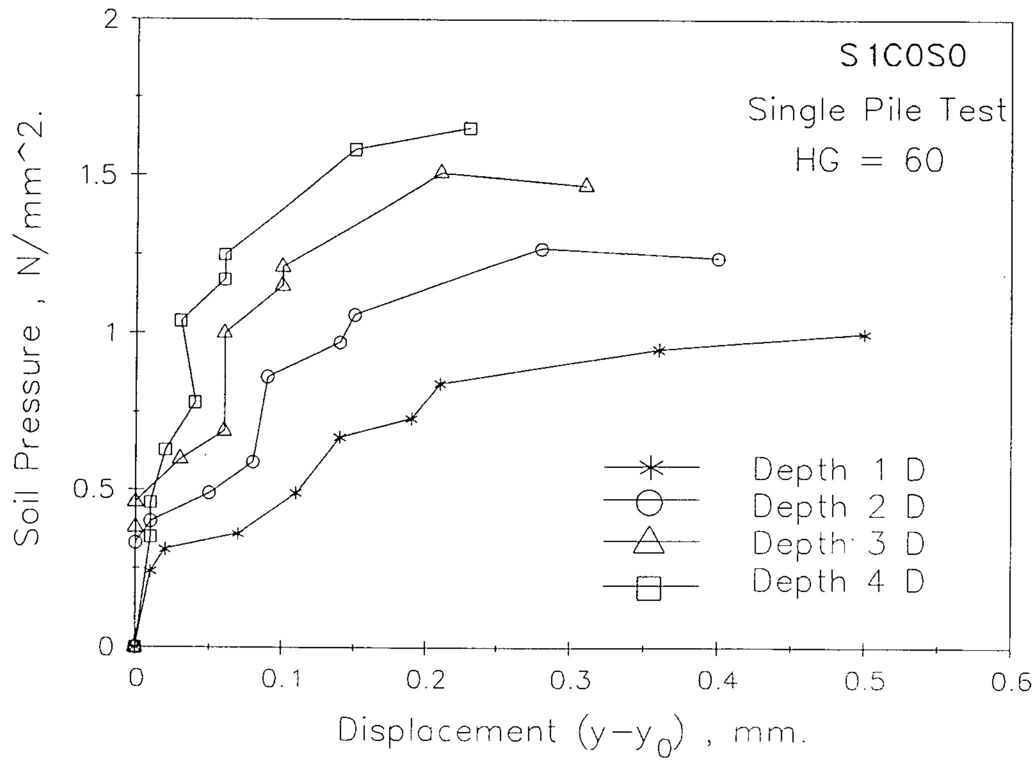


Figure 7.5 P–($y-y_0$) Curves

various depths. As seen in the figure the initial stiffness of the curves increases with the depth.

7.2.2.3 BENDING MOMENT AND SHEAR FORCE PROFILE

Figure 7.6 shows the shear force and bending moment profile along the length of the pile. The general bending moment profile obtained from the test is similar to the bending moment profile given by Davis & Poulos(1981) for a long flexible pile using the elastic solution. They suggested that the depth of the maximum bending moment developed is about 0.1d to 0.4d below the ground level for piles subjected to only horizontal load. In case of piles subjected to only bending moment, the maximum bending moment is at ground surface. The maximum bending moment observed in this test program was observed at a depth of about 0.1d. It should be noted that the pile was very long with L/d ratio of about 50 and the load was applied at an eccentricity of about 10d. This higher depth of the maximum bending moment is due to the fact that the pile is acting as a flexible pile as well as the load is applied at an eccentricity. As seen from the figure 7.6, the bottom half portion of the pile contributes very little towards sharing the load, most of the load is taken by the top half of the pile.

Also Figure 7.7 shows the soil pressure and pile deflection profile along the depth. As expected it can be seen that as the deflections increase the p-y curves near the surface reach the ultimate strength but at larger depths the p-y curves have yet to reach the ultimate strength. This shows that the local soil failure has taken place near the surface. It should be noted that despite the local failure taking place near the surface and large deflections, the pile has not yet reached its ultimate load capacity.

7.2.3 Series II (S2)

Pile Group Of Two Piles (One Pile Loaded)

Three Cases were considered in this series.

1. When the pile is pushed towards the adjacent pile (S2C1).
2. When the pile is pushed away from the adjacent pile(S2C2).
3. When the pile is loaded at right angle to the adjacent pile(S2C3).

In all three cases the tests were conducted at a hydraulic gradient of 60 and the sand sample was prepared at a relative density of 75%. In all cases three spacings were used for the pile group; 2d, 4d, 6d where d is the pile diameter. Only one pile was loaded in the pile group of two piles and henceforth the loaded pile will be referred to as PILE 1 and the adjacent unloaded pile will be referred to as PILE 2.

7.2.3.1 Load Displacement Response

Figure 7.8 shows the deflection of pile 2 vs load on pile 1. The deflection curves for pile 2 are given for different spacings; 2d, 4d and 6d. As seen in the figure the effect of

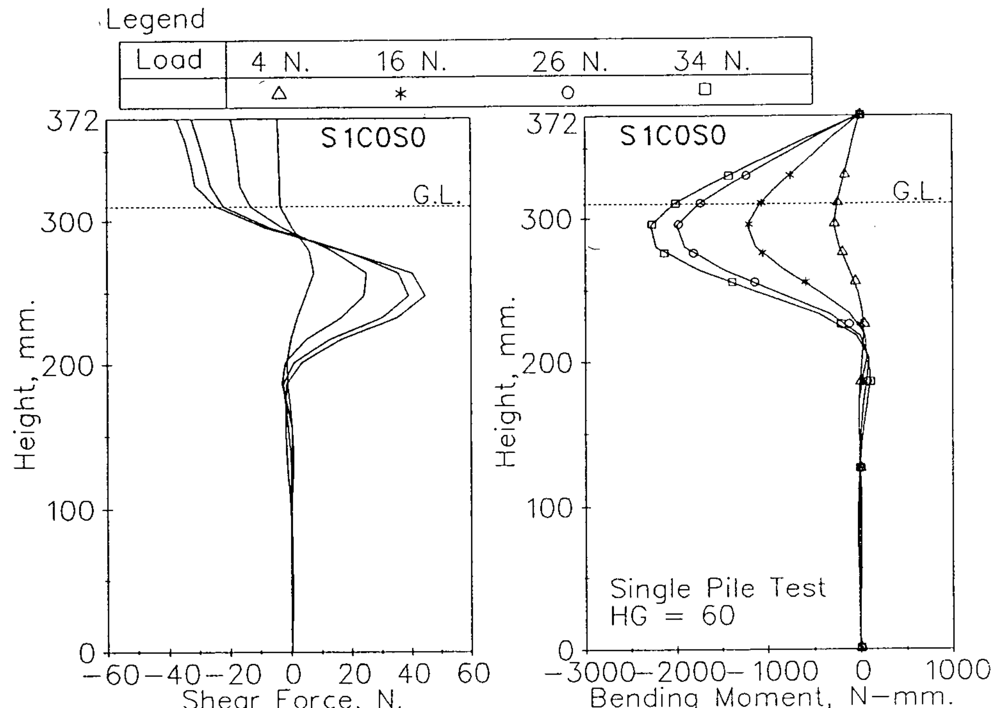


Figure 7.6 Bending Moment Profile - Single Pile

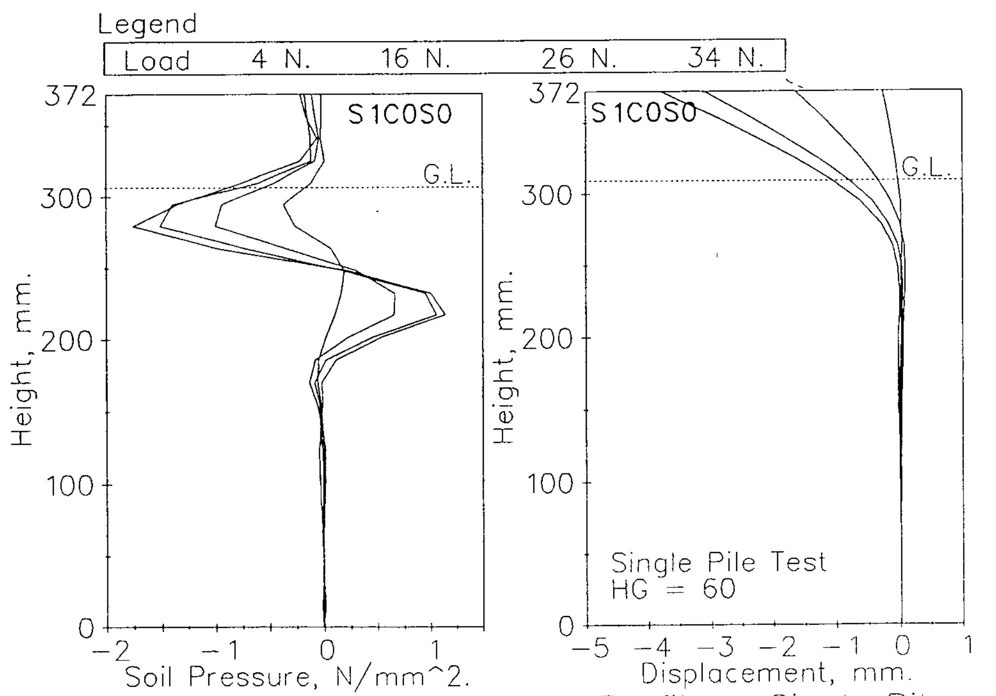


Figure 7.7 Bending Moment Profile - Single Pile

the load on the adjacent pile is nonlinear. Furthermore, the magnitude of this effect or interaction reduces with increasing spacing as expected. Davisson (1970) stated that if the spacing between two piles is more than 8d then the piles will have no interaction effects on each other. As seen from figure at spacing of 6d the interaction effect is small.

Figure 7.9 gives load displacement curves for the pile 1 at spacings 2d, 4d and 6d. It can be seen that the response of the pile becomes stiffer as the spacing between the piles is reduced. This in conjunction with figure 7.8 shows that part of the load applied to the pile 1 is shared by pile 2. This increased stiffness arises for two reasons; first, the soil adjacent to the loaded pile is stiffer due to driving of the unloaded pile and secondly, the unloaded pile itself has a stiffening effect on the loaded pile. As the distance between the piles increases the load transferred decreases as expected.

Figure 7.10 gives a comparison between the load displacement response of two piles and the response of the single pile. The spacing between the two piles is 2d. Figures 7.11 and 7.12 give curves for spacings of 4d and 6d, respectively. If we compare the displacement of pile 1 with the displacement of the single pile at the same load, as expected displacement of the single pile is larger than the displacement of pile 1. It can also be seen that the difference between the two reduces as the spacing between pile 1 and pile 2 is increased. In fact at spacing 6d at low load levels the load-

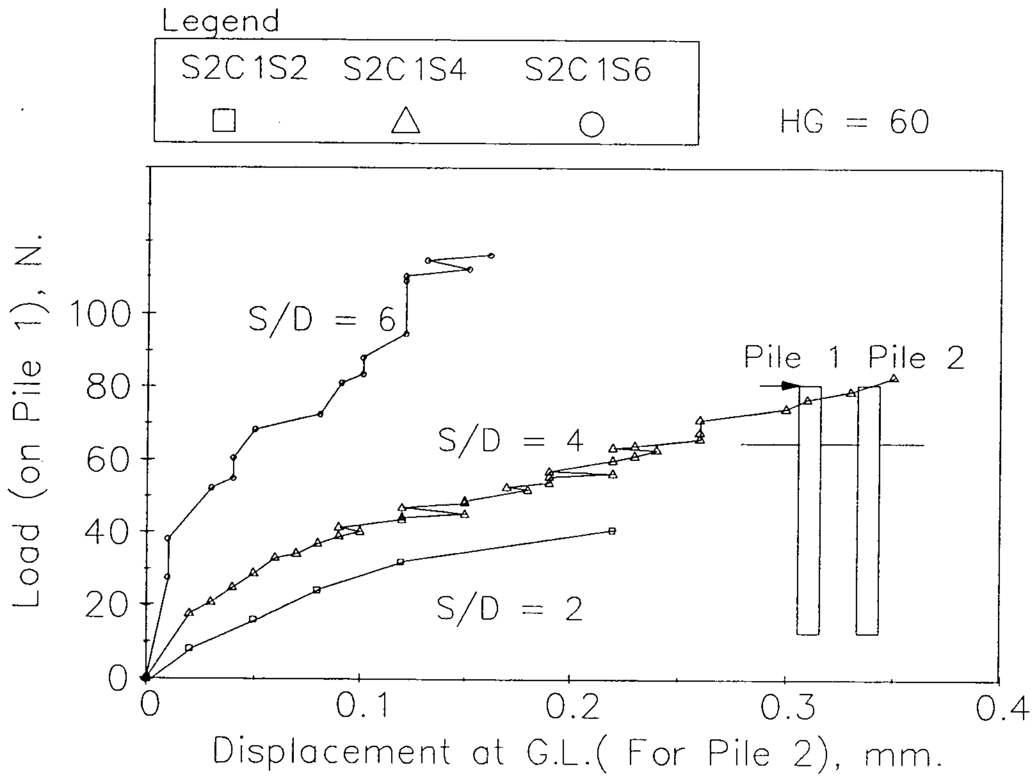


Figure 7.8 Load–Displacement Curves– Adjacent Pile

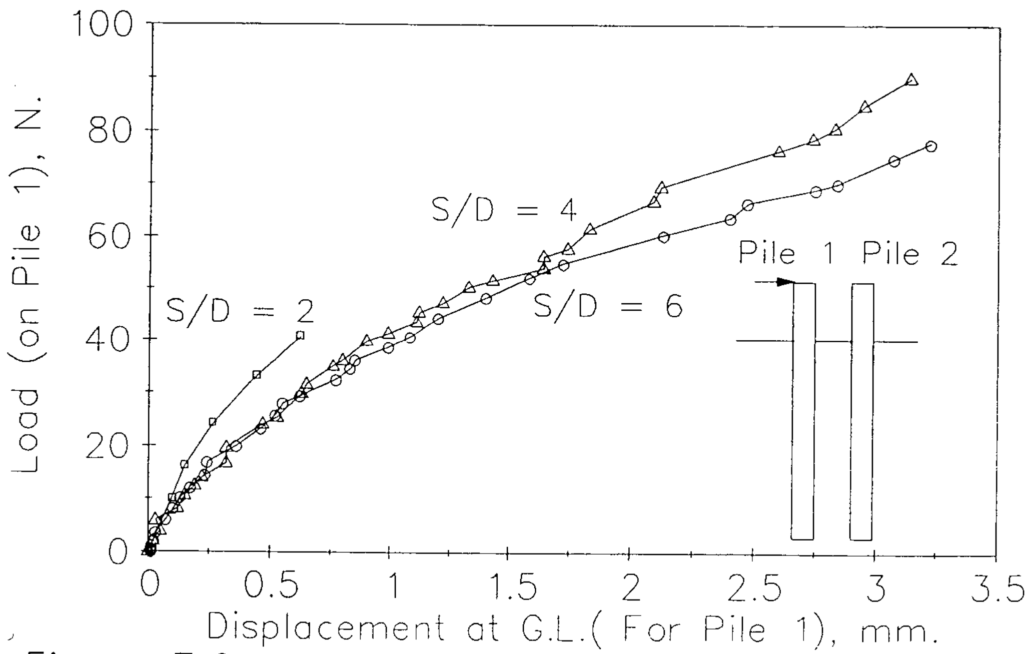


Figure 7.9 Load–Displacement Curves– Loaded Pile

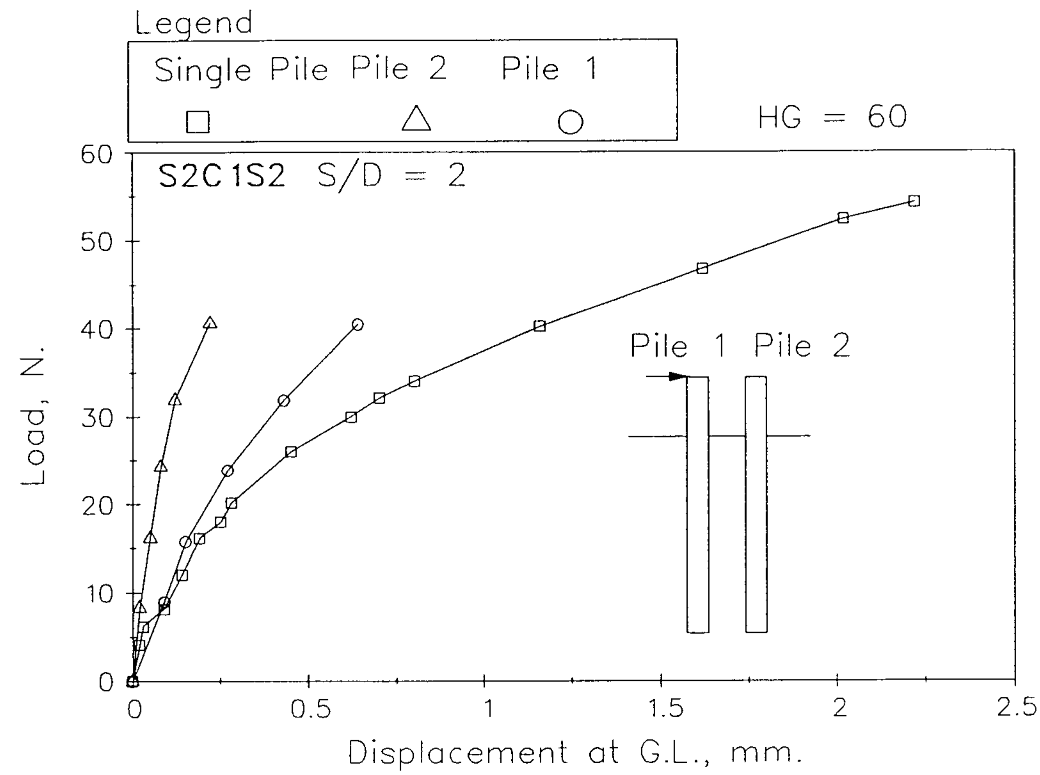


Figure 7.10 Load–Displacement Curves

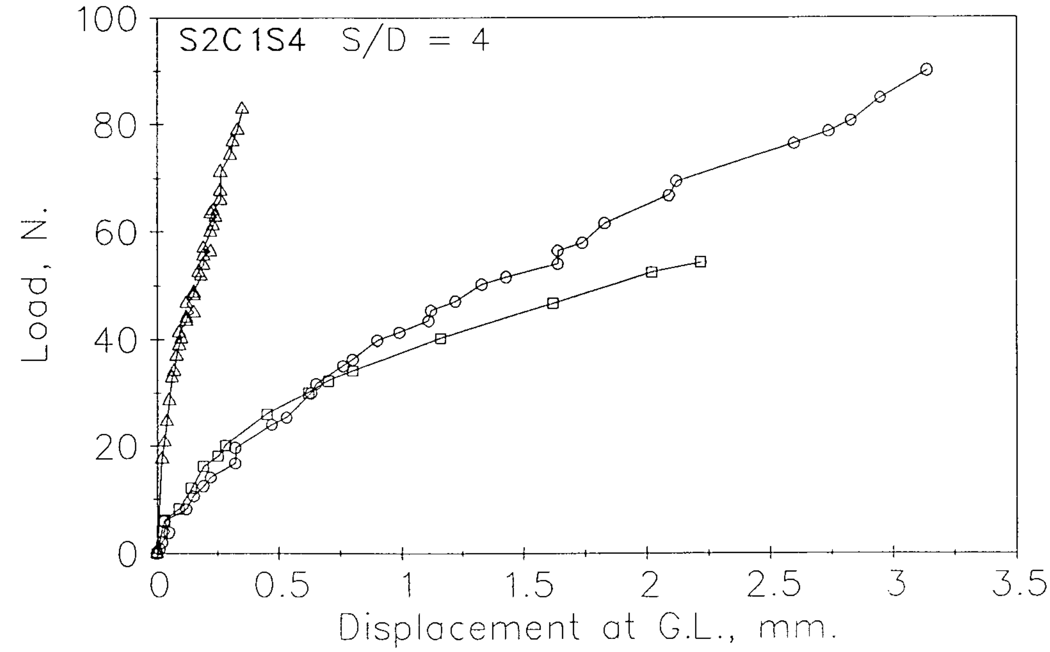


Figure 7.11 Load–Displacement Curves

displacement curves for the single pile and pile 1 are very similar and only at large loads is there a slight difference between the two. It should also be noted that this difference is very small compared to the difference at spacings 2d and 4d.

The comparison of the load-displacement curves for various spacings for PILE 1 is given in figure 7.13. The load-displacement curve for the single pile is also shown. It can be seen that the effect of adjacent pile presence reduces considerably at a distance of 4 to 6 diameters.

The interaction factors for the pile group of two piles were calculated by using the displacements of PILE 1 and PILE 2. These interaction factors were compared with the theoretical interaction factors obtained from the Elastic theory using Poulos's (1971) solutions. Also the interaction factors calculated from the solution given by Randolph(1981), Sharnouby and Novak (1986) are shown in the figure 7.14. It can be seen that the interaction factors calculated from Randolph's solution give the best approximation of the test data. The interaction factors calculated by the Sharnouby and Novak method overpredict the observed response as the spacing between the piles is increased. The calculation of interaction factors from the theory was explained in detail in Chapter 6.

7.2.3.2 Bending moment and shear force distribution

Figure 7.15 shows the experimental bending moment and shear force profiles for PILE 2 at a spacing of 2d for different load

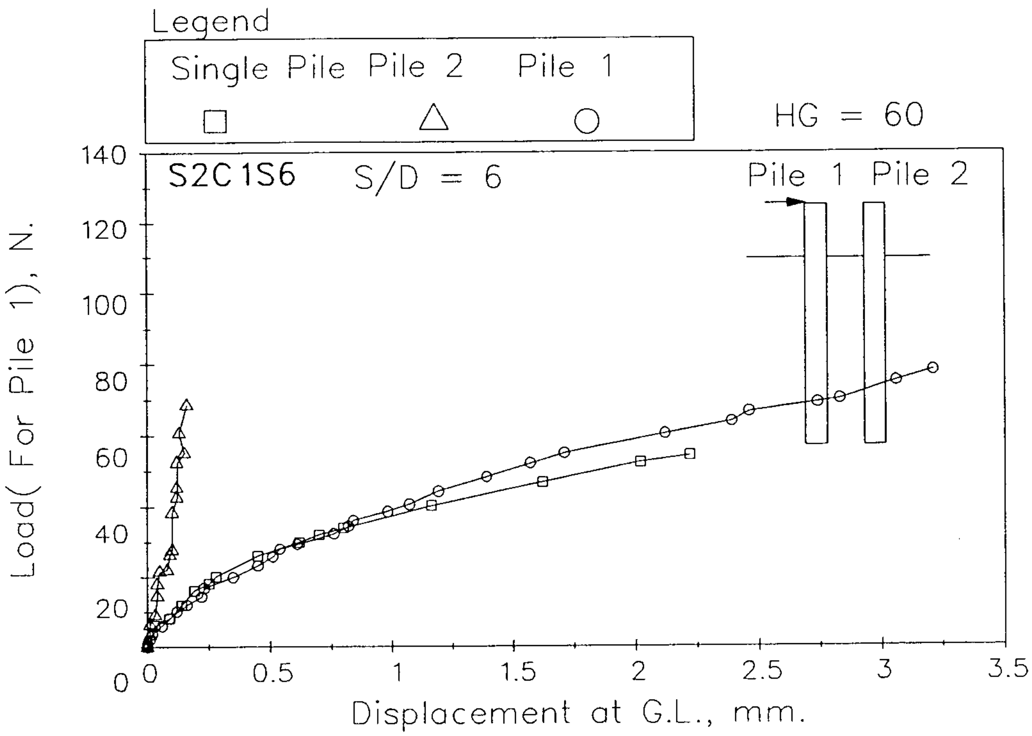


Figure 7.12 Load–Displacement Curves

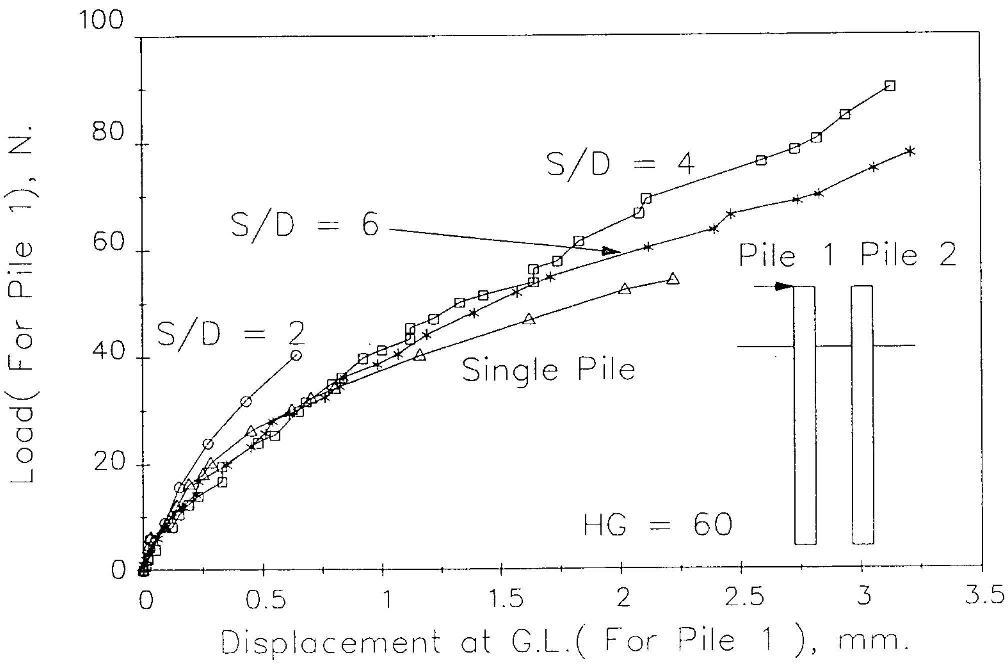


Figure 7.13 Load–Displacement Curves

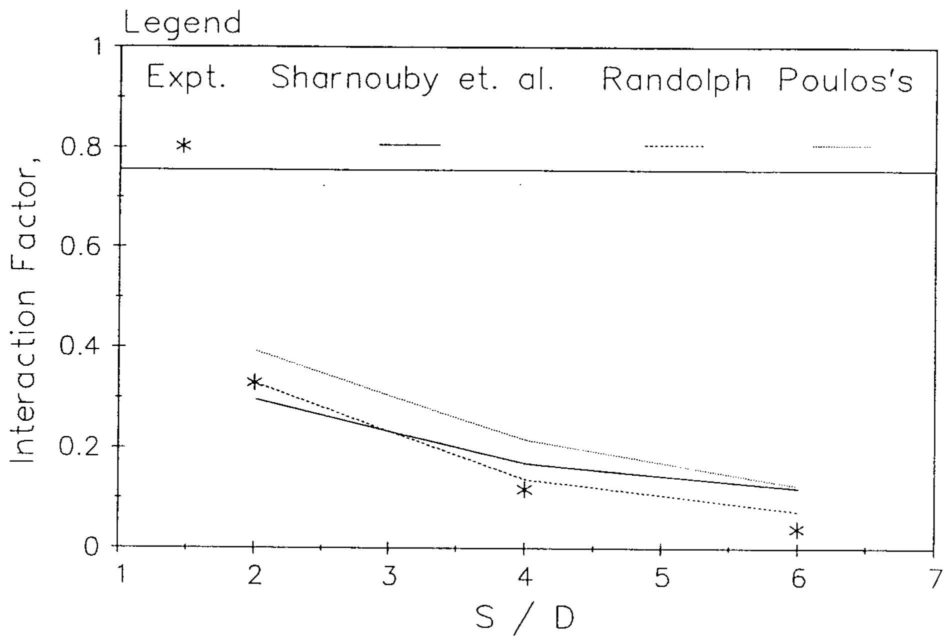


Figure 7.14 Comparison of Interaction Coefficient
NOTE :– The Interaction Factors are calculated by loading an adjacent pile.

steps. Figure 7.16 shows the profiles for PILE 2 at a spacing of 4d for various loading steps while Figure 7.17 shows the profiles at spacing of 6d for various loads. As the applied horizontal load on PILE 1 increases, the depth of maximum bending moment in PILE 2 also increases. If we compare the three profiles at different spacings we can observe that the maximum bending moment in PILE 2 reduces with the increasing distance and at a distance of 6d it is almost non-existent. Also the shear force in PILE 2 at a spacing of 4d is about 40 per cent of that at a spacing of 2d. Thus with the increasing spacing, the effect of load on adjacent pile, i.e. induced bending moment and shear force, reduce very significantly. At a spacing of 6d these induced stresses are about 10 per cent of the stresses at a spacing of 2d.

Figure 7.15 shows that the maximum shear force in PILE 2 occurs above the depth of maximum bending moment. It can be seen from figure 7.15 that at a spacing of 2d the shear force generated in PILE 2 is almost 20 per cent of the load applied on PILE 1 whereas at a spacing of 6d the shear force in PILE 2 is very small compared to the load acting on PILE 1 (less than 1 percent).

Figure 7.18 shows the bending moment and shear force profiles for PILE 1 at a spacing of 2d. Figure 7.19 and 7.20 shows bending moment and shear force profiles at spacings of 4 and 6d for PILE 1. From figure 7.18, it can be seen that as the spacing between PILE 1 and PILE 2 is increased although the

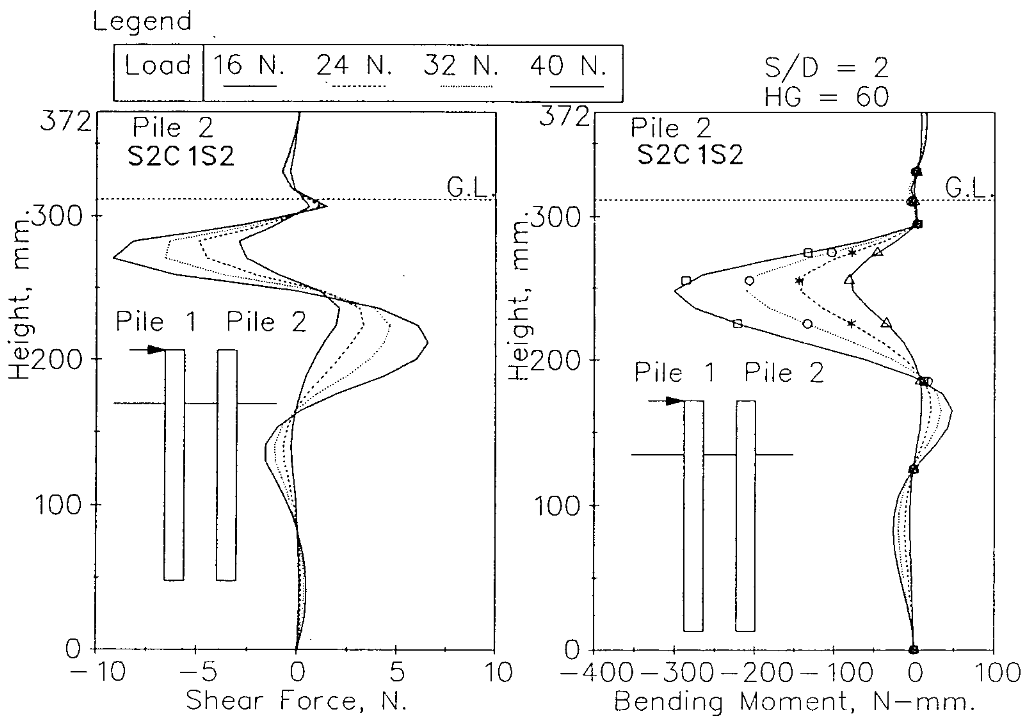


Figure 7.15 Bending Moment Profile

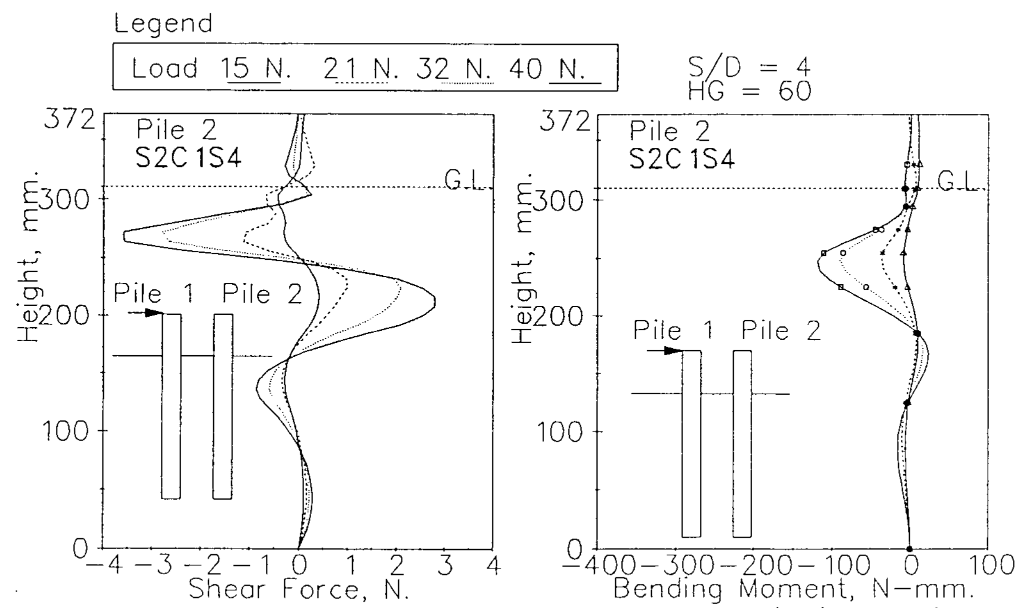


Figure 7.16 Bending Moment Profile (S/D = 4)

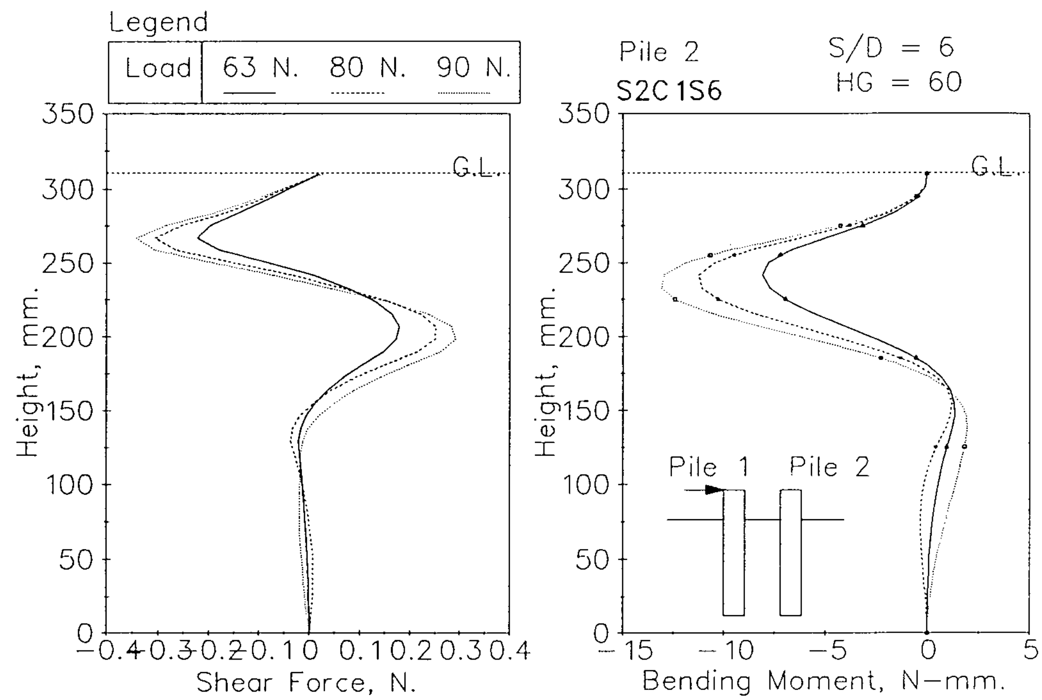


Figure 7.17 Bending Moment Profile (S/D = 6)

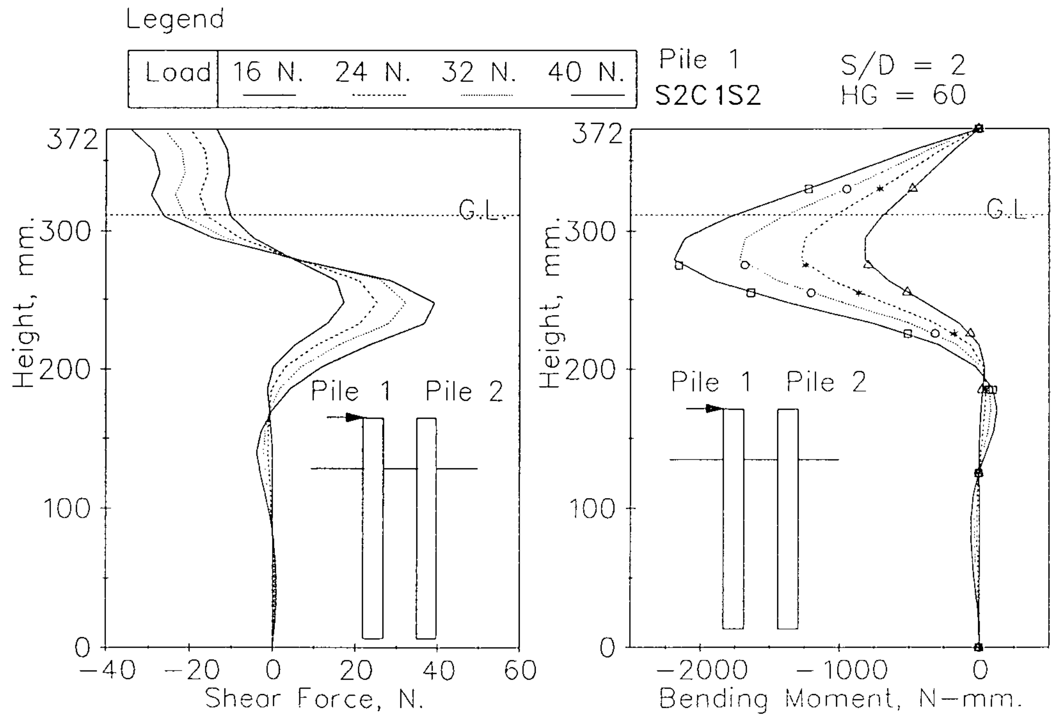


Figure 7.18 Bending Moment Profile (S/D = 2)

maximum bending moment did not change, the depth to the maximum bending moment reduced considerably. According to Poulos (1971), the maximum bending moment for the pile subjected to moment only is at the ground level. Thus as the ratio of the applied force to the applied moment decreases the depth to the maximum bending moment will start reducing. In short, the greater the eccentricity of the applied load, the closer the maximum bending moment is to the ground surface. In the tests conducted, as the spacing between the two piles was increased, the resultant stiffness of load-deflection response of PILE 1 was reduced. This is as expected as the stiffening effect of the adjacent piles arising from both densification and structural rigidity reduces with spacing. The depth of the maximum bending moment below ground surface decreased with the increasing spacing.

The shear force (SF) and bending moment(BM) profiles of PILE 1 and PILE 2 are compared in figure 7.21. It can be seen that at a distance of 2d the bending moment generated in PILE 2 is about 20 per cent of the bending moment in PILE 1. As seen from the figure, the value of the bending moment in PILE 2 is small compared to that in PILE 1 but the two bending moment profiles are very different.

Comparison of the profiles for single pile and PILE 1 for spacings 4d and 6d are given in figures 7.22 and 7.23 respectively. A large amount of reduction can be seen in the PILE 1 than the single pile. This can be contributed to the

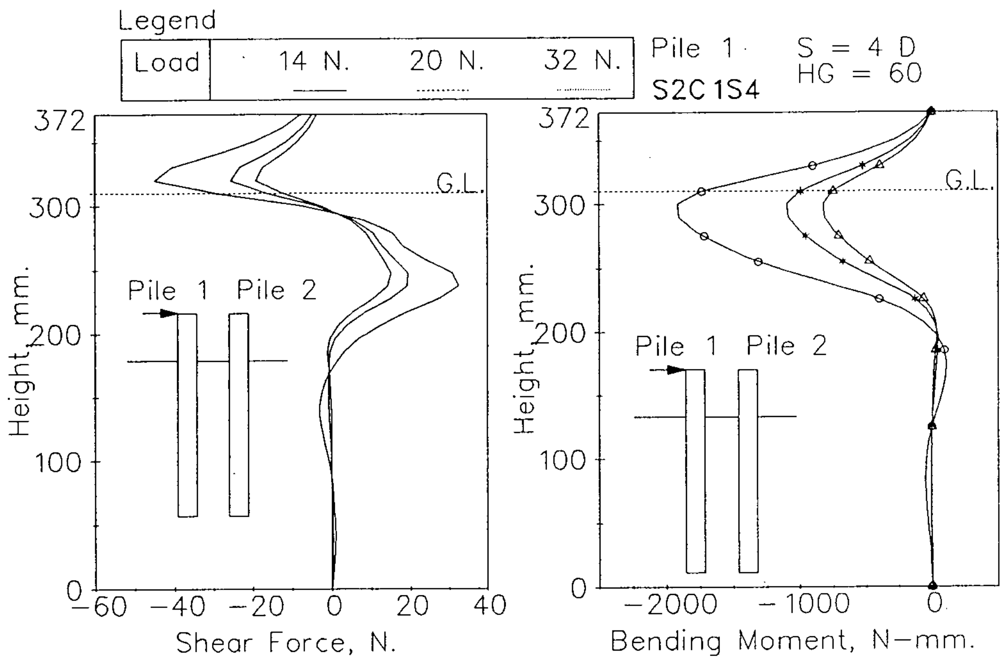


Figure 7.19 Bending Moment Profile (S/D = 4)

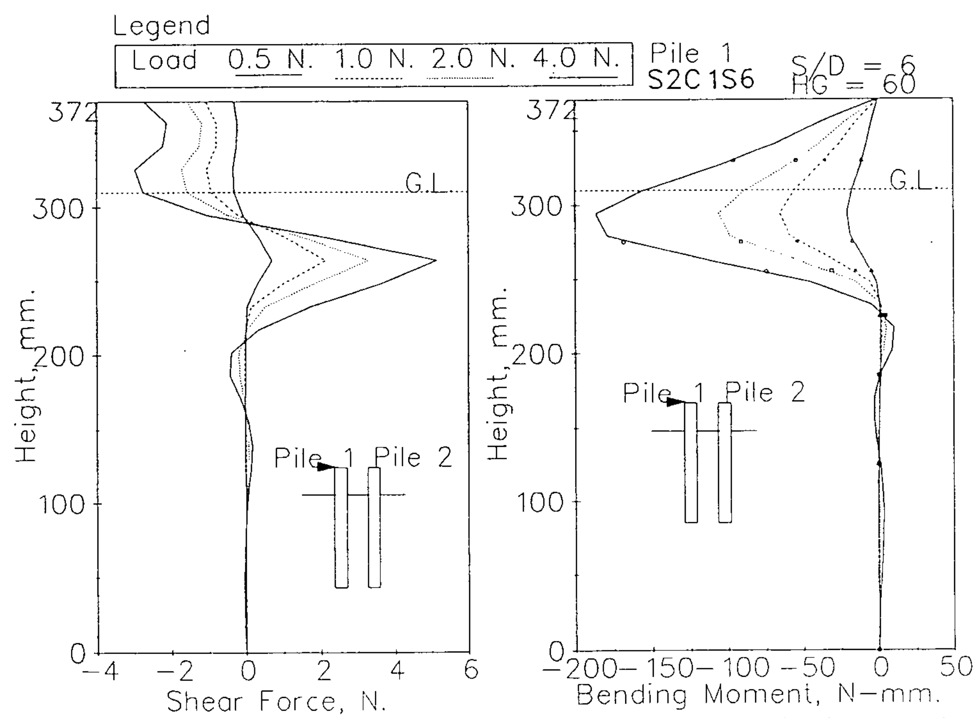


Figure 7.20 Bending Moment Profile (S/D = 6)

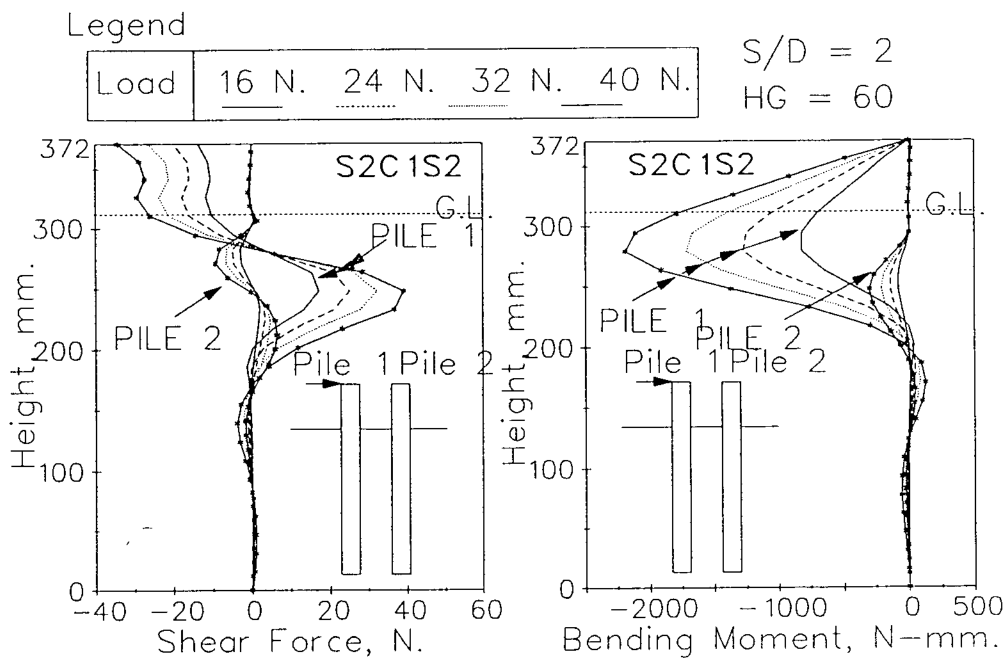


Figure 7.21 Comaprison Of Bending Moment Profiles

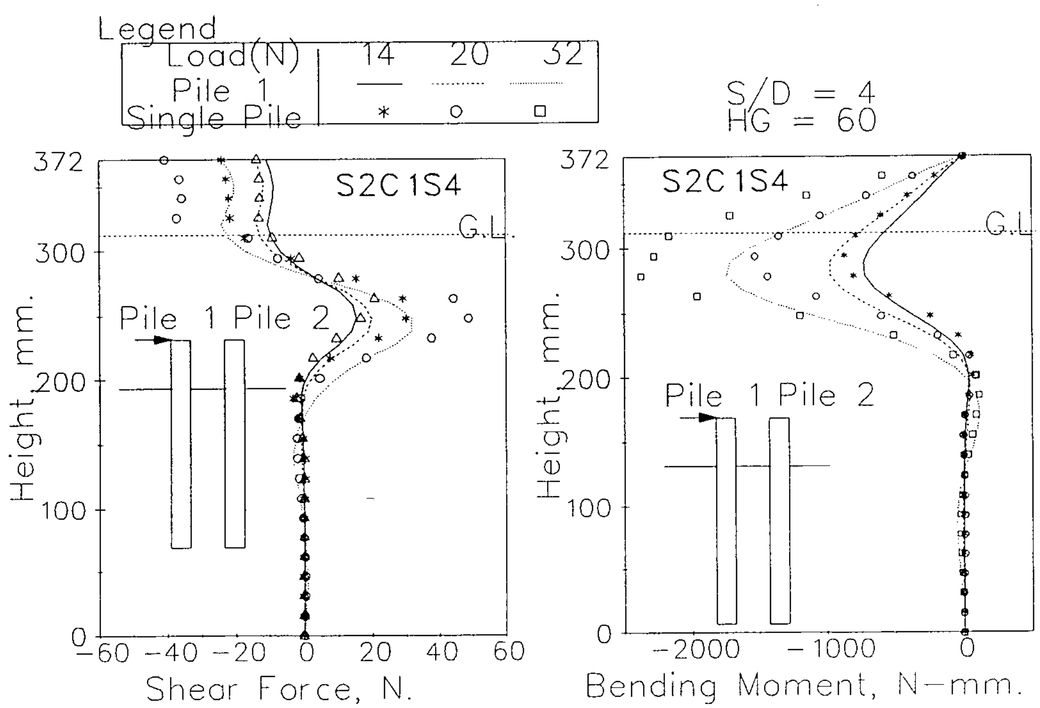


Figure 7.22 Comparison Of Bending Moment

presence of PILE 2. But as seen before, at a spacing of 6d, the PILE 2 is sharing far less bending moment as compared to a spacing of 4d and even in that case the reduction in bending moment is significant. The only reason for this reduction in bending moment can be an increase in the soil stiffness. This increase in stiffness is due to the densification during the pile installation.

Figure 7.24 shows the deflection and soil stress profiles for PILE 2 at a spacing of 2d. The induced deflection profiles for spacings 4d and 6d are given in figures 7.25 and 7.26 respectively. Figures 7.24 to 7.26 show that as the spacing between the piles is increased the induced deflection reduces considerably. During the pile group test program conducted by Schmidt(1981), he conducted a study of the effect of the induced displacement on pile response. He used a two pile group previously subjected to cyclic loading and applied load on one pile and measured the displacements and moments in the other pile. He observed that the induced displacement has no relationship with the pile head response but as the displacements increased the maximum bending moment in the pile increased. It was also observed that the induced displacement increases with decreasing spacing. Thus closer the piles are, more the induced displacement due to the pile group action.

The displacement profiles for the Pile 1 at a spacing of 2d are given in figure 7.27. Figures 7.28 and 7.29 give the displacement profiles of Pile 1 at a spacing of 4d and 6d. If

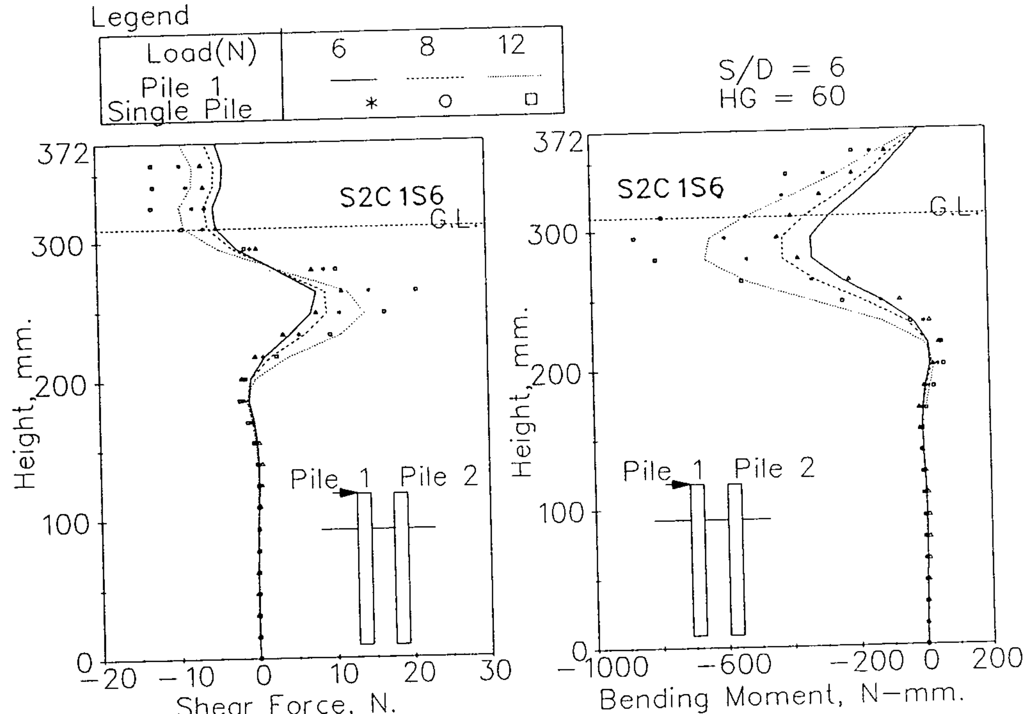


Figure 7.23 Comparison Of Bending Moment

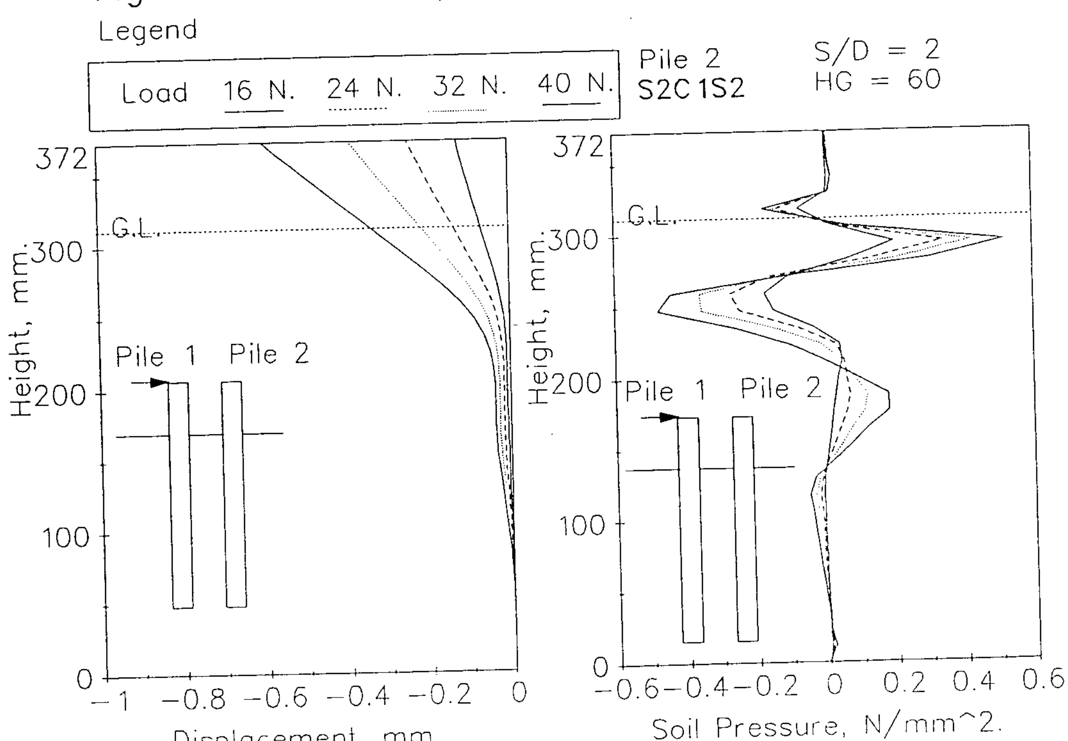


Figure 7.24 Deflection Profile (S/D = 2)

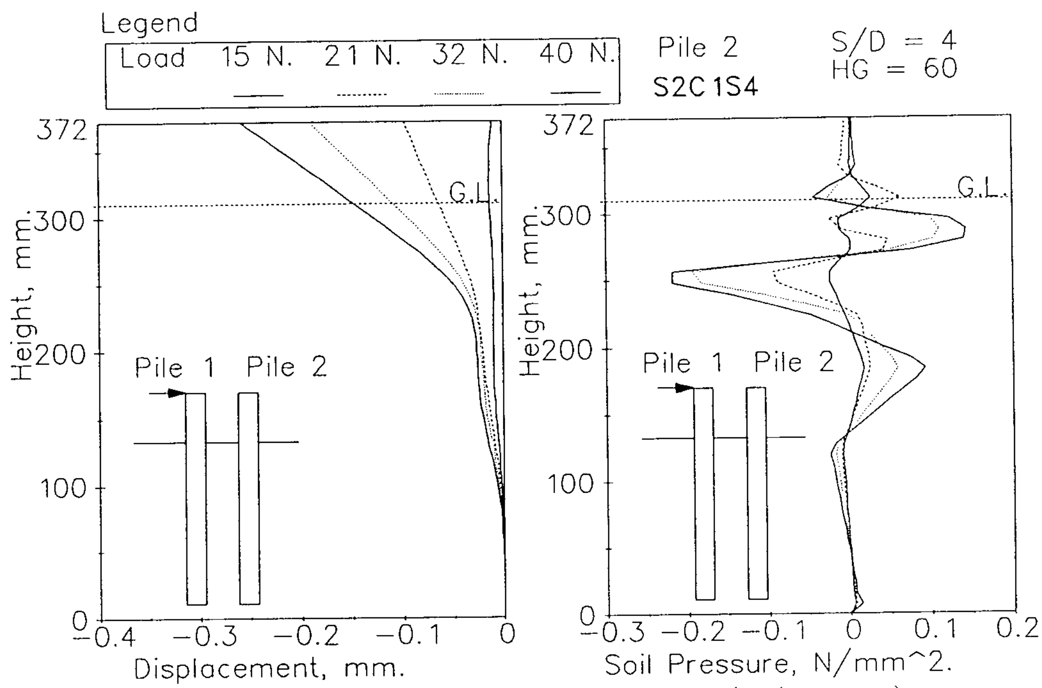


Figure 7.25 Deflection Profile (S/D = 4)

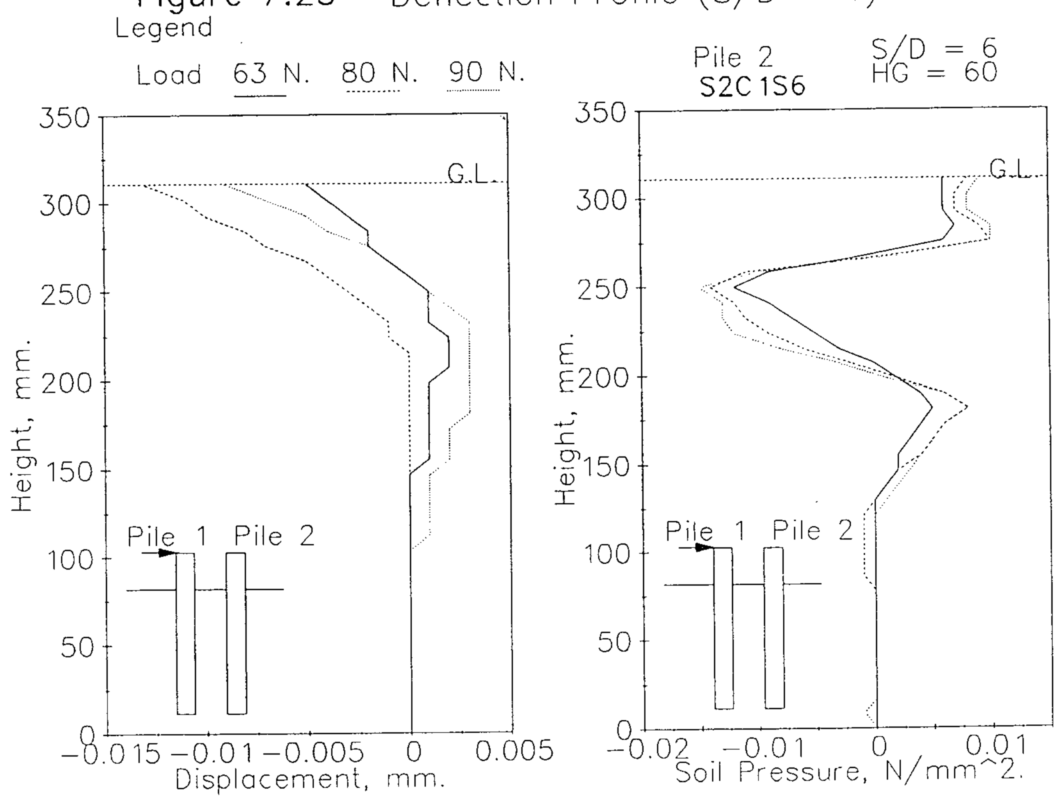


Figure 7.26 Deflection Profile (S/D = 6)

we compare these displacement profiles with the displacement profiles of the single pile at same load, we confirm that at the same load displacement of Pile 1 is smaller than that of the single pile. Thus due to the presence of the adjacent pile the load-deflection response of the Pile 1 is stiffer than that of the single pile. On closer observation, we notice that the displacement at of the pile at the loading point is reduced in far greater amount than at the ground level. Similarly observations can be made in other cases, although this reduction decreases with increasing spacing.

7.2.3.3 P-(y-y_0) CURVE

Figure 7.30 shows P-(y-y_0) curves for PILE 1 at a spacing of 2d. The curves are shown for various depths. These curves show increasing stiffness with increasing depth similar to the single pile response. But comparing the two responses we find that at a spacing of 2d the response of PILE 1 is softer than that of the single pile. Figure 7.31 gives the P-(y-y_0) curves for spacing of 4d while figure 7.32 gives P-(y-y_0) curves for a spacing of 6d. If we compare these responses with that of the single pile we observe that the response of the PILE 1 at a larger spacing is stiffer than that of a single pile.

At a small spacing of 2d, the resistance to the applied load is shared by Pile 1 , Pile 2 and soil. This fact is also shown by the bending moment profiles. In the case of a larger spacing, most of the resistance is shared by only Pile 1 and the

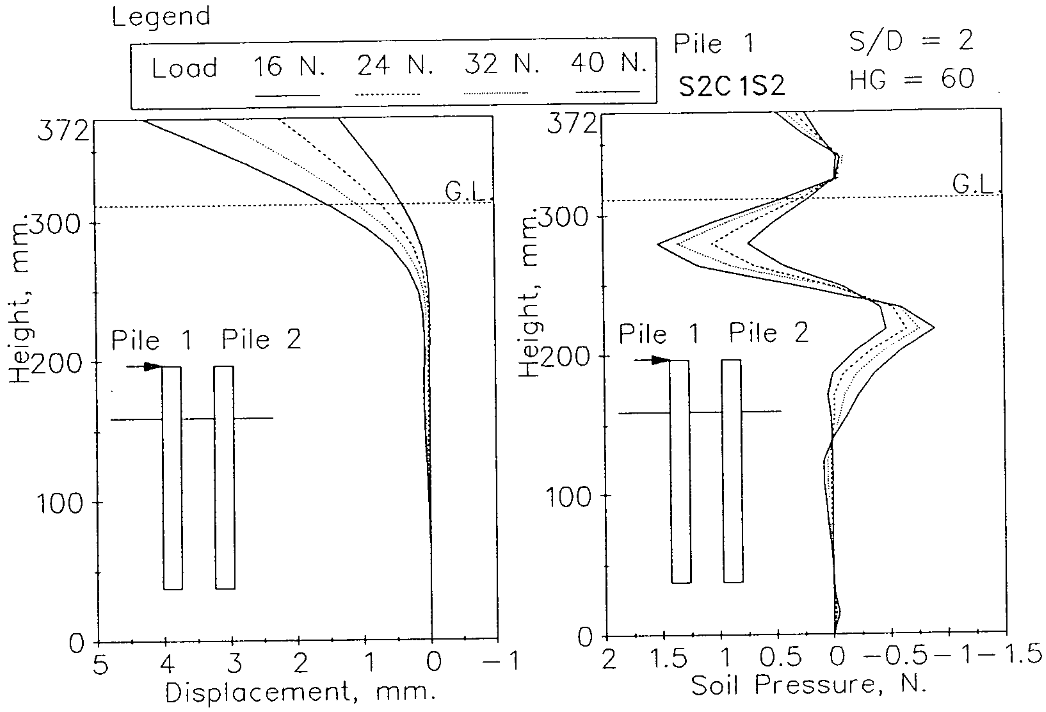


Figure 7.27 Displacement Profile (S/D = 2)

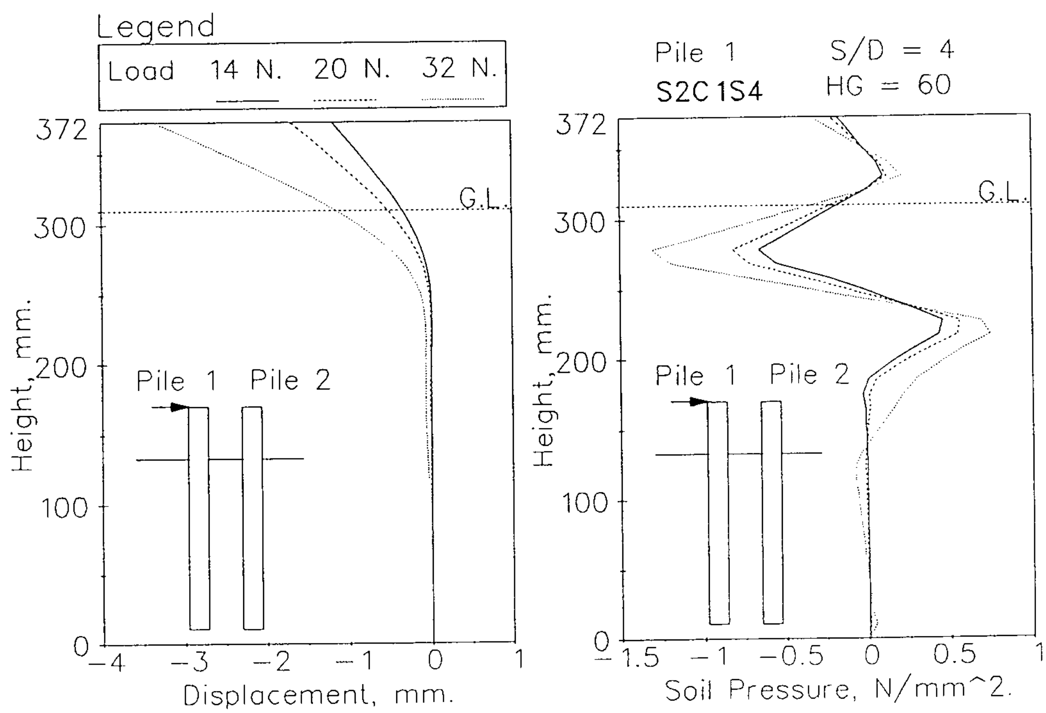


Figure 7.28 Deflection Profile (S/D = 4)

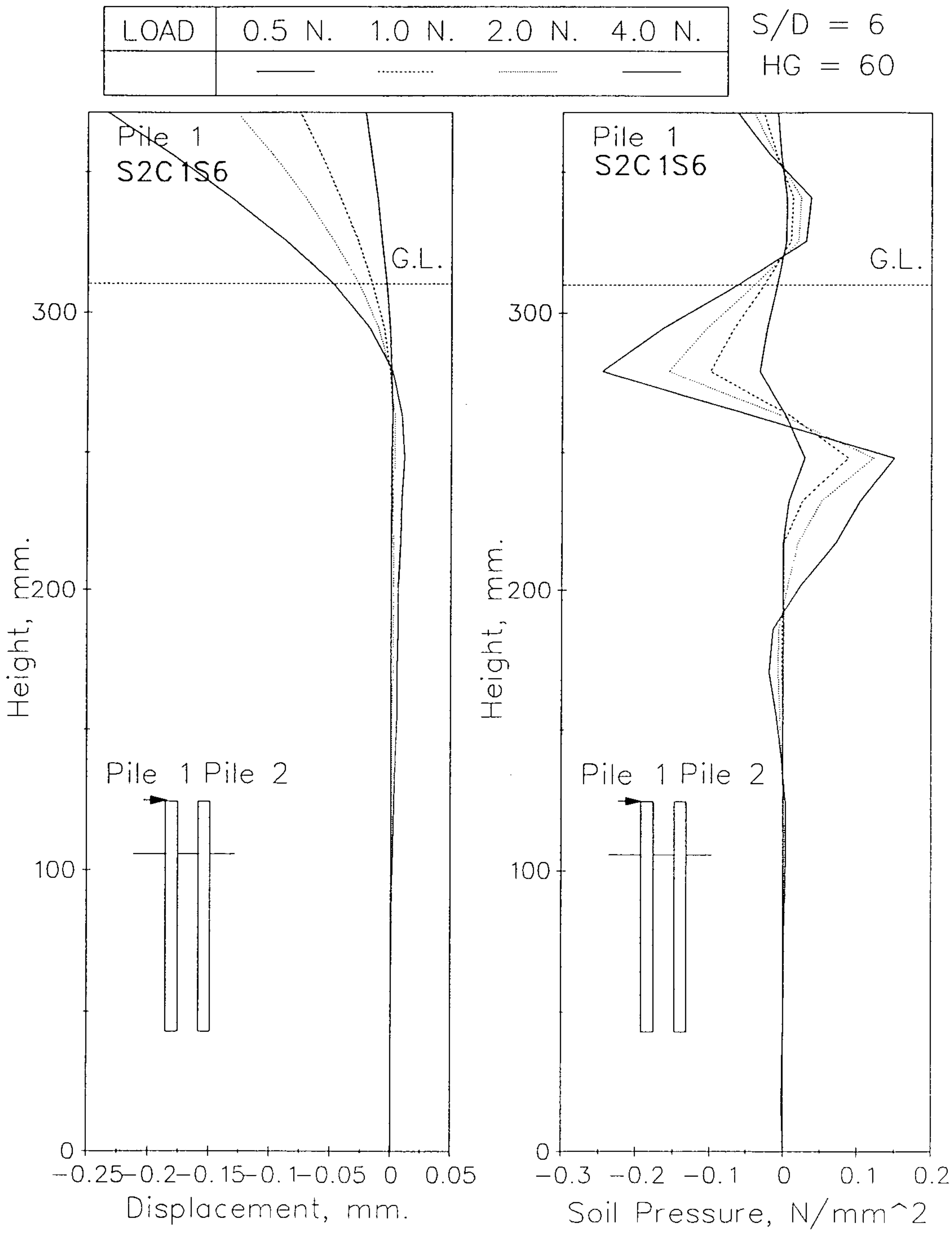


Figure 7.29 Deflection Profile (S/D = 6)

soil. The stiffer P-y curves imply that the soil is densified during pile installation. This leads to the conclusion that at the small spacing effect of densification is small compared to the adjacent pile stiffness while at larger spacing since the adjacent pile stiffness is negligible the effect of densification is very prominent.

7.2.3.4 CASE 2 (β = 90°)

Figure 7.33 compares the load-displacement response of PILE 1 with that of the single pile. The PILE 1 response is for CASE 2 in which the angle between the loading direction and the line joining the pile centres is 90°. The response of the pile is very similar to that of the single pile at smaller loads. At higher loads the PILE 1 response is stiffer due to the compaction of the soil during the installation of the two piles.

Figure 7.34 compares the bending moments in the two piles for two load cases. As seen from the figure PILE 2 registered zero bending moment. The spacing of the piles was 2d. At spacings 4d and 6d similar results were obtained. These shows that the presence of adjacent pile has no direct effect on the single pile response.

7.2.3.5 CASE 3 (β = 180°)

In this case, the load-displacement of PILE 1 was similar

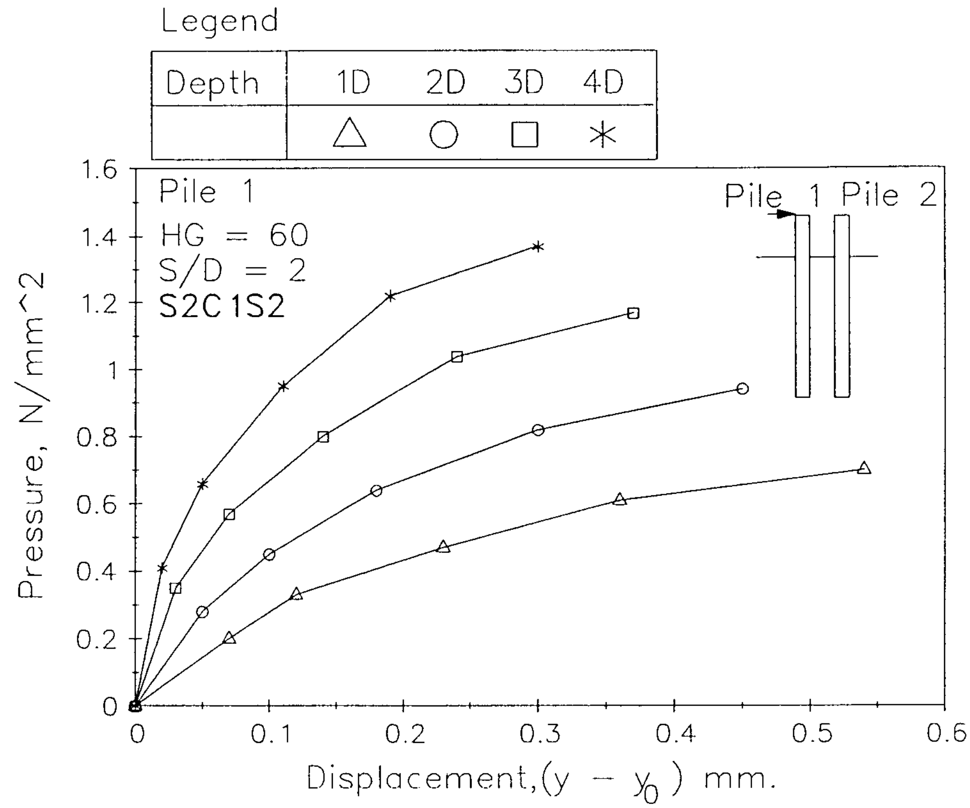


Figure 7.30 P − y Curves (S/D = 2)

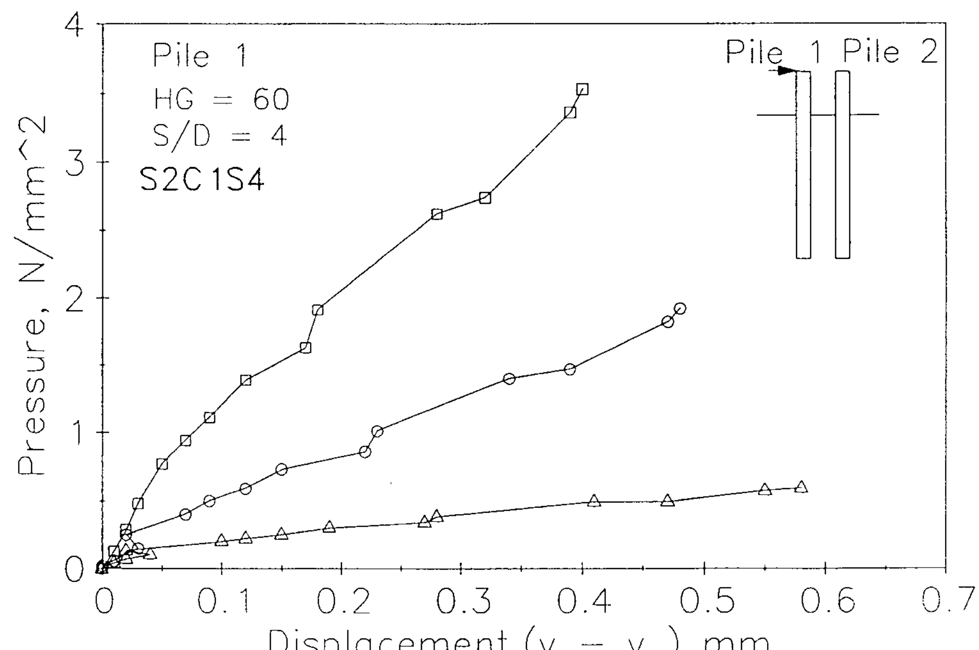


Figure 7.31 P − y Curves (S/D = 4)

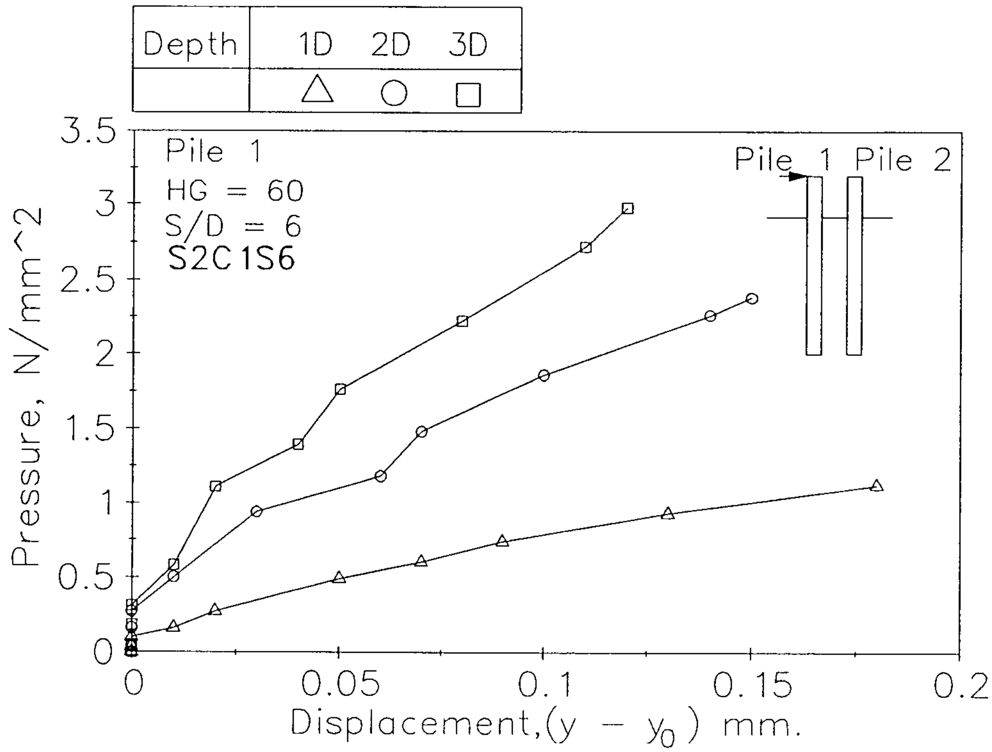


Figure 7.32 P − y Curves (S/D = 6)

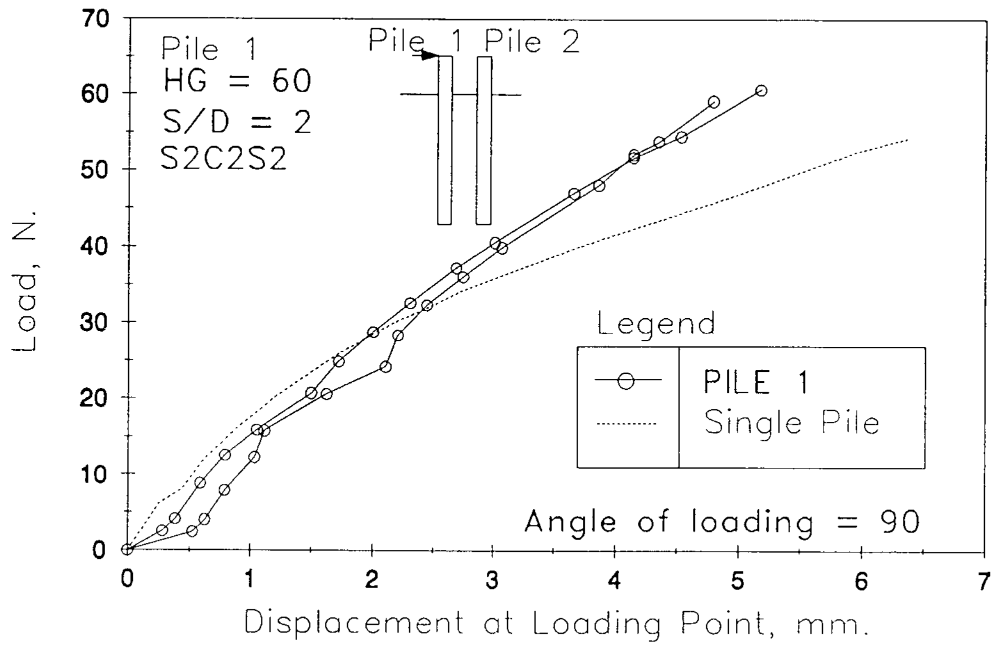


Figure 7.33 Load−Displacement Curve

to that of the single pile. Furthermore PILE 2 registered no effect, neither displacements nor bending moments, due to the load applied on PILE 1. The comparison of the bending moment profiles at two different loads is shown in figure 7.35. While in case 1 the soil between the two piles was densified due to the pile installation, in this the soil in front of the PILE 1 showed no effects due to the installation of PILE 2 behind PILE 1.

7.2.4 SERIES III (S3)

In this series both the piles in the pile group are loaded simultaneously. The distance between the piles was varied from 2d to 4d. The response of the pile was compared to both the single pile as well as the single pile with the adjacent pile. The two piles in the pile group were installed simultaneously by pushing them in the same direction simultaneously. A special pile guide was developed for this purpose.

7.2.4.1 Load Displacement

Figure 7.36 shows the response of a group of two piles at a spacing of 2d to horizontal load applied at an eccentricity. During the load application, the displacement of the two piles was kept equal by a rigid connection between the two piles. The response of two piles is also shown separately in the same figure. The spacing between the two piles in the pile group for

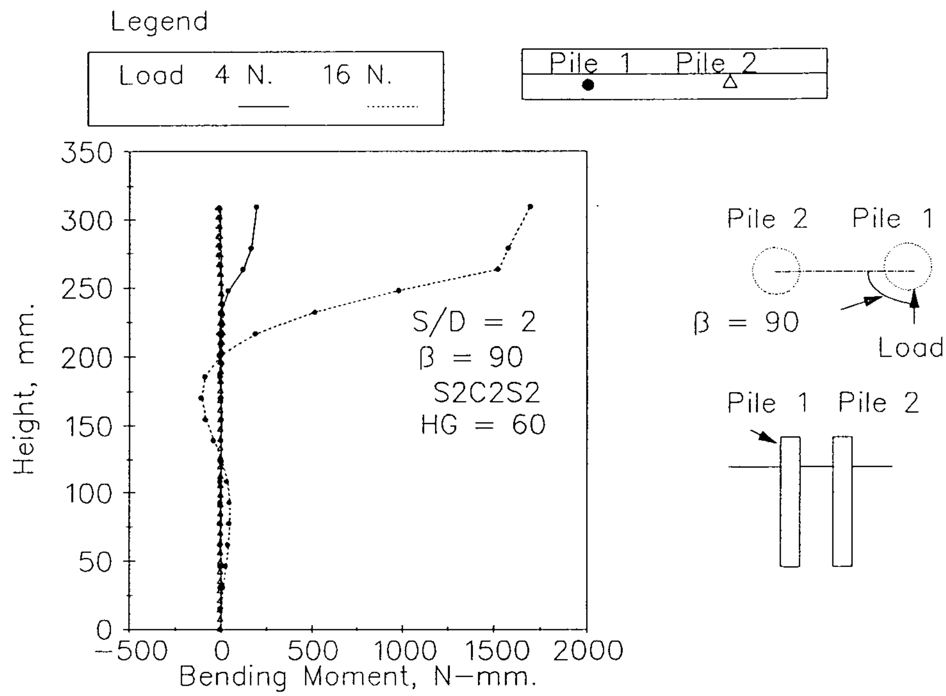


Figure 7.34 Bending Moment Profile

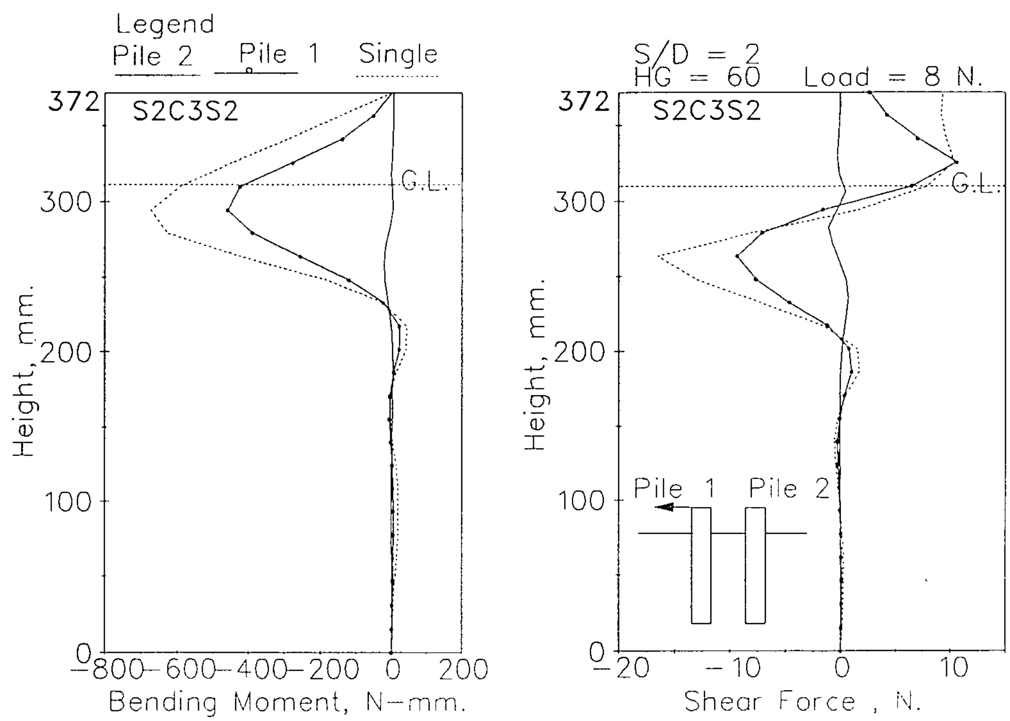


Figure 7.35 Bending Moment Comparison

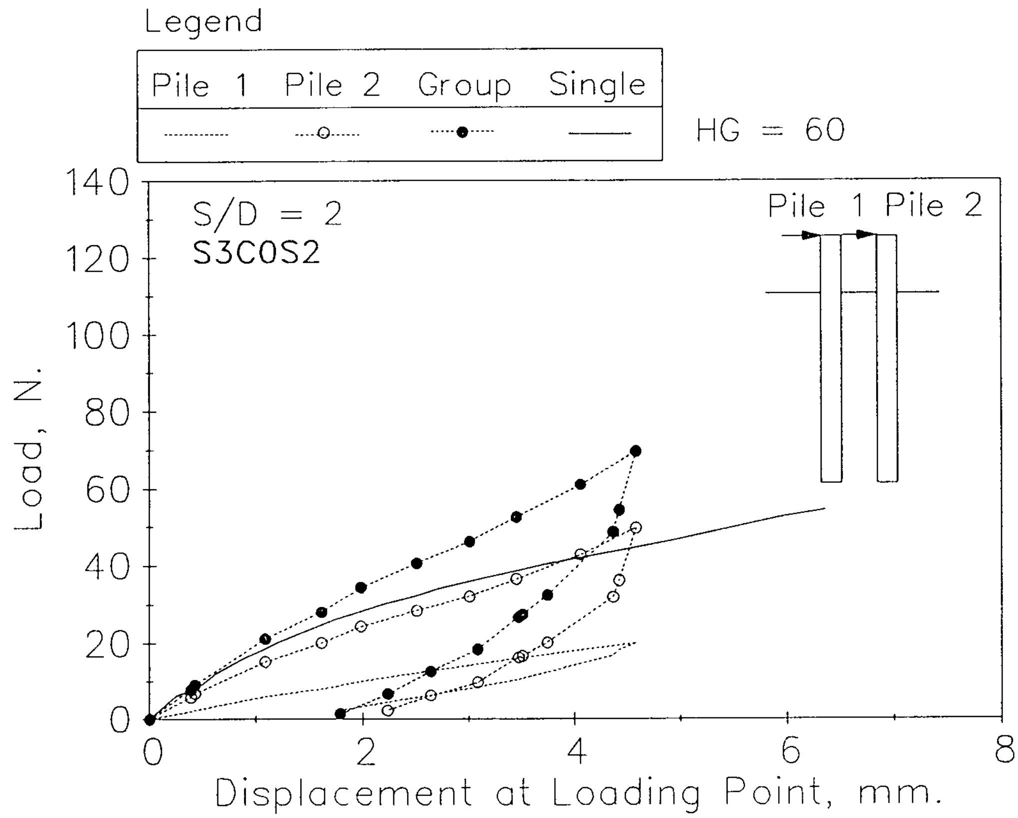


Figure 7.36 Load–Displacement Curve (S/D = 2)

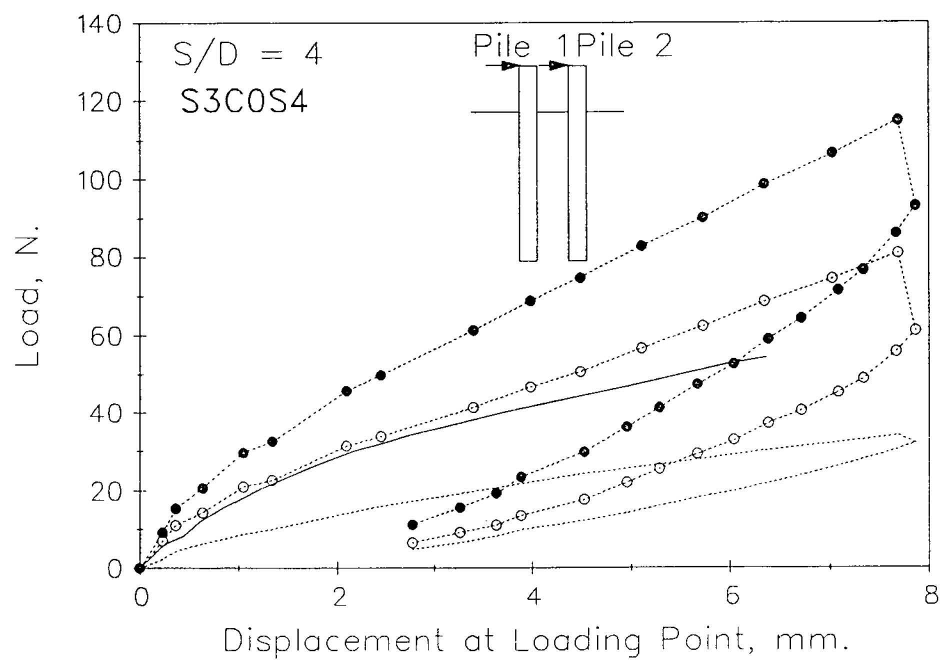


Figure 7.37 Load–Displacement Curve (S/D = 4)

this test was 2d. The response of a single pile subjected to lateral loads is also shown in figure 7.36. As seen from the figure, the response of PILE 1 is softer than the response of the single pile. Response of PILE 2 is much similar to that of the single pile. It should be noted that most of the load applied to the pile group is taken by the PILE 2 (the lead Pile).

Figure 7.37 shows the response of the pile group with spacing of 4d. It also shows the response of the individual piles. These responses are compared with the response of the single pile. Similar to the 2d spacing, the PILE 2 carries most of the load applied to the pile group, but in this case the percentage of the load carried by PILE 1 is more than the previous case. Also the efficiency of the pile group is more for a spacing of 4d than the spacing of 2d. The efficiency of the pile group is defined as the ratio of the maximum load carried by the pile group to the product of number of piles and the load capacity of single pile.

Schmidt (1981) showed that the load carried by the front pile is larger than the rear pile in pile group of two piles. Reese et al (1986) during their experiments on group of laterally loaded piles noticed that the front pile takes larger loads and the P-y curve for the rear pile is softer than that of the single pile. They explained the overlapping of the pressure zones of the two piles as the cause of this behaviour. According to their hypothesis, the two pressure bulbs from the two piles caused weakening of the soil in between the two piles and thus the rear pile carried less load than the front pile.

Byrne et al (1986), Yan and Byrne (1990) suggested that casings and platforms can be modelled by considering free field movements. According to this theory, when a pile moves the adjacent pile is subjected to the free field movements. These movements affect the behaviour of the adjacent pile. Previously it has been seen that pulling adjacent pile away has no effect on the pile in consideration. But in this case since the two piles are joined at the top, thus enforcing equal displacements, there are two effects. One effect is that the free field movements caused by PILE 1 are acting on PILE 2 and in reverse movements of PILE 2 are causing a forced free field movement of PILE 1. This concept is utilized in analysing the pile response in CHAPTER 8.

7.2.4.2 P-y CURVES

Figure 7.38 shows the P-y curves for PILE 1 in the pile group of two piles. The curves are given for a spacing of 2d and for different depths. Figure 7.39 shows the P-y curves for PILE 1 for a pile group of two piles with a spacing of 4d.

If we compare the two figures, the response of the PILE 1 in both the tests is similar. Comparing the response of PILE 1 with that of the single pile we observe that the response of the PILE 1 in the pile group is softer than that of the single pile.

As explained before Reese et al (1986) suggested that this

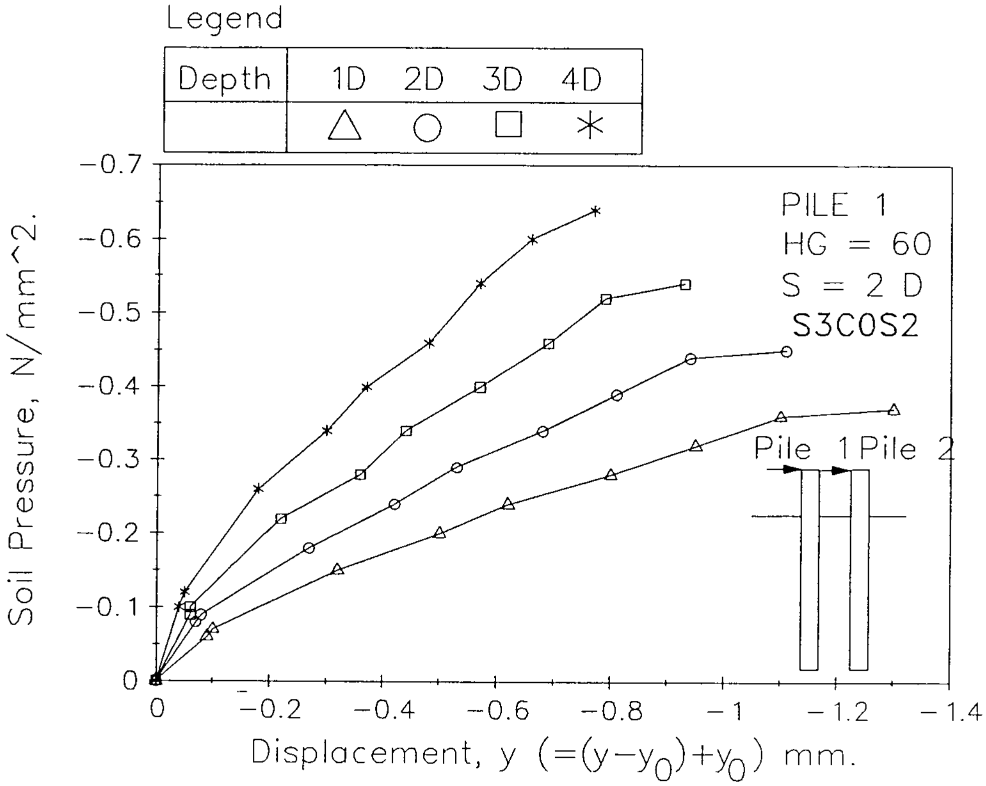


Figure 7.38 P − y Curves (S/D = 2)

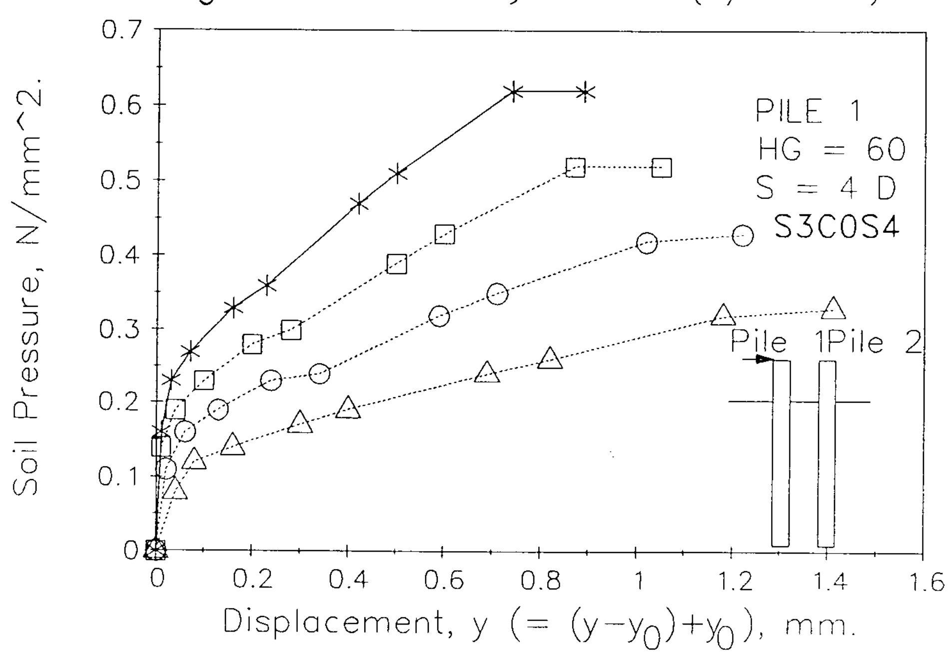


Figure 7.39 P − y Curves (S/D = 4)

is due to the overlapping of the pressure zones form the two piles in the pile group. But as seen from CASE 2, the pile installationcauses densification of the soil in between the two piles. Hence this softening is due to the free field displacements imposed by the equal displacement conditions on the piles in the pile group.

7.2.4.3 Bending moment Profile

The bending moment profile for the trailing pile (PILE 1) is shown in figure 7.40 and figure 7.41 compares it with the profile of single pile. The figure shows the bending moment and shear force profiles for a trailing pile when both the piles are given same displacements under different loads.

When a pile group is subjected to lateral load the pile group response is effect of

1. Pile-soil-pile interaction
2. Pile-pile cap-pile interaction
3. Axial push-pull effect on the pile group due to the eccentricity of the load.

In the tests conducted the pile head connections were free connections. In this connection, the pile head is allowed to rotate freely. Due to this special type of connection only pile-soil-pile interaction was affecting the group response. As seen in the profiles for the single piles the maximum bending moment is about 3 to 4 diameters below the ground surface. In figure 7.41, it can be seen that the maximum bending moment in

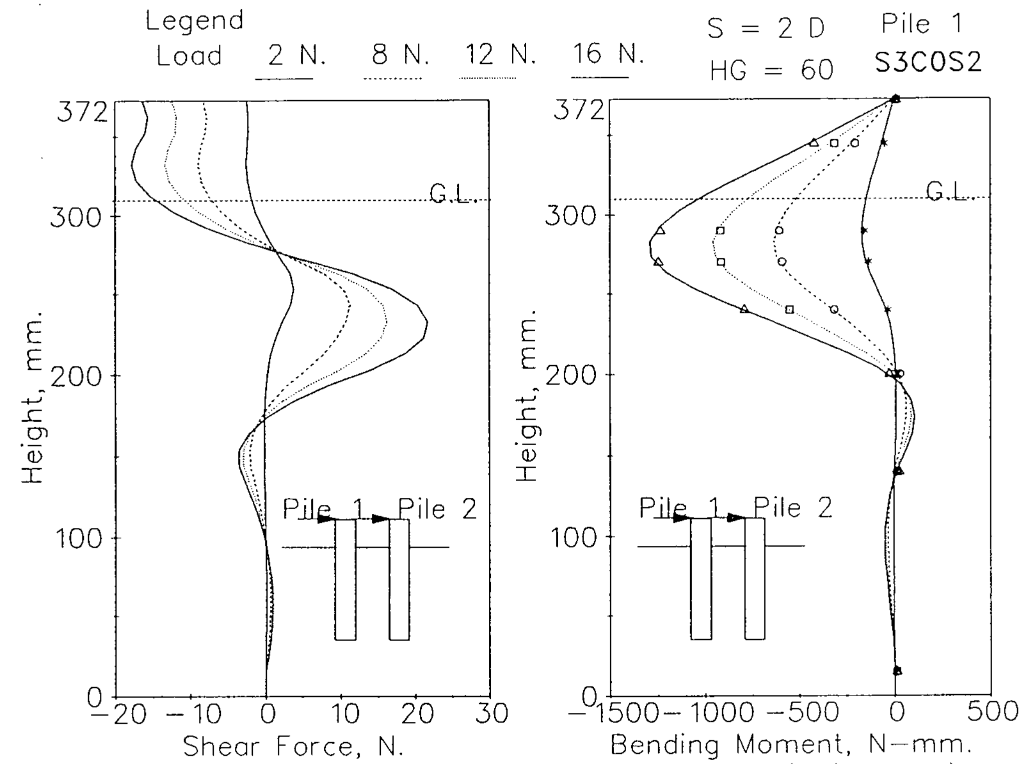


Figure 7.40 Bending Moment Profile (S/D = 2)

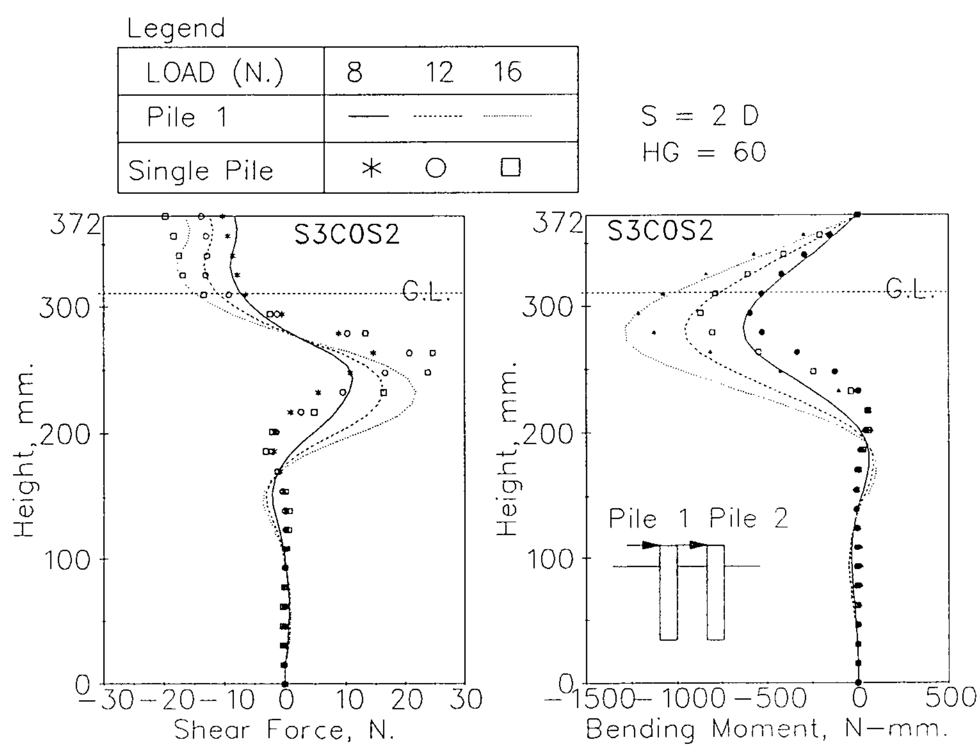


Figure 7.41 Comparison Of Bending Moment

the pile remains the same irrespective of whether the pile was used in the pile group or alone. The only difference is that in the pile used in pile group there is downward shift in the maximum bending moment.This shift can be seen in more prominently in the shear force profiles.

In their study Kulkarni et al(1985) subjected a two pile group to a lateral load in line with the piles. They observed that the bending moment in the rear pile is much less than that in the front pile. Their results show that the bending moment in the rear pile is less than half that of the front pile. According to the elastic solution, both the piles should carry equal load and the bending moments developed in both the piles should be equal. Due to the load transfer mechanism between two piles observed before in the Series 2, the rear pile carries much less load than the front pile. If we compare the maximum bending moment of the front pile and the rear pile at the equal load, we can observe that the both the piles develop equal bending moment.

The figure 7.42 gives the deflection and soil pressure profiles of the trailing pile. Again it can be seen that the length of the pile model below 150mm is practically ineffective in resisting the horizontal load at the top. Similarly the bending moment, shear force profiles for spacing of 4d are given in figure 7.43. When the trailing pile response of the pile group with 4d spacing is compared with that of the single pile in figure 7.44 as expected there is a smaller shift between the

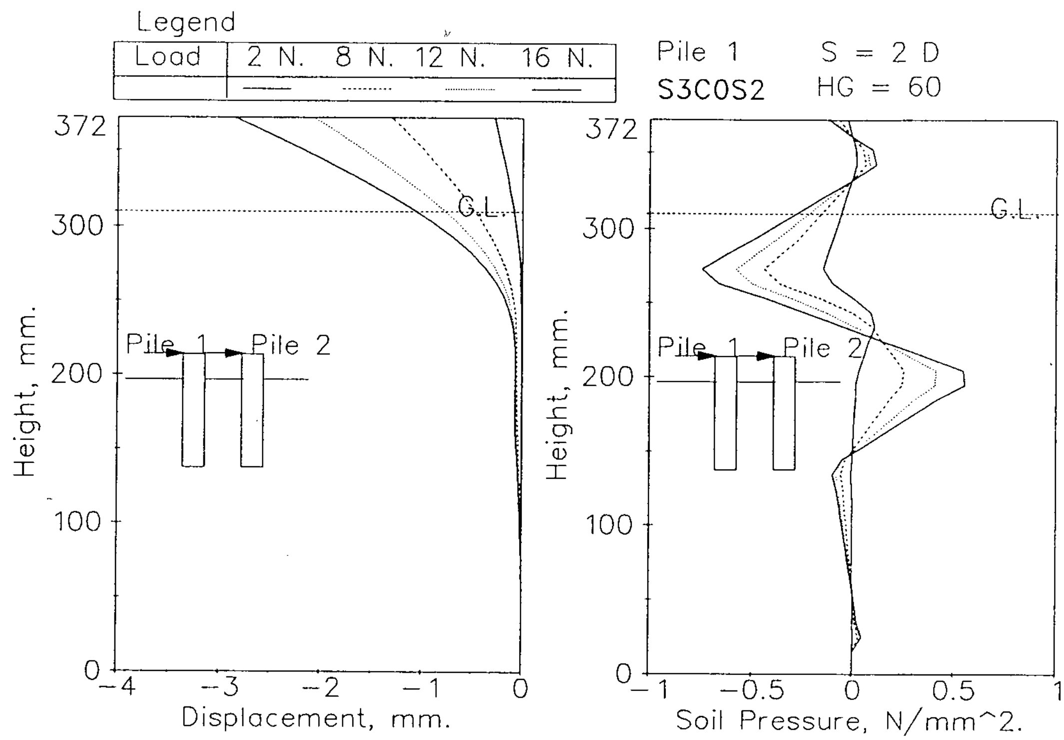


Figure 7.42 Deflection Profile (S/D = 2)

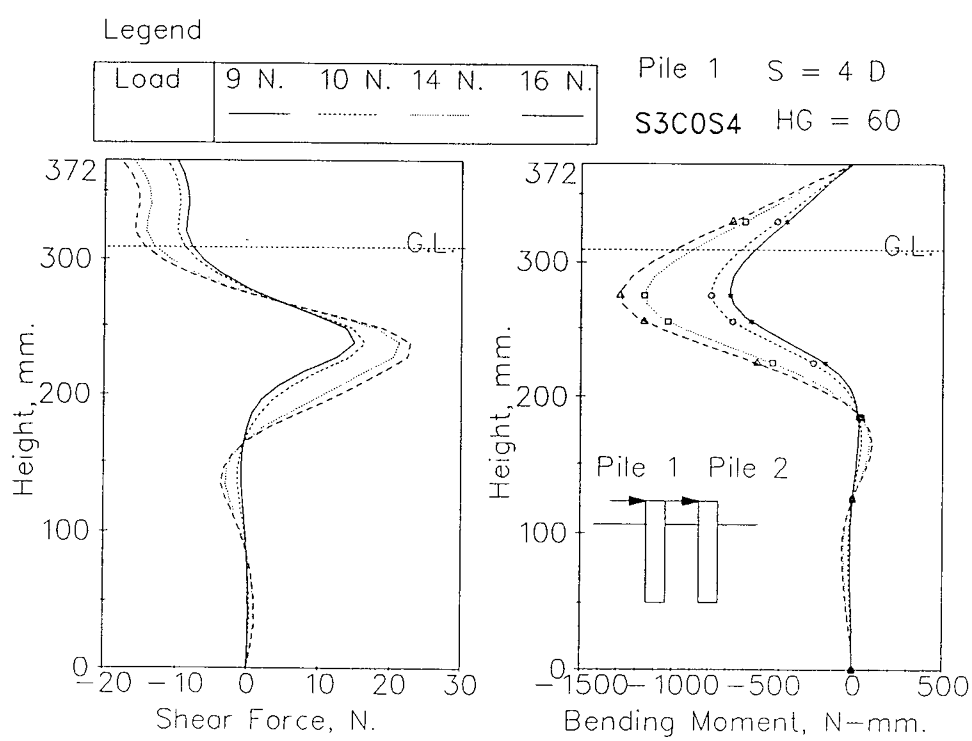


Figure 7.43 Bending Moment Profile (S/D = 4)

maximum bending moment of the two piles than the pile group of spacing 2d. If figures 7.41 and 7.44 are compared, it can be seen that the difference in depth of maximum bending moment between PILE 1 and single pile reduces as the spacing is increased. Also the amount of the negative bending moment developed in the mid-section of the pile is much less in the pile group than the single pile.

The deflection and soil pressure profiles for the trailing pile in pile group of spacing 4d are given in the figure 7.45. The figure 7.46 compares the bending moment and shear force profiles for the SERIES 1, 2,and 3 for the pile group of spacing 2d. It can be seen that in the SERIES 1 and 3 the maximum bending moment is of the same magnitude while in the SERIES 2 the bending moment in the pile is reduced considerably due to the presence of the adjacent pile.

This is due to the fact that, since in SERIES 2 the second pile is not loaded, part of the bending moment is transferred to the adjacent pile and at the same time, densification of the soil also reduces the bending moment in the pile. In SERIES 3, although the surrounding soil is densified, the effect is not taken into account since the pile in front(PILE 2) is also moving the same distance (at ground level) in the same direction. Similar, observations can be made in the figure 7.47 for a pile group of spacing 4d. Figure 7.47 compares the bending moment and shear force profiles of the Pile 1 in all the three cases.

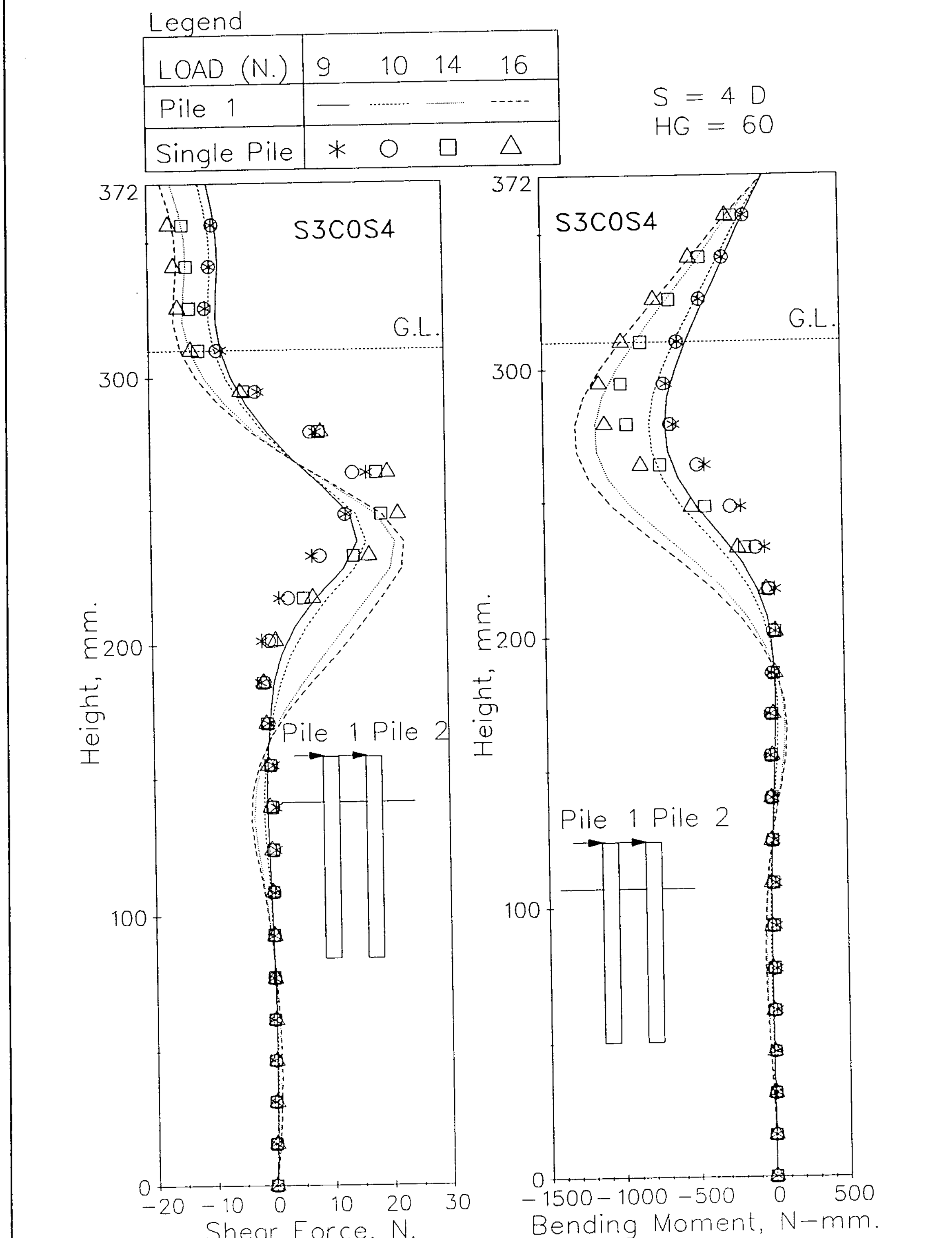


Figure 7.44 Comparison Of Bending Moment

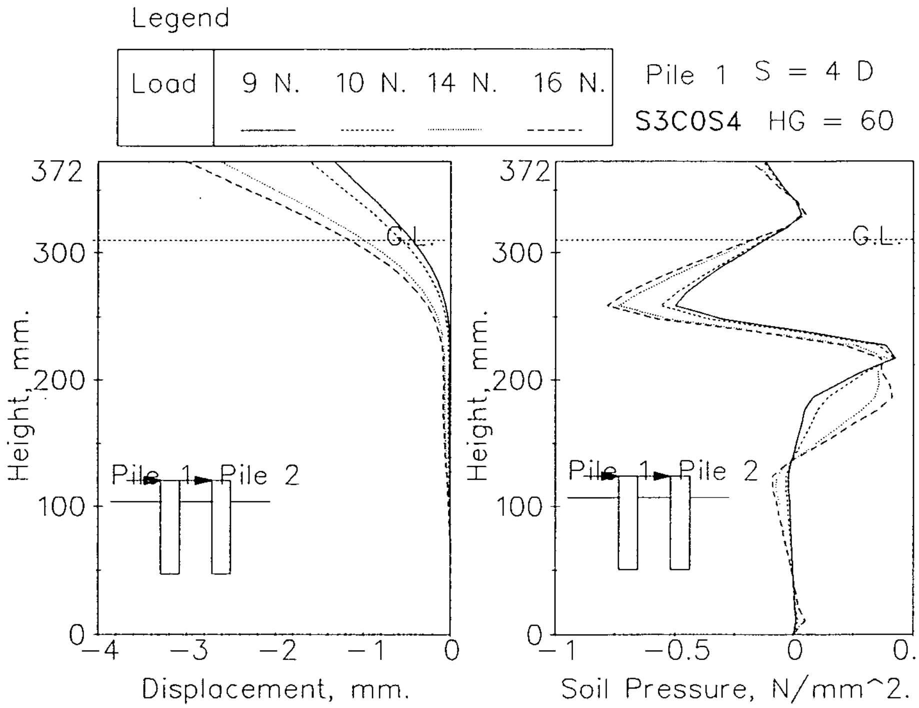
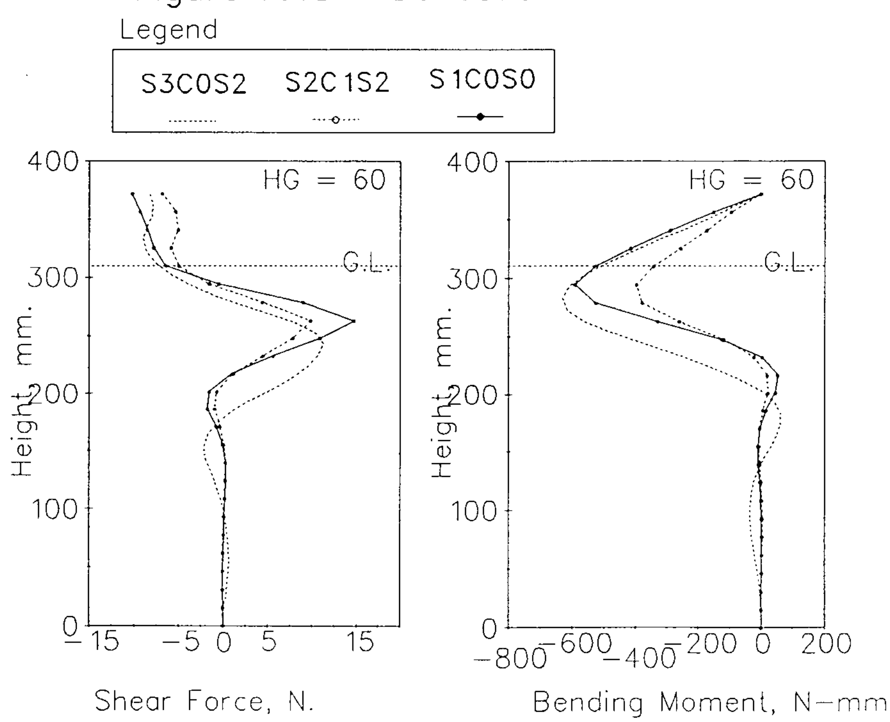


Figure 7.45 Deflection Profile



S3C0S2 :− Pile Group With Both Piles Loaded S=2d
S2C1S2 :− Pile Group With One Pile Pushed S=2d
S1C0S0 :− Single Pile

Figure 7.46 Comparison Of Bending Moment

7.3 SUMMARY AND CONCLUSIONS

This chapter gives the results of the lateral load tests conducted on the single pile and group of two piles. The tests were conducted at a hydraulic gradient of 60 and soil comprised of the fine Ottawa sand. The tests were conducted in three series

- SERIES 1 - Single pile loaded horizontally
- SERIES 2 - Single pile loaded horizontally with an adjacent pile present.
- SERIES 3 - Pile group of two piles loaded horizontally

The results of the tests were compared with the single pile results. In the series 2 the piles were tested at three angles, 0°,90° and 180°. It was found that the loading in the 90° or 180° has very little effect on the adjacent pile. In both these cases, both the piles showed no effect of the presence of the adjacent pile on either the load-displacement response or changes in stress conditions.

In case of the 0° of loading, it was found out that the installation of adjacent pile tends to increase the density of the surrounding soil. Due to this effect the response of the pile in SERIES 2 was stiffer than the single pile. Since in SERIES 3 both the piles in the pile group are loaded and the displacements of both the piles are made equal it was important to notice the load sharing of the two piles. It was seen that, the front pile of the two piles shared a large amount of the

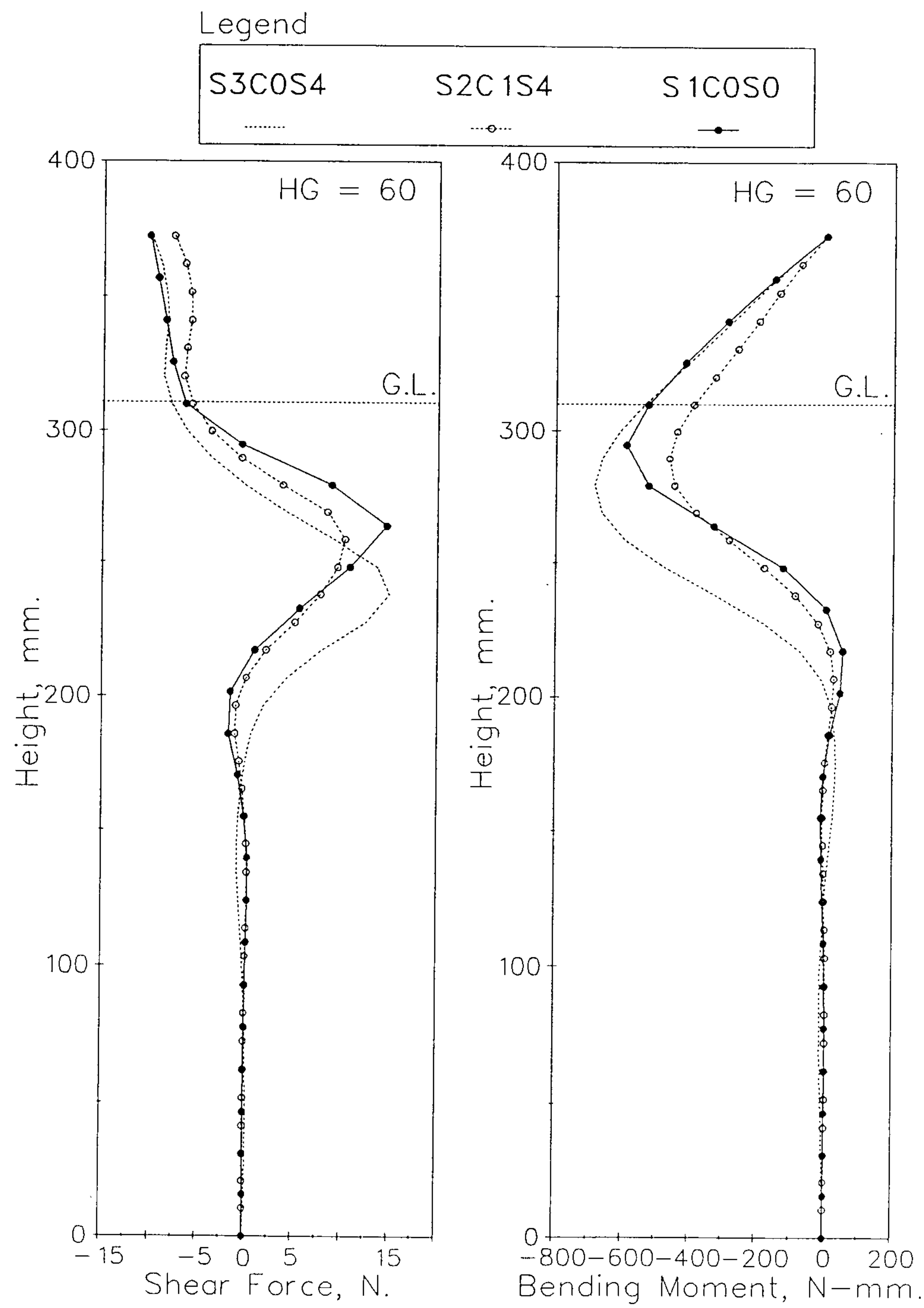


Figure 7.47 Comparison Of Bending Moment

S3C0S4 :— Pile Group With Both Piles Loaded S=4d

S2C1S4 :— Pile Group With One Pile Pushed S=4d

S1C0S0 :— Single Pile

load. The response of the front pile was found to be similar to the response of the single pile. The densification effect due to the installation of adjacent pile was not observed for the front pile.

In case of the rear pile, it was noticed that the rear pile shares very little amount of the total load applied to the group at a small spacing. As the spacing increases, the amount of load shared by the rear pile also increases. This may be due to the fact that the two piles were subjected to the same displacements. From SERIES 2, it is seen that the same rear pile showed stiffening effects, due to the densification of sand during pile group installation. Irrespective of this effect, the rear pile showed a very response to the pile group loads. This is mainly due to the reason that most of the load applied to the soil is taken by the front pile and as the soil in front of the front pile is softer than the soil in between the two piles, it yields first. As soon as this yielding starts, the front pile starts moving with the soil and the rear pile and the soil mass in between also starts moving without actually reaching the yield point.

It was observed during the loading of the pile group that the depth of the maximum bending moment developed in the pile increases with the decreasing spacing. This is due to the fact that as the spacing decreases the front pile shares the load applied on the rear pile through the soil. Due to this the maximum bending moment in the front pile in the pile group is greater than the single pile and at greater depth.

In the next chapter an attempt is made to predict the behaviour of the pile in the pile group and to predict the bending moments and shear forces in the pile correctly using the LATPILE program.

CHAPTER 8 : PREDICTION OF THE PILE GROUP RESPONSE

8.1 INTRODUCTION

In this chapter, the results from the analysis of the pile group of two piles using the program LATPILE are compared with the experimental results. The details of the analysis are explained below. The program LATPILE is first used for the single pile test data. The LATPILE program uses the P-y curve approach to analyze the pile. The P-y curves are specified at various depths, the boundary conditions are given and then the appropriate loads are applied on the pile. The program and the concept used in the analysis are discussed briefly before the discussion of the prediction results.

8.2 THE FREE FIELD CONCEPT AND ITS APPLICATION

The LATPILE program uses the P-y curve approach for analysing the pile foundation. In this approach the soil system is replaced by a system of horizontal nonlinear springs. The spring is attached at one end to the pile and at the other end to a free end instead of fixed support. Thus free field movements cause the spring ends to deflect resulting in load and deflection in the piles.

The relationship between the force and deflection of the spring of stiffness K is given by

$P = K\ y$ .... eq. (8.1)

The displacement of the free field ends of the springs are assumed to be known. Thus the soil reaction term is now given by

$P = k\ (y - y_0)$ .... eq. (8.2)

Where P is the Soil force per unit depth of the pile, known as soil reaction

K is the soil stiffness coefficient or the spring stiffness

y is the deflection of the pile relative to its initial position.

y_0 is the free field deflection, assumed to be known

The governing equation becomes

$$E\,I\,\frac{d^4y}{dx^4} + P_x\,\frac{d^2y}{dx^2} + k\,(y - y_0) = 0 \qquad \text{....... eq. (8.3)}$$

where E is the Young's modulus of the pile;

I is the second moment of area of the pile about its neutral axis;

P_x is the axial force in the pile and

x is vertical coordinate.

This equation can be rewritten as follows

$$E\,I\,\frac{d^4y}{dx^4} + P_x\,\frac{d^2y}{dx^2} + k\,y = k\,y_0 = p_0 \qquad \text{....... eq. (8.4)}$$

This equation represents a condition when the spring ends are fixed and the pile is subjected to a lateral load k y_0. If the soil is linear then the soil spring stiffness is constant and since y_0 is known, the response of the pile can be obtained easily.

These deflections of the free field ends of the springs may be due to ground movement or loads from adjacent piles. If this concept is applied to the pile group analysis then it can be said that the load on one pile causes a free field displacement in the adjacent pile which if determined can be used as the free field deflection of the pile. Free field deflection is defined as the deflection at the location of a pile that would occur from loading adjacent piles. Thus the analysis of a pile within a group now reduces to calculating the response of the single pile under its applied load together with its response to the free field deflections arising from the loads on the other piles in the pile group.

In this chapter an attempt is made to analyze the pile group of two piles using the above concept.

8.3 PREDICTION OF PILE RESPONSE

The P-y curves developed from the single pile test results were used in the LATPILE program. The analysis was carried out in three steps as follows.

In the first step the response of the single pile under a horizontal load was compared with the experimental results and is given in Figure 8.1. It was found that LATPILE predicts the response of the single pile very accurately if the P-y curves obtained from the experiments are used.

In the second step, pile response to loads on adjacent piles was analysed. The pile displacement, moment and shear force profiles were obtained from SERIES 2. In the LATPILE analysis, the pile was subjected to free field displacement equal to the pile displacement observed in the experiment. The difference between the input free field displacement and the pile displacement was very small. The bending moment and shear force profiles from the analysis were compared with the profiles obtained from the experiments. Figures 8.2 to 8.6 give comparisons of various load cases and spacings. It should be noted that although the distance between the piles is increasing and the bending moment is reducing considerably, the LATPILE predictions are very good.

In the third step, LATPILE program was used to analyse the response of a single pile in group. The pile was analysed under a combination of load and free field displacement. At this point it should be remembered that the loads on both the piles are different. Therefore the free field displacement will correspond to a different magnitude of load than the applied load.

8.3.1 COMPARISON OF BENDING MOMENT

The comparison between the experimental results for the

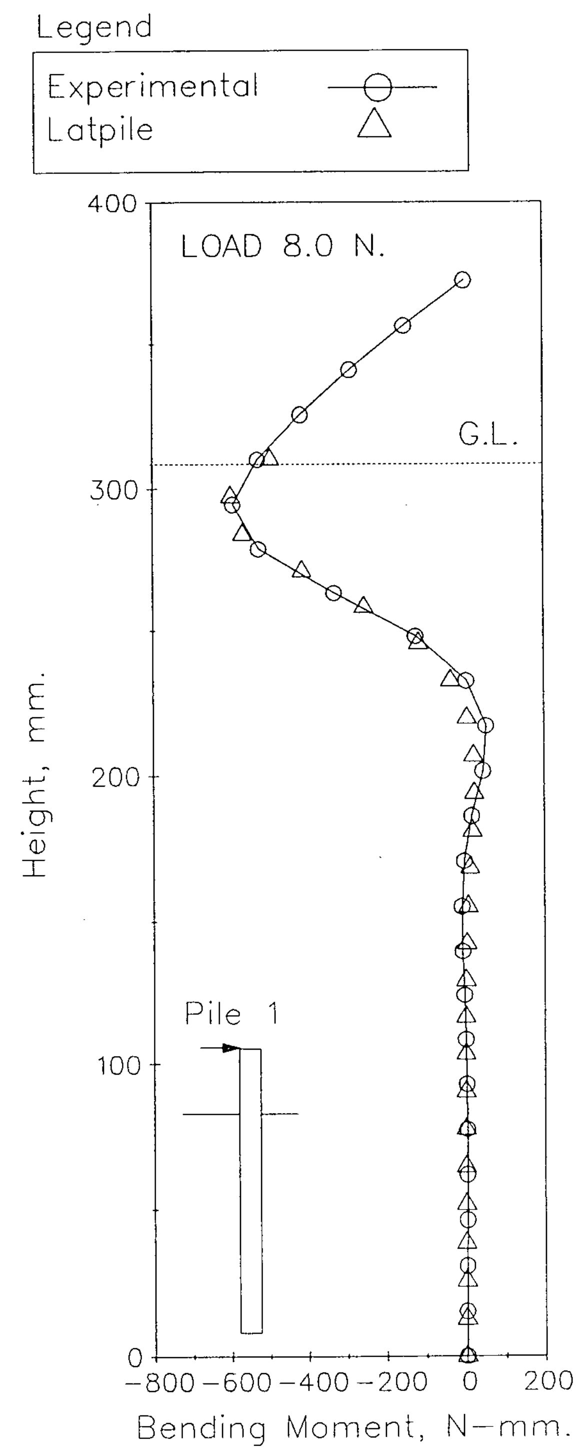


Figure 8.1 Comparison Of Bending Moment From Experiment and Latpile For Single Pile

trailing pile and single pile are given in figures 8.7-8.8 with the LATPILE prediction for the trailing pile. It is quite clear that although the program is predicting the maximum bending moment very closely the bending moment profile along the length of the pile is quite different from that obtained form the experiment. During the loading of the pile it is observed that due to the pile soil gaping the depth of maximum bending moment tends to increase along the pile length. This may account for the small discrepancy in the analytical results and the experimental results. One more important consideration is that the adjacent pile in SERIES 2 showed no effect when the loaded pile was pulled away for the purposes of analysis. In the experiment however both the piles were displaced equally hence for the purposes of analysis it was assumed that the free field effect in either direction will be same.

8.3.2 LOAD DISPLACEMENT RESPONSE

The load displacement curves were used to calculate the interaction factor α which is then compared with the interaction factor from various methods. The comparison is given in figure 7.14.

The interaction factors are calculated using methods based on the elastic theory. These methods are discussed in detail in CHAPTER 3. The interaction factors were calculated for a pile group of two piles with the load applied at an eccentricity of

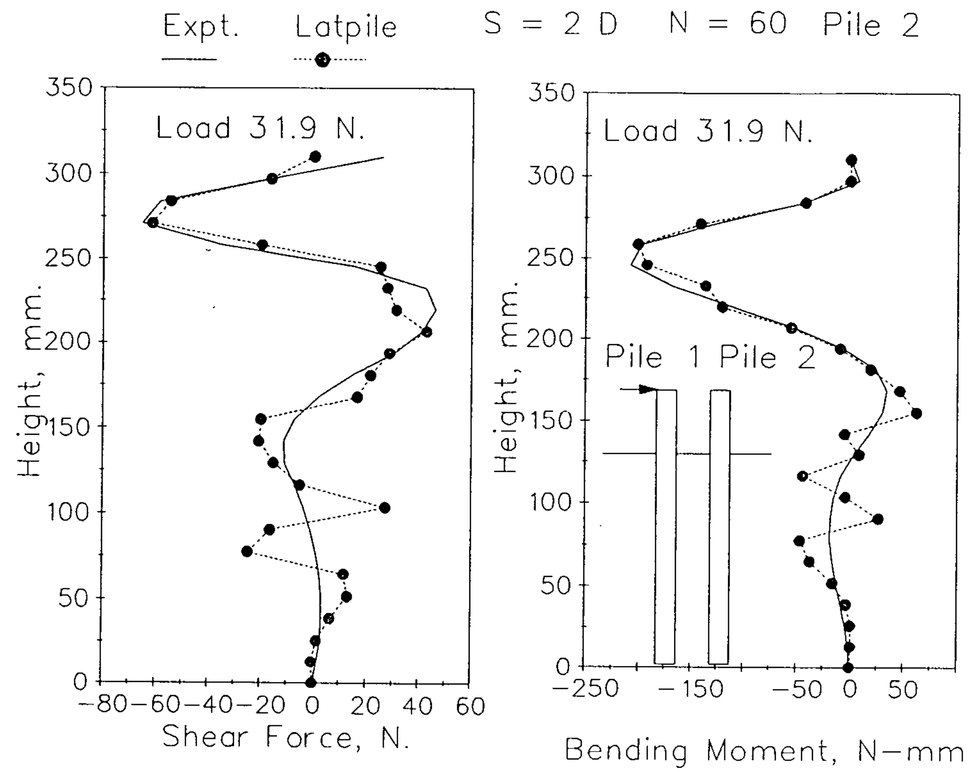


Figure 8.2

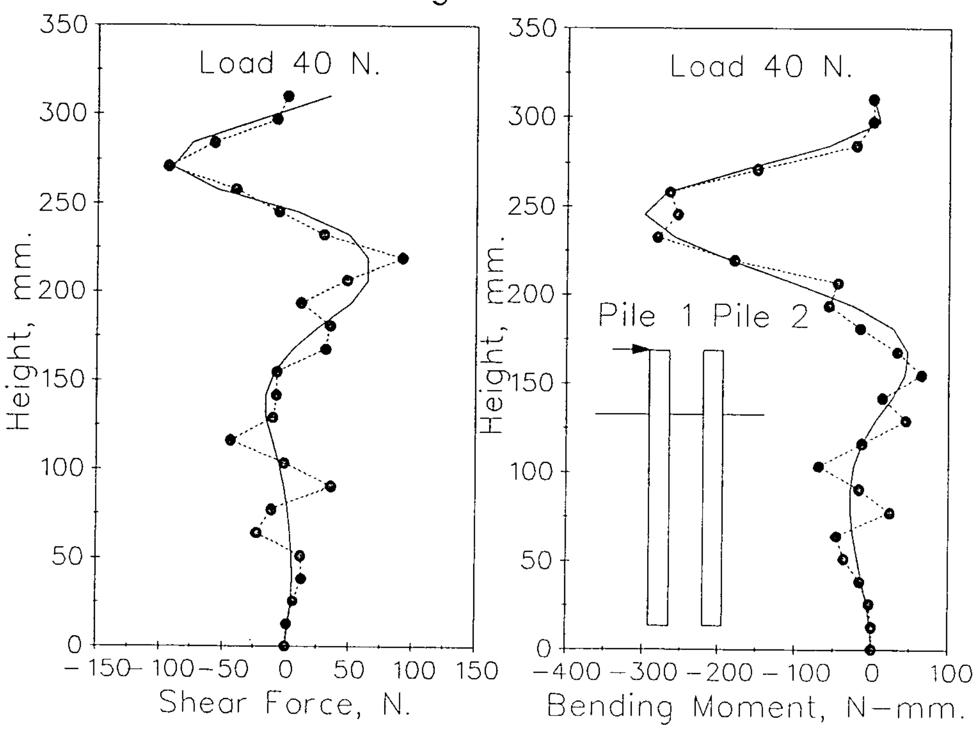


Figure 8.3

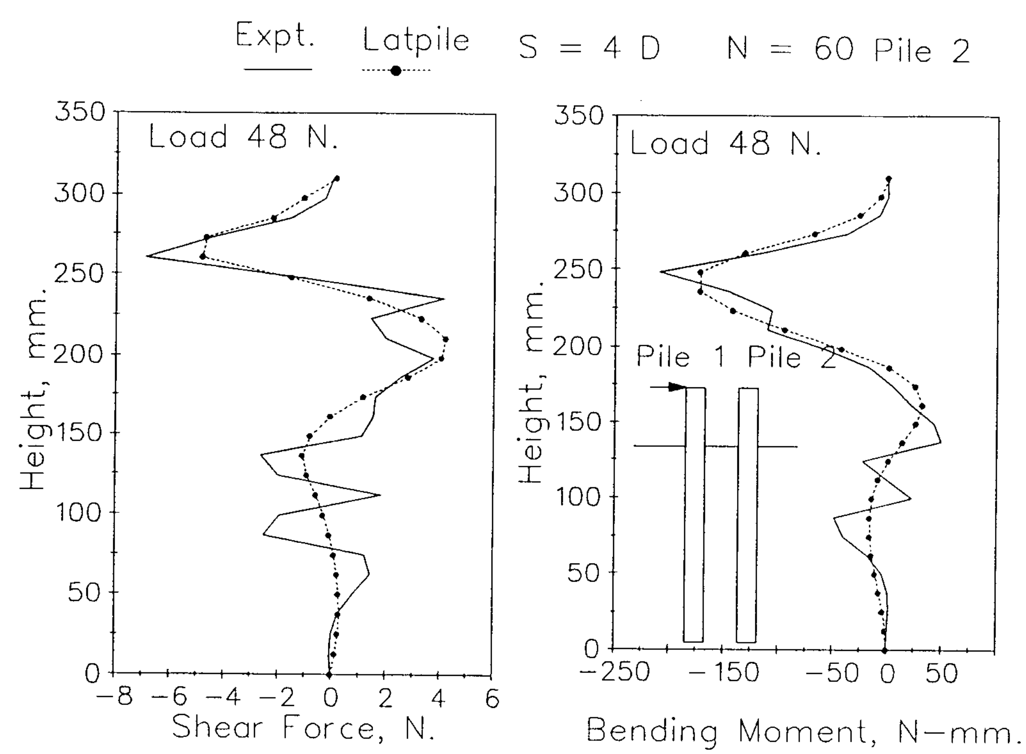


Figure 8.4

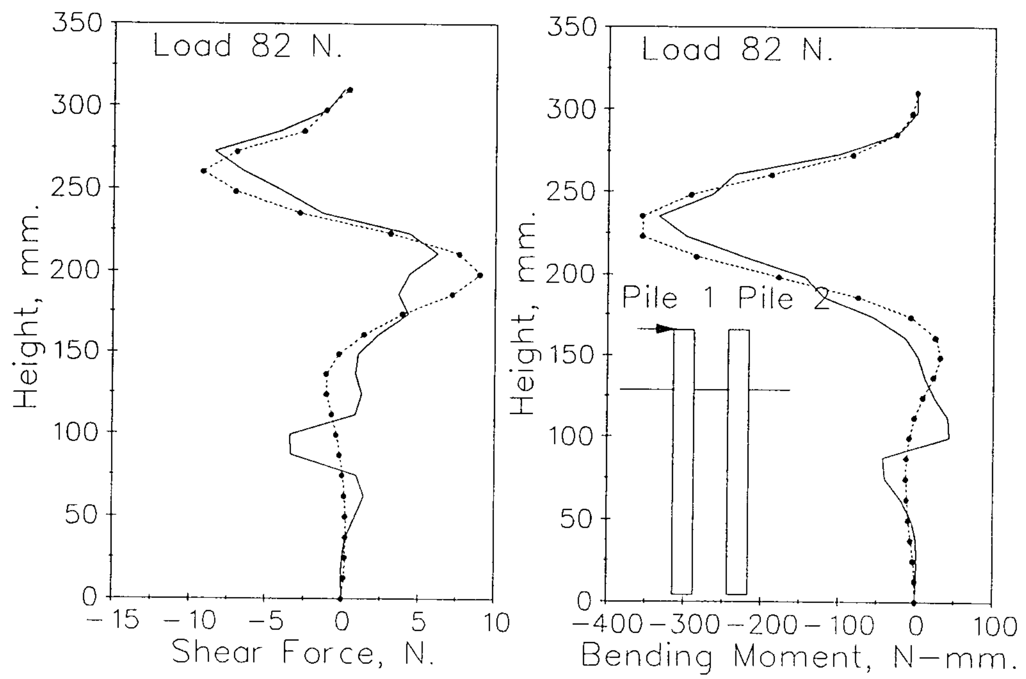
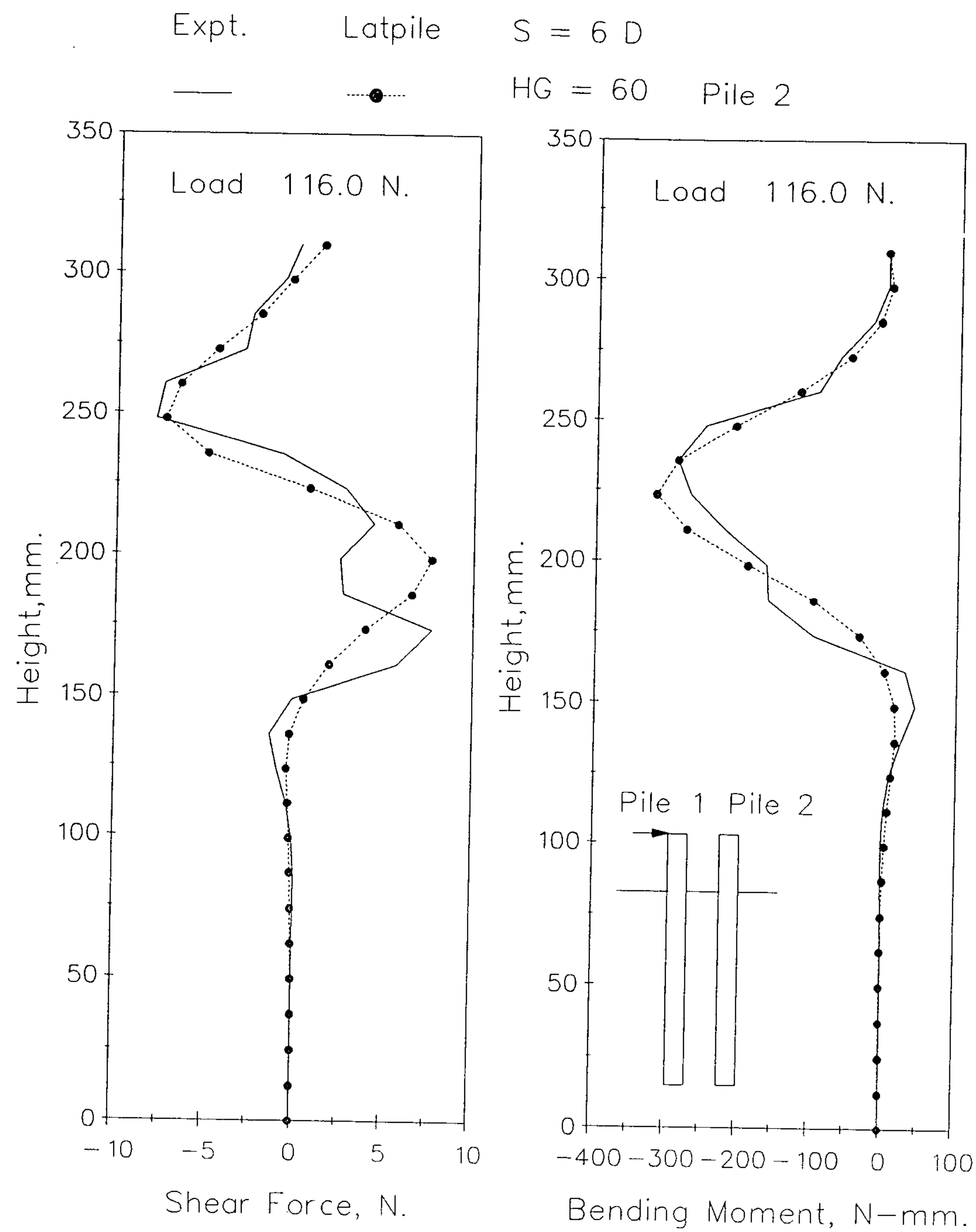


Figure 8.5

62mm. The effect of the boundary was taken into consideration while considering the interaction factors.

8.4 SUMMARY

In this chapter, the results of the SERIES 2 were used to analyze pile groups in SERIES 2 and 3. In SERIES 2, very good results were obtained, whereas in SERIES 3 some difference was observed although this difference was very small. In both the cases, the induced displacements of the pile were obtained from the SERIES 2 and were used as the free field displacements in LATPILE program. The program calculated the pile displacements and bending moments based on the free field as explained earlier. The results were remarkably good for the back analysis of SERIES 2. The bending moments and the displacement profiles of the pile were exactly as recorded during the experiment. In SERIES 3, the slight difference in the profiles of the LATPILE results and the experimental data is due to the fact that in addition to the free field imposed on the pile, the densification of the soil in front of the rear pile has to be taken into account. Since most of the load is being carried by the front pile, and the soil ahead of the front pile is not affected the results of the LATPILE are fairly close to the experimental data. A more detailed study on the applicability of this method to various cases of pile groups is required.



Expt.
Latpile
S = 6 D
HG = 60
Pile 2
Load 116.0 N.
Load 116.0 N.
Height,mm.
Height,mm.
350
300
250
200
150
100
50
0
−10
−5
0
5
10
−400
−300
−200
−100
0
100
Pile 1 Pile 2
Shear Force, N.
Bending Moment, N−mm.

Figure 8.6

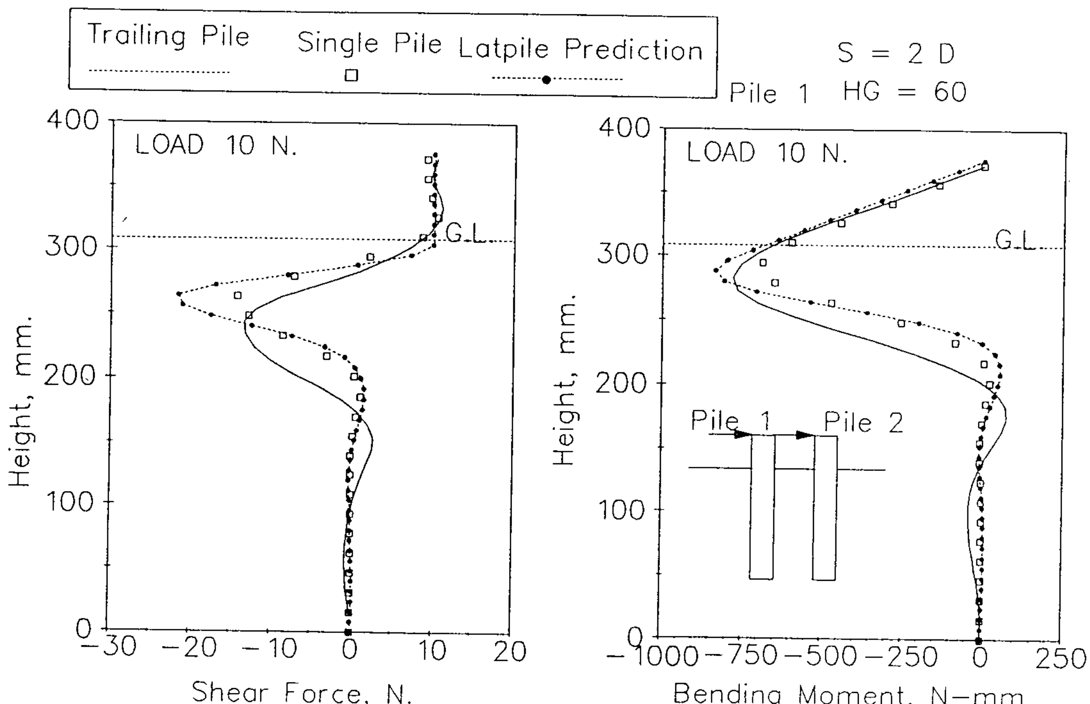


Figure 8.7

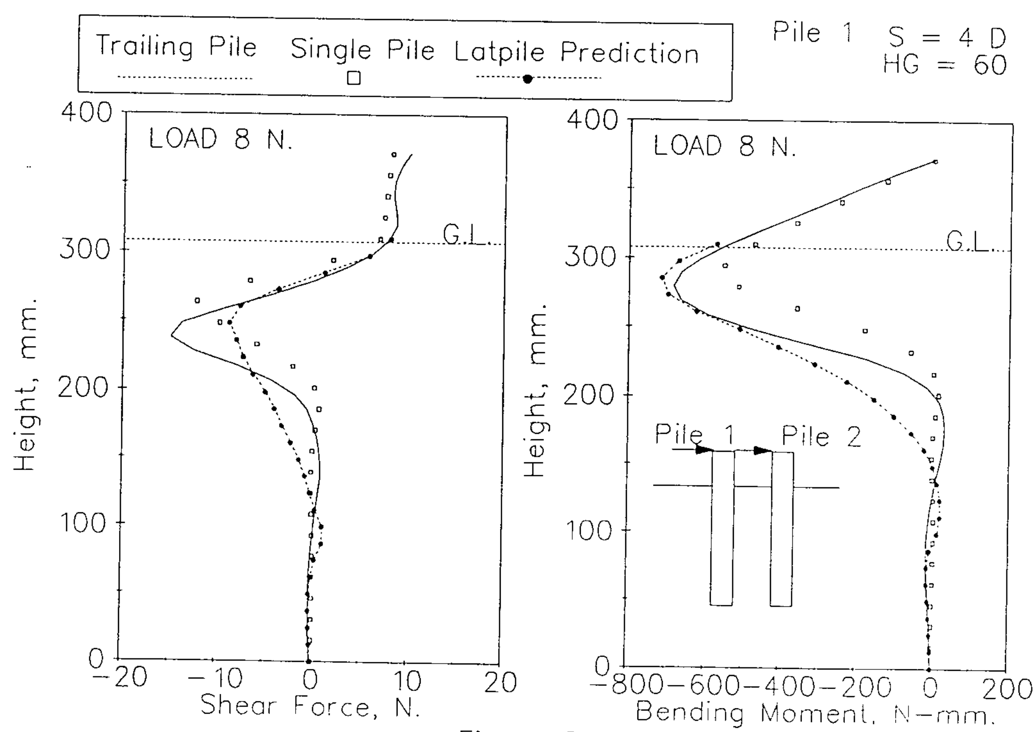


Figure 8.8

CHAPTER 9 : SUMMARY AND CONCLUSION

In this study the pile group response to the lateral load is studied in the laboratory using the Hydraulic Gradient Similitude Testing Device. While conducting model tests, it is important to conduct the test at field stress level of the soil sample so as to get realistic results. The HGS method is a very effective way to conduct a model test at field stress levels.

The objectives of the study were to generate a database for the pile group response under lateral loads and study the interaction between two piles as well as pile-soil interaction effects. It was found that the hydraulic gradient test results are repeatable and reliable.

The Hydraulic Gradient Similitude Method is a very simple and cost-effective way of conducting model studies. The main drawback is that, it can be used only for soil-structures consisting of uniform sand with horizontal flow boundaries. But even with these restrictions a vast number of problems including the pile groups can be studied very effectively. Some of the applications of the Hydraulic Gradient Similitude modelling technique are as follows

- Pile group response to cyclic loading as well as earthquake loading.
- Response of single pile and pile groups in liquefied material.

- Effects of pile driving on the pile capacity and the energy transfer during the pile driving.

Tests were conducted on single piles, single piles in the presence of adjacent pile and pile group comprised of two piles. The results from all the tests were discussed in the chapter 7. Tests were conducted with various spacings between the piles and with various angles of loading.

From the tests on the single pile, bending moment, shear force and deflection profiles were obtained. When the single pile was loaded in the presence of adjacent pile, it was found that the adjacent pile increases load resistance by its own stiffness and by densifying the soil surrounding the pile. The effect of the adjacent pile depends on its position relative to the loaded pile. When the adjacent pile was at 90° to the applied load or when the adjacent pile was away from the direction of the applied load it was observed that the adjacent pile has no effect on the pile response.

In the pile group testing, it was seen that the trailing pile shared less load than the leading pile. As the pile spacing increased the percentage of load carried by the trailing pile also increased. The maximum bending moment developed in the leading pile is at a lower depth than that of the single pile. Also the maximum bending moment in leading pile is similar to that developed in single pile.

Thus whether the piles are loaded simultaneously or not, loading a trailing pile will cause deflection of the leading pile and bending moment will be developed in the leading pile as consequence of the load applied on trailing pile. Whereas the loading of leading pile will not affect trailing pile except through pile-cap-pile interaction.

All the test results were compared with the results from LATPILE analysis. In LATPILE analysis P-y curves obtained from the single pile test were used. LATPILE analysis was used to predict single pile response, response of the adjacent pile and response of the leading pile in the pile group. It was seen that LATPILE predictions and test results agree quite well for single pile as well as for the adjacent pile. In case of the leading pile in a pile group, although the maximum bending moment predictions were good, the bending moment profile is quite different than the experimental profile.

REFERENCES

1 Alizadeh, M. and Davisson, M.T. - Lateral Load Tests on Piles - Arkansas River Project, Journal Of Soil Mechanics and Foundation Division, ASCE, 1970, Vol. 96, SM5, pp. 1583-1604.

2 API-RP2A - Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms. American Petroleum Institute, Washington, D.C., 17th Ed., Aprill, 1987.

3 Aurora R.P. - Lateral Response of a Group Of Large Diameter Piles - A Case Study of a North Sea Platform; Fifteenth Annual Offshore Techonology Conference in Houston, Texas, May 2-5, 1983.

4 Barton Y.O., Finn W.D.L., Parry R.H.G. and Towhatta I. - Lateral Pile Response and P-y Curves from Centrifuge Tests, Proceedings of 15th Annual Meeting of Offshore Technology Conference, Vol. 1, pp.503-508,1983.

5 Barton Y.O. - Laterally Loaded Model Piles In Sand; Centrifuge Tests and Finite Element Analyses, Ph.D. Thesis, Cambridge University, Engineering Department, June 1982.

6 Broms B.B. - Design of Laterally Loaded Piles; Journal of the Soil Mechanics and Foundation Division; Proceedings of ASCE, SM3, May 1965, pp.79-99.

7 Brown D.A., Morrison Clark and Reese L.C. - Lateral Load Behaviour of Pile Group in Sand; Journal of Geotechnical Engineering, Vol.114, NO.11, November 1988.

8 Brown D.A., Morrison C. and Reese L.C. - Response of a Single Pile in Sand Subjected to Cyclic Lateral Loading, Symposium on Offshore and Arctic Operations, The American Society of Mechanical Engineers, New York, pp139-143, 1987.

9 Butterfield R. and Banerjee P.K. - The Elastic Analysis of Compressible Piles and Pile Groups; Geotechnique 21, No. 1, pp. 43-60(1971).

10 Byrne P. M., Anderson D.L. and Janzen W. - Response of Piles and Casings to Horizontal Free-field soil Displacements; Canadian Geotechnical Journal, Vol. 21, 1984, pp. 720-725.

11 Chow Y.K. - Iterative Analysis of Pile soil Interaction; Geotechnique 37, No. 3, pp. 321-333, 1987.

12 Chow Y.K. - Axial and Lateral Response of Pile Groups Embedded in Non-homogeneous Soils, International Journal For Numerical and Analytical Methods in Geomechanics, Vol. 11, pp. 621-638(1987).

13 Cox W.R., Reese L.C. and Grubbs B.R. - Field Testing of Laterally Loaded Piles in Sand, Proceedings of 6th Annual OTC, Houston, Texas, Vol. 2, Paper No. OTC2079, pp.459-472, 1974.

14 Davisson M.T. and Salley J.R. - Model Study of Laterally Loaded Piles; Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM5, September, 1970,pp.1605-1627.

15 Duncan J.M. and Chang C-Y. (1970) - Nonlinear Analysis of

stress and strain in soils; Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM5, September 1970.

16 Franklin J.N. and Scott R.F. - Beam Equation with Variable Foundation Coefficient, Proceedings of ASCE, Vol. 105, EM5, pp811-827, Oct.1979.

17 Jamiolkowski M. and Garassino A. - Soil Modulus for Laterally Loaded Piles, Special Session 10, 9th Int. Conf. SMFE, Tokyo, July,1977.

18 Janes M. and Novak M. - Pile Group Response to Large Lateral Loads; Proceedings of the 42nd Canadian Geotechnical Conference, Winnipeg, Manitoba, 1989, pp. 216-222.

19 Kay s., Kolk H.J. and Van Hooydonk W.R. - Site Specific Design of Laterally Loaded Piles; Proceedings of the Conference on Geotechnical Practice in Offshore Engineering, University of Texas, Austin, Texas, April 27-29, 1983,pp. 557-580.

20 Kooijman A.P. - Comparison of an Elasto-plastic Quasi Three Dimensional Model for Laterally Loaded Piles with Field Tests; Proceedings of the 3rd International Symposium on Numerical Models in Geomechanics (NUMOG III), May 8-11, 1989, Niagara Falls, pp.675-682.

21 Krawinkler H. - State of the Art Report: Scale Effects in Static and Dynamic Model Testing of Structures; Proceedings of 9th World Conference on Earthquake Engineering, Aug. 2-9, 1988, Tokyo-Kyoto, Japan, Vol. VIII, pp. 865-876.

22 Kubo K. - Experimental Study Of the Behaviour of Laterally Loaded Piles, Proc. 6th Int. Conf. SMFE, Montreal, Vol.II, pp.275-279, 1965.

23 Kulkarni K.R., Chandrasekaran V.S. and King G.J.W. - Centrifugal Model Studies on Laterally Loaded Pile Groups in Sand; Proceedings of the XIth ICSMFE San Fransisco, August 12-16,1985, Vol. 2 Technical papers pg. 1113-1116, 2/c/4.

24 Leung C.F. and Chow Y.K. - Response of Pile Groups Subjected to Lateral Loads; International Journal For Numerical and Analytical Methods in Geomechanics, Vol. 11, pp. 307-314(1987).

25 Mattes N.S. and Poulos H.G. - Model tests on Piles in Clay, Proc. 1 st Austr.-NZ Conf. on Geomechanics, Melbourne,pp245-259, 1971.

26 Meyerhof G.G., Sastry V.V.R.N. and Yalcin A.S. - Lateral Resistance and Deflection of Flexible Piles; Canadian Geotechnical Journal, Vol. 25, 1988, pp. 511-522.

27 Muqtadir A., Desai C.S. and Scheele F. - Three Dimensional Analysis of Pile Group Foundation and Anchor-soil System; Fifth International Conference on Numerical Methods in Geomechanics, Nagoya, April 1-5, pp. 755-763,1985.

28 Murchinson J.M. and O'Naill M.W. - Evaluation of P-y Relationships in Cohesionless Soils, Proc. Symposium on Analyses and Design of Pile Foundations, San Fransisco, California, Oct. 1984.

29 Nair K., Gray H. and Donovan N. C. - Analysis of Pile Group Behaviour; Performance of Deep Foundations, ASTM STP 444, 1969, pp. 118-159.

30 Najjar Y.M. and Zaman M.M. - Effect of Loading Sequence and Soil Non-linearity on The Response of a Pile Group Foundation Using a Three Dimensional Finite Element Analysis; Numerical Methods in Geomechanics 1988 Rotterdam, pp. 1127-1134.

31 Ochoa M. and O'Neill M.W. - Lateral Pile Interaction Factors in Submerged Sand; Journal of Geotechnical Engineering, ASCe, Vol. 115, No. 3, March 1989, pp. 359-378.

32 Poulos H.G. - Behaviour of Laterally Loaded Piles : II - Pile Groups; Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, SM5, May 1971, pp. 733-751.

33 Poulos H.G. - Lateral Load Deflection Prediction for Pile Groups; Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT1, Jan. 1975, pp. 19-34.

34 Poulos H.G. - Group Factors For Pile Deflection Estimation; Journal of Geotechnical Engineering Division, ASCE, Vol. 105, No. GT12, Dec. 1979, pp.1489-1509.

35 Poulos H.G. and Davis E.H. - Pile Foundation Analysis and Design, John Wiley and Sons, New York,1981.

36 Poulos H.G. - Behaviour of Laterally Loaded Piles : I - Single Piles; Journal Of the Soil Mechanics and Foundations

Division, Proceedings of the American Society of Civil Engineers, SM5, May 1971, pp. 711-731.

37 Poulos H.G. - An Approach for the Analysis of Offshore Pile Groups; Numerical Methods in Offshore Piling, ICE, London, 1980, pp.119-126.

38 Poulos H.G. and Hull T. S. - Analytical Geomechanics in Foundation Engineering - A Study of Laterally Loaded piles; Research Report No. R667 The University of Sydney, N.S.W. 2006 Australia, December 1992.

39 Randolph M.F. - The Response of Flexible Piles to Lateral Loading; Geotechnique 31, NO. 2, 247-259(1981).

40 Reese L.C. Wright S.G. and Aurora R.P. - Analysis of a Pile Group Under Lateral Loading; Laterally Loaded Deep Foundations: Analysis and Performance, ASTM STP 835, 1984, pp. 56-71.

41 Reese L.C., Cox W.R. and Koop F.D. - Analysis of Laterally Loaded Piles in Sand; Sixth Annual Offshore Techonology Conference, May 6-8,1974.

42 Robinsky, E.I. and Morrison, C.E. - Sand Displacement and Compaction Around Model Friction Piles; Canadian Geotechnical Journal, Vol. 1, No. 2:81(1964).

43 Schmidt H.G. - Horizontal Load Tests on Files of Large Diameter Bored Piles; Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, San Fransisco, Vol. 3, Aug. 12-16, 1985, pp. 1569-1573.

44 Schmidt H.G. - Group Action of Laterally Loaded Bored

Piles; Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 2, June 15-19, 1981, pp. 833-837.

45 Scott R.F. - Foundation Analysis, Prentice Hall, Inc. Englewood Cliff, N.J., 1981.

46 Scott R.F. - Centrifuge Studies of Cyclic Lateral Load-Displacement Behaviour of Single Piles, Research Program For American Petroleum Institute, OSAPR Project 8, 1976.

47 Selby A.G. and Poulos H.G. - Some Lateral Load Tests on Model Pile Groups, Research Report No. R457, The University of Sydney, Oct. 1983.

48 Sharnouby B.L. and Novak M. - Flexibility Coefficients and Interaction Factors for Pile Group Analysis; Canadian Geotechnical Journal 23, 441-450(1986).

49 Sharnouby B.L. and Novak M. - Static and Low Frequency Response of Pile Groups; Canadian Geotechnical Journal 22, 79-94(1985).

50 Shibata, T., Yashima A., and Kimura M. - Model Tests and Analyses of Laterally Loaded Pile Groups; Soils and Foundations, Vol. 29, No. 1, pp.31-44(1989).

51 Tamura A., Ozawa Y. Sunami S. and Murakami S. - Reduction in Horizontal Bearing Capacity of Pile Group; Numerical Methods in Geomechanics, Edmonton 1982, pp. 865-874.

52 Terzaghi K. - Evaluation of Coefficients of Subgrade Reaction, Geotechnique, Vol. 5, No. 4, 297, 1955.

53 Ting J.M., Kauffman C.R. and Lovicsek M. - Centrifuge

Static and Dynamic Lateral Pile Behaviour, Canadian Geotechnical Journal, Vol. 24, pp198-207,1987.

54 Trochanis A.M., Bielak J. and Christiano P. - Three Dimensional Nonlinear Study of Piles; Journal of Geotechnical Engineering, Vol. 117, No. 3, March 1991, pp. 429-447.

55 Trochanis A.M., Bielak J. and Christaino P. - Simplified Model For Analysis of One or Two Piles; Journal of Geotechnical Engineering, Vol. 117, No. 3, March 1991, pp. 448-465.

56 Yan L. - Numerical Studies on Some Aspects with Pressuremeter and Laterally Loaded Piles, M.A.Sc. Thesis, Dept. of Civil Eng., University of British Columbia, Vancouver, Canada,1986.

57 Yan L. - Hydraulic Gradient Similitude Method For Geotechnical Modelling Tests With Special Emphasis on Laterally Loaded Piles; Ph.D. Thesis, Sept.1990.

58 Yan L. and Byrne P.M. - Laboratory Small Scale Modelling Tests using the Hydraulic Gradient Similitude Method, 1991 Geotechnical Congress, ASCE, Colorado, Denver,1991a.

59 Yan L. and Byrne P.M. - Stress State and Stress Ratio Effects on Seismic Downhole and Crosshole Tests, 2nd Int.Conf. on Recent advances in Geomechanical Earthquake Engineering and Soil Dynamics, St. Louis, March, 1991b.

60 Yoshida I. and Yoshinaka R. - A Method to Estimate Modulus of Horizontal Subgrade For a Pile, Soils and Foundation,

Vol 12, No. 3, 1972.

61 Zelikson A. - Geotechnical Models using the Hydraulic Gradient Similitude Methods, Geotechnique, Vol. 19, pp495-508, 1969.